TRANSPORTATION TECHNOLOGY SUPPORT
FOR DEVELOPING COUNTRIES

SYNTHESIS 4

Structural Design of Low-Volume Roads

prepared under contract AID/OTR-C-1591, project 931-1116,
U.S. Agency for International Development

Transportation Research Board
National Research Council

NATIONAL ACADEMY OF SCIENCES WASHINGTON, D.C. 1982

Library of Congress Cataloging in Publication Data
National Research Council (U.S.). Transportation Research Board.
Structural design of low-volume roads.

(Transportation technology support for developing countries. Synthesis; 4)
"Prepared under contract AID/OTR-C-1591, project 931-1116, U.S. Agency for International Development."
1. Underdeveloped areas–Roads–Design. I. Title. II. Series.
TE175.N32 1982 625.7'4'091724 82-14082
ISBN 0-309-03321-7

Notice

The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competence and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

Cover photo: Labor-intensive road surfacing in Kenya (courtesy of TRRL, United Kingdom).

Contents

Project Description

The development of agriculture, the distribution of food, the provision of health services, and the access to information through educational services and other forms of communication in rural regions of developing countries all heavily depend on transport facilities. Although rail and water facilities may play important roles in certain areas, a dominant and universal need is for road systems that provide an assured and yet relatively inexpensive means for the movement of people and goods. The bulk of this need is for low-volume roads that generally carry only 5 to 10 vehicles a day and that seldom carry as many as 400 vehicles a day.

The planning, design, construction, and maintenance of low-volume roads for rural regions of developing countries can be greatly enhanced with respect to economics, quality, and performance by the use of low-volume road technology that is available in many parts of the world.

In October 1977 the Transportation Research Board (TRB) began a special project under the sponsorship of the U.S. Agency for International Development (AID) to enhance rural transportation in developing countries by providing improved access to existing information on the planning, design, construction, and maintenance of low-volume roads. With advice and guidance from a project steering committee, TRB defines, produces, and transmits information products through a network of correspondents in developing countries. Broad goals for the ultimate impact of the project work are to promote effective use of existing information in the economic development of transportation infrastructure and thereby to enhance other aspects of rural development throughout the world.

In addition to the packaging and distribution of technical information, personal interactions with users are provided through field visits, conferences in the United States and abroad, and other forms of communication.

STEERING COMMITTEE

The Steering Committee is composed of experts who have knowledge of the physical and social characteristics of developing countries, knowledge of the needs of developing countries for transportation, knowledge of existing transportation technology, and experience in its use.

Major functions of the Steering Committee are to assist in the definition of users and their needs, the definition of information products that match user needs, and the identification of informational and human resources for development of the information products. Through its membership the committee provides liaison with project-related activities and provides guidance for interactions with users. In general the Steering Committee gives overview advice and direction for all aspects of the project work.

The project staff has responsibility for the preparation and transmittal of information products, the development of a correspondence network throughout the user community, and interactions with users.

INFORMATION PRODUCTS

The two major products of this project are compendiums of previously published information on relatively narrow topics and syntheses of knowledge and practice on somewhat broader subjects. Compendiums are prepared by project staff, consultants are employed to prepare syntheses. In addition, proceedings of international conferences on low-volume roads are prepared and transmitted to the project correspondents. In summary, this project aims to produce and distribute between 20 and 30 publications that cover much of what is known about low-volume road technology.

INTERACTIONS WITH USERS

A number of mechanisms are used to provide interactions between the project and users of the information products. Review forms are transmitted with each publication so that recipients have an opportunity to say how the products are beneficial and how they may be improved. Through visits to developing countries, the project staff acquires first-hand suggestions for the project work. Additional opportunities for interaction with users arise through international conferences in which there is project participation.

Foreword and Acknowledgment

This publication is the fourth in a series of syntheses produced by the Transportation Research Board's project on Transportation Technology Support for Developing Countries. A list of all project publications that have been completed to date appears on the inside back cover of this book.

It is planned that each synthesis be published first in the English language and that separate French and Spanish versions be published as soon thereafter as the respective translations can be completed.

The objective of the book is to provide useful and practical information for those in developing countries who have responsibility for the structural design of low-volume roads. Feedback from project correspondents will be solicited and used to assess the degree to which this objective has been attained and to influence the nature of later syntheses.

The Transportation Research Board acknowledges the valuable advice and direction that have been given by the project Steering Committee and is especially grateful to Wilbur J. Morin, Lyon Associates, Inc.; Adrian Pelzner, U.S. Department of Agriculture Forest Service; and Eldon J. Yoder, Purdue University, who provided special assistance during the development of this particular synthesis.

CHAPTER 1

Introduction

A major factor in the economic status of any country is the road network that is available for the movement of people and goods. A wide range of road structures may be built and used in the network. The range extends from structures that serve very low volumes of traffic to those that must accommodate high-density, high-load traffic. Roads within this range may be classified as follows:

(a) earth roads;
(b) granular surface roads;
(c) granular pavements with less than 1 in of bituminous surface;
(d) granular pavements with more than 1 in of bituminous surface, generally with unbound layers although stabilization might be used;
(e) full-depth asphalt pavements; and
(f) concrete pavements.

The ability of a road to perform adequately under high-volume, heavy-load traffic over a long service life (15-25 years) increases as the road structure progresses from type (a) to types (e) and (f). Cost and engineering efforts expended in design and construction also increase from type (a) to types (e) and (f).

For purposes of this synthesis, types (a), (b), and (c) are applicable to low-volume roads. The behavior of type (c) is largely the same as for granular-surfaced roads because thin bituminous surface is normally a surface treatment rather than a higher type premix surface. High-volume roads are then represented by types (d), (e), and (f).

Although the progression from type (a) to types (e) and (f) generally represents increased ability to accommodate traffic, there is no universally accepted traffic value that clearly indicates what type of road structure should be used in a given situation. An approximate range of 400 to 500 vehicles per day (vpd) will be used to differentiate between low-volume and high-volume roads.

It is not only the total number of vehicles per day that influences the structural design and performance of a given road. Axle load, tire pressure, and gear geometry have even greater influence on structural performance. This is particularly true for certain types of low-volume roads that are built for special purposes such as mine-haul and timber-haul.

Although traffic volume does not provide a definite criterion for choosing between structural types, traffic levels from 150 to 400 vpd generally result in the use of a type (c) structure. In some parts of the world, however, relatively thick (2-3 in) bituminous concrete surfacings may be used for these traffic levels. Moreover, the use of thin bituminous surfacings (seal coats and surface treatments) need not be restricted to low traffic volumes. For example, New Zealand has successfully used the type (c) category for traffic levels of up to 2000 vpd over a 10-year design period and with little or no maintenance (1).

The main objective of this synthesis is to present structural design concepts and methodology that are primarily applicable to roads that serve less than 500 vpd and that are composed of a granular layer, with or without a thin bituminous surface. Thus the synthesis is concerned for the most part with only type (b) and type (c) road structures.

Low-volume roads can be very important components of the total road network in any country. Table 1 shows the percentage of unpaved roads in the total road network of selected developing countries. In nearly every country more than half of the roads are unpaved. Even in more developed countries such as the United States, a high percentage of the road network is unpaved. Of 3.8 million miles of roads in the United States, 2 million miles are either unsurfaced or surfaced with granular materials. Another 0.9 million miles have thin surface treatments or seal coats. Thus approximately 76% of the total U.S. network is in the unpaved road category (2).

The development of design procedures that are applicable to all parts of the world and to all types of materials, environments, loads, and construction quality is a very formidable task. Although a wealth of information is available for the design of high type pavements, there is a lack of knowledge about the design and performance of low-volume unpaved roads.

In spite of this missing knowledge, many design concepts and engineering fundamentals are available and applicable to all types of road structures. Much is also known about the basic design factors that affect the performance of road structures. In this synthesis design and performance concepts are presented in Chapters 2 and 3; design factors are presented in Chapters 4-8. Design methods for low-volume roads are presented in Chapter 9 and illustrated in Chapter 10.

Table 1. Percentage of unpaved roads[a] in total road network of selected countries 1975 data (1).

Country	Unpaved Road (%)	Country	Unpaved Road (%)
Angola	89	Sierra Leone	83
India	65	Sri Lanka	30
Indonesia	75	Thailand	52
Kenya	92	Tunisia	48
Malawi	86	Upper Volta	91
Mozambique	91	Zambia	89
Nigeria	83		

[a]Roads that do not have an all-weather surface of bitumen or concrete.

CHAPTER 2

Fundamentals of Design and Performance

In many respects the successful design of low-volume roads is a far more difficult engineering challenge than design of high-traffic facilities. The main reason is that low-volume roads invariably become synonymous with low-cost facilities and are therefore associated with severe economic, maintenance and construction restraints.

In most cases the designer will not find it economically possible to specify materials or construction controls that are normally used for meeting high-quality standards and performance levels. The designer may not have the option of designing a structure for a specific service life as is normally done with high-volume roads. Rather design may simply amount to a determination of the probable service life of the road in question.

Regardless of the restraints imposed, it is the designer's responsibility to minimize the amount of road maintenance that will be required and to maximize the service life by proper use and application of engineering fundamentals.

The engineer should consider structural design as one of several phases in a cycle that begins with design factor considerations and that ends with reconstruction or major rehabilitation at the end of the design period.

DESIGN AND PERFORMANCE PHASES

In the first phase, four primary factors and their interactive effects must be considered. These design factors involve knowledge of (a) future traffic (vehicle types and repetitions of each vehicle), (b) the subgrade or natural foundation soil, (c) available pavement materials (granular base/subbase material as well as potential thin bituminous surface material), and (d) the specific environmental conditions at the design site. Due consideration should also be given to the feasibility and possibility of upgrading materials to improve and extend the pavement performance. These factors must be evaluated in light of major restraints that are imposed on the structural design by economic, maintenance, and construction capabilities of the road agency.

Although these factors are important by themselves, it is also important to evaluate their potential interactive effects. For example, a clay soil may or may not be a good subgrade. If it is present in a fairly dry region, and has a deep water table, and the structure has good drainage, then the in situ soil strength may be adequate. But in the presence of high in situ moisture conditions (shallow or high water table, poor drainage, high rain area), this soil would probably be one of the poorest possible subgrades.

Several structural design methods will be discussed in later chapters. The designer should employ several methods, then assess the collective results before reaching a final decision on the method to be used. It is noteworthy that no one design procedure is universally applicable for all conditions and locations.

Even for low-volume road conditions, there may be several valid design combinations of thickness, material quality, and construction practices. The overall design objective should be to make economic and engineering comparisons among several alternative designs. The comparisons should include possibilities for future maintenance, rehabilitation, or upgrading through stage construction.

Every effort should be made to develop structural designs that are consistent with local construction capabilities. In many parts of the world, the use of labor-intensive construction and maintenance techniques is the rule rather than the exception. In these regions, it may be more realistic and perhaps more economical to increase the structural pavement thickness to accommodate a poorly compacted subgrade than to specify a degree of compaction that could only be obtained by heavy mechanical compactors.

Construction control objectives will ensure that all factors assumed for use in the design phase are actually met. The designer should develop realistic quality control specifications that are compatible with construction capabilities.

After the road is constructed and opened to traffic, a verification phase should begin. The verification or feedback phase should be viewed as a systematic procedure for gathering experience. It is necessary because of uncertain accuracy of the design method used and uncertain selection of proper design input values. Although uncertainty may be reduced by conservative selection of design input values, the economic consequences of possible overdesigns should not be neglected.

Simple periodic condition surveys, conducted in a systematic manner, can either verify or point out the necessity to modify the design method used. This, in essence, is the basis of all empirical design procedures that have developed from localized experience. Without this phase, agencies may continue to use design methods that are not applicable to local conditions. The best design procedure, especially for low-volume road conditions, is one that has been verified through local experience.

The combined effects of traffic and environment will lead in time to the deterioration of any road structure to the point where even increased routine maintenance activity will no longer improve performance. At this time, major maintenance activity in the form of either restoration, reconstruction, or rehabilitation is necessary. If the original design was successful, then this failure condition will be reached at the time that was selected as the design period in the design methodology.

Although the designer must take all of the foregoing phases into account, this synthesis deals mainly with only the first two phases. The phases not covered in this synthesis are treated in detail in other publications that are listed in the inside back cover of this report.

STRUCTURAL VERSUS FUNCTIONAL FAILURE

The term <u>design</u>, in an engineering sense, usually implies analysis of conditions that lead to the failure of a structure. The analysis must take into account the losses that failure will bring and also the costs of repair or restitution. When designing structures, such as bridges, dams, and multistory buildings, the loss factor is normally very high. Historically, engineering practice has been to design these structures so that no failure will occur. This is generally accomplished through the use of relatively large factors of safety.

In contrast, the failure of a road or highway is seldom catastrophic with respect to loss of life or extensive property damage. Because of this, the design of a road structure is based on a design period that optimizes the economic investment in the road structure. It is understood that the structure will be in a failure condition at the end of the design period. A good pavement design is one in which failure occurs in accordance with the design period that was selected by the engineer. Thus a failed structure does not reflect an unsuccessful or poor design unless the failure occurs long before the design period has ended. If the road lasts longer than its intended life, then it has been overdesigned relative to the design period and is probably an uneconomical structure for the performance that was required.

Although road structures have been designed and constructed for many years, engineers still differ on the definition of failure. At present, there are two general types of failures associated with road structures.

Structural failure is defined as a collapse of the structure, or a breakdown of one or more of the structural components, that is due to vehicular traffic and that makes the structure incapable of sustaining the loads imposed on its surface (3).

For flexible pavements, load-associated permanent deformation (rutting) is the primary manifestation of structural failure. When the structure has a bituminous surfacing (regardless of the type or thickness), cracking distress must also be considered. For high-volume traffic conditions on major roads, an average rut depth of about 0.5 in (roughly 10 to 20 mm) has usually been defined as a failure condition. However, for low-volume granular roads, a greater degree of rutting is usually allowed if no bituminous surfacing is used. Most design procedures for these road types are based on failure at rut depths from 2 to 3 in (50 to 80 mm) (4, 5).

The use of a structural design procedure obviously involves comparisons of stress with the strength of the structural materials. Because of the importance of this design concept, the next section of this chapter presents fundamentals of stress theories that relate to structural design.

The use of functional failure concepts for pavement design formally emerged from the American Association of State Highway Officials (AASHO) Road Test that was conducted in the United States during 1956-1961. The basic function of a road is to carry traffic from one location to another in as smooth and safe a manner as possible. Functional failure therefore occurs when the structure becomes unduly rough and unsafe for the traffic it carries. Thus the primary manifestation of functional failure is road roughness.

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.

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

Performance is described by the serviceability history of the road structure as time and accumulated traffic increases. Figure 1 shows the hypothetical serviceability histories of two different road sections. By using these plots, performance can be defined by the length of time, or accumulated amount of traffic, for which serviceability remains above the terminal serviceability (failure) value. Thus road A has performed perhaps twice as well as road B.

A significant feature of functional failure concepts is the recognition that structures can fail either by structural or by nonstructural distress mechanisms such as environmentally induced roughness. Because design procedures can rest on either structural or functional failure conditions, it is important for the design engineer to understand the failure conditions behind any particular design procedure.

FUNDAMENTALS OF STRESS DISTRIBUTION

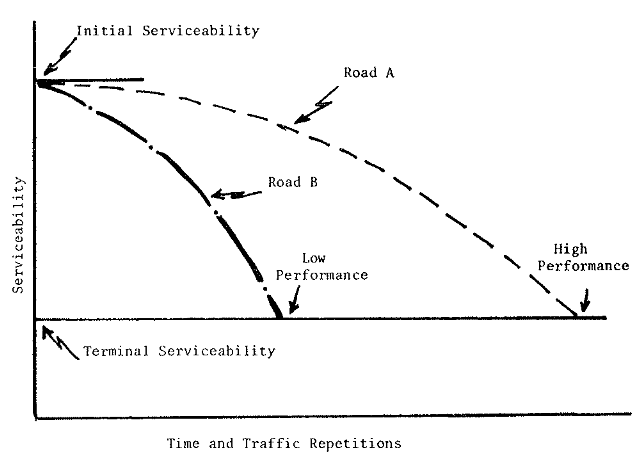
Historically, almost all flexible pavement design methods have evolved from viewing the road as a structural system. As a result pavement structures are designed so that stresses caused by vehicle loads will not exceed the strength of any structural component.

Each engineering material such as soil, timber, concrete, or steel has a specific set of physical properties and can sustain differing types of stresses to varying degrees. For example, steel can tolerate a much higher tensile stress before failure than can concrete. Most soils have little or no tensile strength. In addition, a particular material may fail when the tensile stress exceeds the tensile strength, when compressive stress exceeds the compressive strength, or when shear stress exceeds the shear strength.

For the subgrade soils and unbound granular subbase and base materials that make up flexible pavement structures, the normal failure mechanism is shear failure. That is, the structural material fails by slippage of particles over each other. This occurs because shear stresses are greater than the shear strength of the material.

The failure movement is a downward and outward displacement of the material and gives rise to deformations or ruts at the surface of the structure. Each vehicle will cause some incremental rutting or surface displacement. Thus vehicle type and load repetitions must be viewed together as major sources of rutting failure. Rutting of pavements is sometimes referred to as a repetitive shear failure. This is the basic structural design factor. Finally, because the structure may be constructed of layers of different materials, failure is possible in any layer.

Figure 1. Serviceability performance concepts.

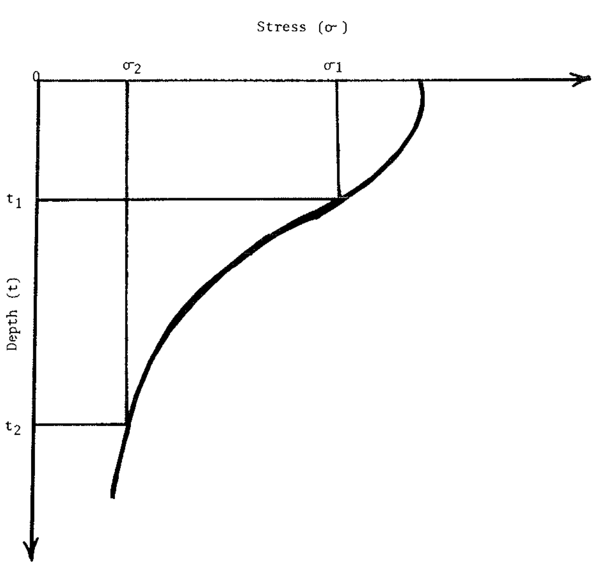


The general relationship between stress and depth within a flexible pavement structure is shown in Figure 2. Stresses for a typical vehicle are largest near the surface and quickly decrease with depth. At depth t_1 the stress caused by the vehicle is σ_1 and at a depth t_2 the stress is σ_2. If the pavement were to be constructed on a very strong subgrade soil with a strength greater than or equal to σ_1, then the total thickness of pavement required would be t_1. In other words, at a pavement thickness of t_1 or greater, the stress in the subgrade soil is less than or equal to the strength of the soil. Likewise, if a weak subgrade soil with a strength equal to σ_2 is encountered, a thickness of at least t_2 would be required to prevent shear failure in the subgrade. As a result, it can be seen that the total thickness of pavement is significantly affected by the strength of subgrade soil over which the pavement is to be constructed. In general, the factor of subgrade strength is most significant in affecting the total pavement thickness requirements for flexible pavements. This factor is discussed in detail in Chapter 5.

Since stress decreases with depth, the highest-quality materials should be placed in the upper portions of the pavement near the surface. Lower-quality materials can be used in the lower portions of the pavement near the subgrade.

From a basic structural viewpoint, a road structure should be designed so that layers of materials with increased strength or quality are placed from the subgrade toward the pavement surface. This is conceptually illustrated in Figure 3a. If the total thickness of pavement above the subgrade is quite large, it will be more economical to use a lower-quality layer (subbase) directly over the subgrade along with a higher-quality upper layer. The higher-quality layer becomes the base course for pavements with an asphalt surface; it is the surface layer for granular surfaces. Use of the high-quality material for the total pavement thickness would be uneconomical as shown in Figure 3b. Figures 3c and 3d illustrate subgrade failure and base failure conditions, respectively. The role of base/subbase layer quality and properties in design is discussed in detail in Chapter 7.

Figure 2. Typical distribution of stress with depth for flexible pavement structures.



Distribution of stress is a direct result of the particular characteristics of the vehicle wheel load. In stress theories, both the total load (P) and contact pressure (p) directly influence the resulting stress pattern.

Figure 4 shows the effect of both of these parameters on the resulting stress. Figure 4a shows stresses from two different wheel loads at the same tire pressure. Stresses in Figure 4b are for tires at different pressures under equal loads. The figures show the following:

1. The effect of load changes extends into the lower layers of the structure. Thus an increase in wheel load magnitude normally will necessitate a thicker structure and perhaps a higher-quality material in the subbase layer.
2. The effect of tire pressure changes is greatest in the upper pavement layers. Thus, increasing vehicle tire pressure will necessitate higher-quality layer material near the pavement surface.

Natural soils and unbound granular material used as subbase or base materials can be strengthened by one of two major ways, by physical stabilization through compaction or by chemical stabilization with lime, cement, or bitumen additives. As a general rule, chemical stabilization is much more expensive than physical stabilization and may not be economically feasible.

In general, the addition of various additives will increase the rigidity and strength of the material when these mixtures are properly designed. The increased rigidity tends to reduce the distribution of stresses with depth throughout the pavement as shown in Figure 5. At a given depth

Figure 3. Flexible pavement stress-strength considerations.

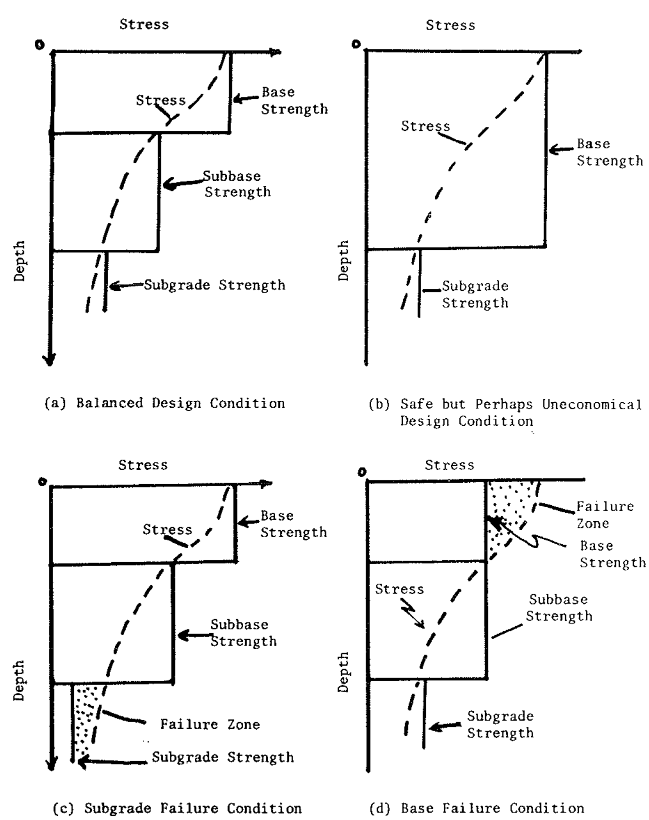


Figure 4. Effect of tire load and pressure on stress distribution.

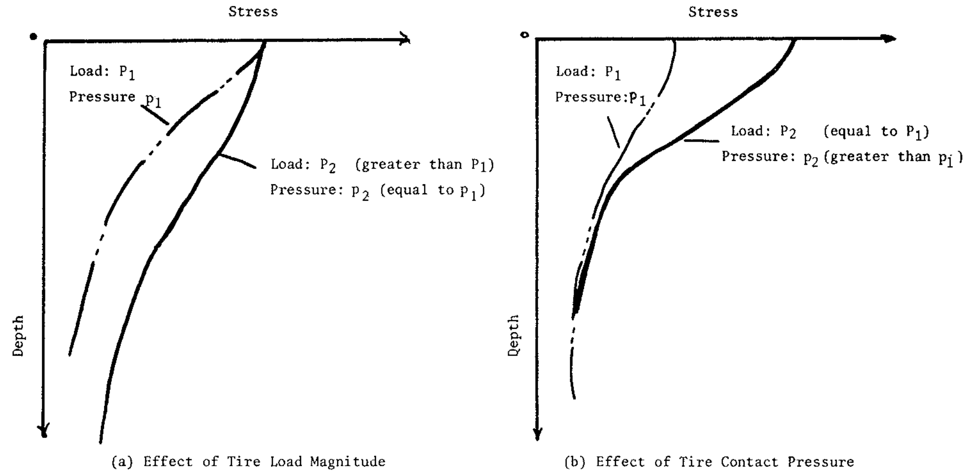
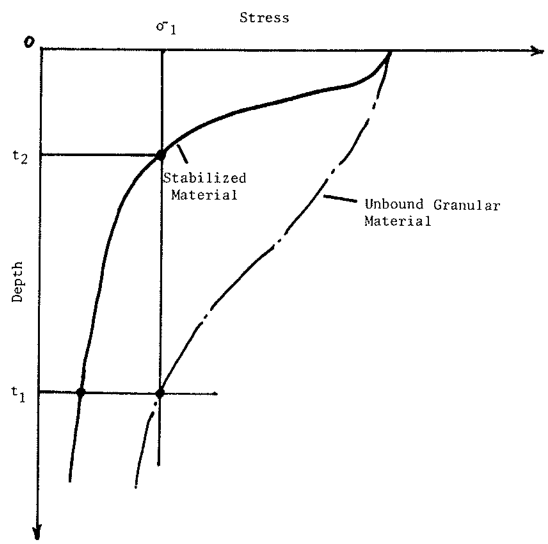


Figure 5. Effect of stabilization on stress distribution.



(t_1) there is less stress in the stabilized material than in the corresponding unbound granular material. Thus better performance and longer life can be expected from stabilized materials.

An alternative way of viewing Figure 5 is to consider a constant stress condition, such as σ_1. The figure shows that less thickness (t_2) of the stabilized material is needed than for the unstabilized material (t_1) when both are at strength level σ_1. This fact obviously justifies an analysis to determine if it is economic to stabilize unbound granular materials. Chapter 8 discusses further the effects of improving material quality.

SERVICEABILITY AND PERFORMANCE CONCEPTS

General concepts of pavement serviceability and performance were introduced earlier as a basis for defining functional failure. In addition, serviceability measurements can be used as a quality control tool in the construction process. They can also be the basis for periodic evaluations of road performance during the design life of the road. Serviceability can provide a basis for setting maintenance priorities and for projecting future maintenance needs.

Two broad methods have been used to evaluate serviceability, one is through Present Serviceability Ratings (PSR) that are made subjectively by a team of raters. After observing and riding over a given road section, each rater gives a numerical serviceability value to the section, generally on a scale of 0 to 5. A rating of 1 or below implies that the road section is failing completely to carry out its function; a rating of 4 to 5 implies that the section is quite smooth and safe for use. The PSR for a given section is defined to be the average of all values that have been given by the rating team.

A second and more commonly used method for evaluating serviceability is through a Present Serviceability Index (PSI) whose values are calculated from measurements of road roughness and other distress factors such as cracking and rutting (6). Formulas have been developed for converting the measurements into PSI values. In all such formulas, more than 90% of the PSI value comes from the roughness measurement. Thus for all practical purposes, PSI is inversely related to roughness. That is, low roughness is equivalent to high serviceability and high roughness is equivalent to low serviceability. In summary, present serviceability can be evaluated through subjective ratings to obtain PSR values or can be calculated from roughness measurements to obtain PSI values.

Road roughness as experienced by the road user is a function of the road surface profile, speed, and vehicle characteristics. The most significant factors are the road profile and the speed. The road profile, in turn, is affected by the longitudinal variation, transverse variations, and horizontal alignments of the road. Of these three, the most significant is the longitudinal variation and its resulting influence on roughness.

Numerous devices and procedures are available for measuring road roughness. Included are the following:

1. Rod and level surveys,
2. Profilograph (rolling straightedge),
3. Profilometers (slope and CHLOE),
4. BPR roughometer and TRRL bump integrator,
5. Surface Dynamics (SD) and General Motors profilometer, and
6. Car road meters (PCA and Mays meter).

Specifics for these devices and methods are beyond the scope of this synthesis; however, discussions of them will be found in other works (3, 7, 8). Since definitive correlations between roughness devices are not available, it is not possible to compare roughness values from a particular device in one region with values from another device in another region. Each road agency should attempt to build its own experience base with whatever device or technique is most appropriate for that agency.

Recent studies on major projects in Bolivia, Kenya, and Brazil have indicated typical roughness values for low-volume road conditions. Table 2 summarizes road roughness readings taken with the Mays meter roughness device (a portable car road meter) on surface-treated and gravel roads in Bolivia (9). The output of the Mays meter is in millimeters of roughness per kilometers (or inches per mile). In essence, this value represents the summation of roughness (deviations from a true plane) per unit of road length.

As can be seen, for surface-treated roads, roughness values (R) vary from about 800 to 3000 mm/km; values for gravel-surfaced roads vary from about 4400 to nearly 16,000 mm/km. In the Bolivian study, the highest R value recorded within a granular-surfaced section was about 22,000 mm/km. In the Kenya study (10, 11, 12, 13), roughness measurements were conducted with the TRRL (5th wheel) bump integrator device that is similar to the BPR roughometer. For gravel-surfaced roads, the range in roughness was found to be 2200 to 20,600 mm/km. It was concluded that a good gravel road had a roughness near 5000 mm/km, while a poor road had roughness values in excess of 10,000 mm/km.

Significant changes in measured roughness occur between wet and dry seasons for gravel-surfaced roads. Results from the Bolivian study are shown in Table 3 and clearly indicate that road roughness is greater in dry seasons than wet seasons. Thus, the effect of seasonal climatic (rainfall) conditions should be taken into account when roughness measurements are made.

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 two to three weeks. Table 4 illustrates the effects of grading on the Mays meter roughness value (9).

The combined effects of both grading and seasonal conditions on roughness or serviceability can produce relatively large deviations about the average trend line of the serviceability-traffic relationship. This effect is shown schematically in Figure 6.

Table 2. Typical road roughness values for Bolivian roads (9).

R (mm/km)[a]		R (mm/km)[a]	
Surface-Treated Roads	Gravel-Surfaced Roads	Surface-Treated Roads	Gravel-Surfaced Roads
927	8 776	2997	8 179
1029	12 751	1245	8 001
813	9 855	1067	10 820
1803	12 649		4 394
2718	14 986		15 596

[a]Mays meter roughness values.

Table 3. Seasonal climatic influence on roughness (9).

Dry Season R[a] (mm/km)	Wet Season R[a] (mm/km)	Roughness Change (%)
8 776	5837	-33
7 500	3795	-49
12 751	4621	-64
7 140	3385	-53
9 855	7184	-27
14 986	5866	-61
8 179	7164	-12
7 990	5811	-27
5 610	5662	+1
10 820	6201	-43
4 394	3774	-14

[a]Mays meter roughness values.

Table 4. Effect of grading on roughness (9).

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

DISTRESS MECHANISMS

Distress factors that lead to structural or functional failure conditions are shown in Table 5 for granular surface and surface treatment roads.

There are more individual distress manifestations for granular surface roads than for surface-treated roads. Because of its relatively thin layer effect (0.5-1.0 in) and in-place construction, a single surface treatment layer does not significantly increase the structural rigidity of the road. Thus, the value of this layer is not to increase strength but to accomplish two important functions. They are

1. To provide a waterproof seal on the surface to minimize the effects of surface water infiltrating into the granular base/subbase layers and subgrade soil. The strength of most pavement materials, especially fine-grained soils, is significantly affected by moisture. Thus, while the surface-treated layer itself does not increase the structural strength, it may effectively increase the in-place strength of all pavement layers and thus increase the service life of the road.
2. To protect the unbound granular base/subbase materials from the various distress mechanisms associated with the disruptive effects of traffic and environment.

Figure 6. Variations in low-volume road serviceability.

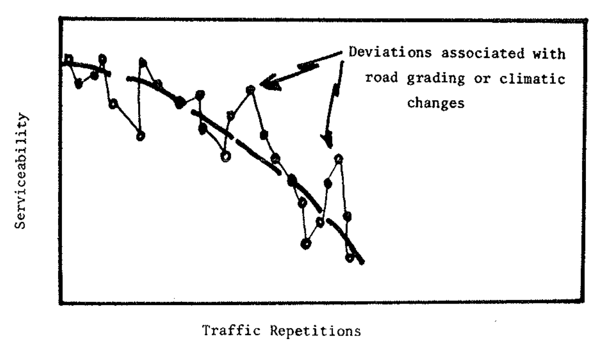
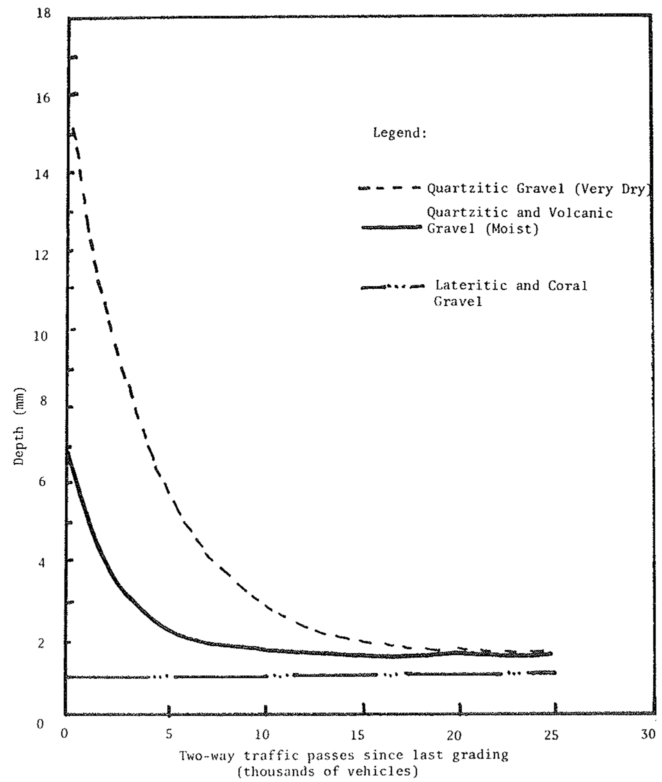


Table 5. Major distress types of low-volume roads.

Distress Factor	Effect On . . .	Applicability for Granular Surface	Applicability for Surface Treatment
1. Dusting	Safety, environment	Yes	-
2. Surface looseness	Safety, roughness	Yes	-
3. Gravel loss	Structural deformation, roughness	Yes	-
4. Surface deformations	Structural deformation, roughness	Yes	Yes
a. shear displacement			
b. layer material densification			
c. layer material intrusion			
5. Surface heaving	Roughness[a]	Yes	Yes
a. frost heave			
b. expansive clays			
6. Corrugations (washboarding)	Roughness	Yes	Possibly
7. Surface erosion (gulleying)	Roughness	Yes	-
8. Potholes	Roughness	Yes	Only if cracked area not maintained
9. Surface cracking	Structural deformation, roughness	-	Yes

[a]Greatly increased if surface profile changes are highly variable in the longitudinal direction.

Figure 7. Depth of loose material for granular surface roads in Kenya.



One of the major advantages of surface-treated roads is to eliminate the need for periodic and extensive maintenance grading operations. However, a surface-treated layer is a costly addition to the pavement structure and is not maintenance free. When surface-treated layers are present, routine maintenance activities must focus on keeping the surface free of cracks that would self-defeat the function or purpose of the layer.

For granular-surfaced roads, permanent deformations of 2 to 3 in are considered to be a failure level from a structural viewpoint. In surface-treated roads this magnitude of rut cannot be tolerated without extensive cracking damage to the bituminous layer. Thus, higher levels of base material quality, stricter construction control, and higher design standards are generally needed for surface-treated roads.

For granular-surfaced roads Table 5 indicates that many distress factors may act on the granular 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. Granular material for the base/subbase layers should be selected, and a sound maintenance plan should be used to minimize the other possible distress modes. For bituminous surface-treated pavements more reliance must be placed on the structural design to minimize or control the rate of deformation. Sealing and patching maintenance activities are necessary to control roughness and ride quality.

Dusting of granular surface roads is a loss of fine material that is brought about by the abrasive action of traffic on the road. Significant environmental problems may be created by the settling of fines in adjacent land areas that are used for agriculture and live stock. In addition, dust clouds caused by traffic create a danger to trailing vehicles. The problem of dusting is severely aggravated by dry weather, soft and highly abrasive aggregate, poor gradation of the granular material, and high traffic intensities.

The abrasive action of traffic on granular surface roads will eventually loosen the larger aggregate particles from the soil binder. This leads to dusting and loose aggregate particles on the surface of the road. Studies in Kenya (14) have shown that both the depth of loose material and moisture content affect safety and fuel consumption. It was found that a speed reduction of about 1.5 to 3.0 km/h was obtained for a 10% increase in moisture content.

Predictive equations for the depth of loose material on granular surface roads have been developed from research studies conducted in Kenya (10, 13). The aggregate types studied were lateritic, coral, quartzitic, and volcanic gravels. Graphs that represent these equations are shown in Figure 7.

The figure shows that depth of loose material is greatest immediately after grading, then decreases quickly to a relatively constant value. For the lateritic and coral gravel-surfaced roads in Kenya, almost 95% of the measurements were below 1 mm after 200 vehicles even though the initial readings after grading were as high as 9 mm. Studies in Brazil (15) have also shown that the thickness of loose material within 2 m from the road edge is consider-

Figure 8. Gravel loss relationships for Kenyan conditions.

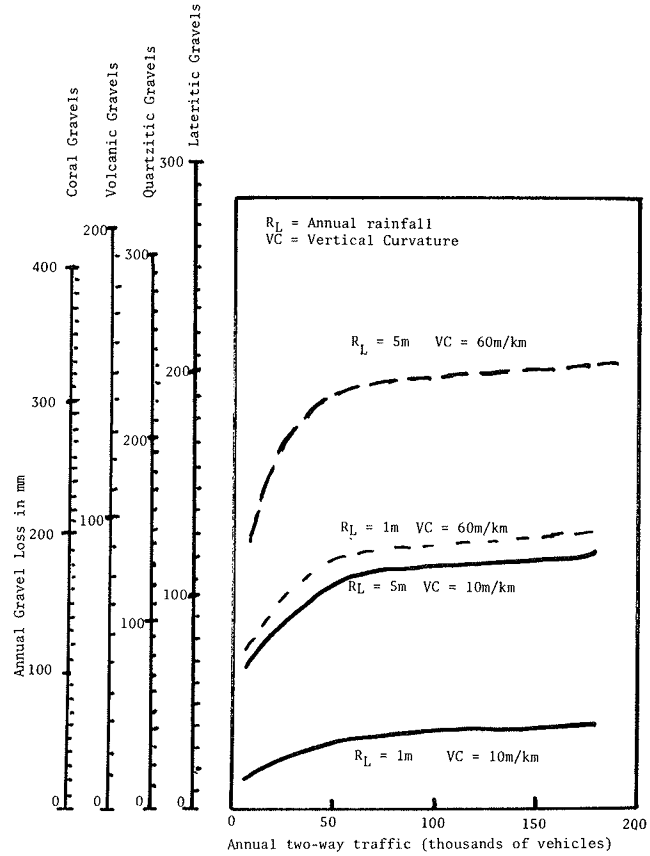
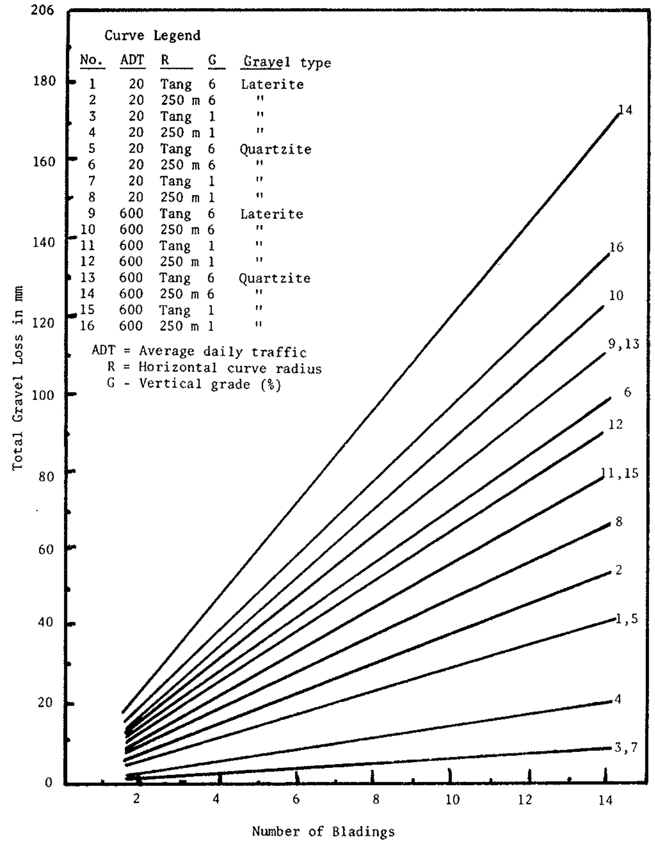


Figure 9. Gravel loss relationships for Brazilian conditions.



ably greater than at other transverse locations along the road.

The loss of gravel is a significant distress mechanism for granular-surfaced roads. The need for regraveling roads may be viewed as equivalent to the need for periodic resurfacing of high-type road structures. Gravel loss is significant because it leads to premature or accelerated structural pavement failure. In visualizing the significance of this distress, it is well to recall the fundamentals of stress distribution discussed earlier in this chapter. The loss of gravel reduces the effective pavement thickness in time, so that stress (and hence rutting failure) increases in all structural layers. Reduction in thickness will thus lead to a shorter life than was assumed in the design analysis.

Two major research studies on granular-surfaced roads have resulted in predictive equations for gravel loss. In the Kenya study (10, 13) annual gravel loss for a particular type of material was found to depend on the annual traffic volume, annual rainfall (R_L), and vertical curvature (VC). Selected plots of the predictive equations are shown in Figure 8. The figure shows, for example, that an annual loss of about 95 mm of volcanic gravels can be expected when the traffic volume is 400 vehicles/day or 146 thousand vehicles/year, rainfall is 1 m/year, and the vertical curvature is 6% or 60 m/km.

Figure 8 indicates that rainfall, material type, and vertical curvature are the most significant factors affecting gravel loss, especially for average daily traffic (ADT) values greater than about 150 vehicles/day or 50,000 vehicles/year.

The Brazilian study (15) 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 for various combinations of traffic volume, horizontal curvature, vertical grade, and type of material are shown in Figure 9. It can be seen that the gravel loss is somewhat similar to that found in the Kenya study for 6 to 12 gradings within a year. The average Kenyan rainfall condition was 1100 mm/year.

Both studies show that material type, traffic, and vertical curvature are common factors affecting gravel loss. However, the Brazilian equation excludes the rainfall term that was found to be significant in the Kenya study. The Brazilian study includes both the number of bladings (standard of maintenance) and a horizontal curvature factor.

Surface deformations can occur from a variety of causes, but the main causes are repetitive shear movements within the structure. Surface deformations may also be due to densification that results from repeated traffic loadings and the intrusion of aggregate particles into the subgrade soil.

The inherent variability in materials and construction along the road leads to differential deformations that significantly alter the road surface profile. Thus uniformity of both materials and construction has a direct bearing on the roughness of any given road.

In contrast to deformations of the road profile, heaving of the surface may also result in increased roughness. In general, heaving is not associated with vehicle loads but is usually due to either frost or highly expansive clays. These two problems must be considered in the design stage.

The development of road corrugations is a significant roughness factor on granular-surfaced roads. These high-amplitude, short-wavelength profile changes can induce vehicle resonance at speeds that are normally associated with low-volume road conditions. The corrugation is often accentuated as the vertical road grade increases.

Erosion of a granular surface generally occurs when the granular material lacks a plastic binder and is subjected to periods of high-intensity rainfall. Proper attention to pavement cross-ection slopes and drainage is necessary to control this type of distress.

The development of potholes is due to localized erosion or raveling of the granular surface. With continued traffic, the loose material is expelled and the area and depth of the hole increase. Without proper maintenance, this type of distress can cause significant damage to vehicles.

The development of cracks in the surface layer is a failure condition because the layer can no longer serve its intended function. Surface water will infiltrate through these cracks and significantly accelerate the deformation distress adjacent to the area. If not sealed, the cracks will enlarge, become interconnected, and eventually form potholes. If this occurs, patching of large areas, rather than simple crack sealing, will be necessary.

Predictive models for rutting in the vehicle wheel paths have been developed from the Kenya and Brazil studies on granular-surfaced roads (10, 13, 15). In general, the outer wheel path nearest the shoulder or pavement edge is more deeply rutted than the inner wheel path. Studies in Brazil, for example, have shown that the mean rut depths in the outer path are about 9% greater than that found in the inner path. Rut depth values usually refer to the outer wheel path conditions. In the Kenya study, rut depth equations were developed for two types of granular surface material as shown in Figure 10.

In the Brazilian study, rut depth measurements were made with a 4-ft straightedge on lateritic and quartzitic gravel-surfaced roads. Predictive equations that were developed show that rut depth depends on type of gravel, climatic season (wet or dry), vertical and horizontal curvature, traffic per day, and accumulated traffic since the road has been graded. Tables 6a and 6b show predicted rut depths for selected values of the Brazilian conditions.

The tables show that the dry season rutting occurs at a relatively slow rate. However, during the wet season, a substantial increase in rutting can be seen. This is probably due to the significant influence of moisture on material strength. From an engineering viewpoint, two significant factors in rutting are not incorporated into either of the predictive models presented. These are the type of subgrade soil and the type of vehicle loads that occur within the traffic mix. More definitive predictions of rut depth would have to take these factors into account.

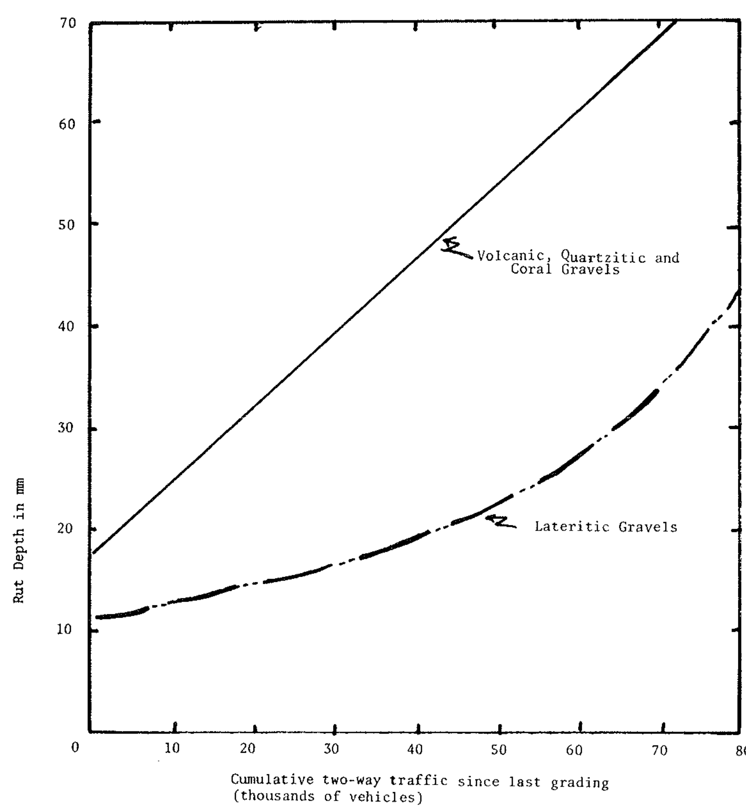
PREDICTION OF ROAD ROUGHNESS

Both the Kenya and Brazilian studies developed prediction equations for road roughness. In the Kenya work, the TRRL bump integrator device was used. The Brazilian equations were developed for the General Motors profilometer.

Separate equations were found in the Kenya study for different types of surface material, depending on the gravel surface type. Figure 11 shows how the two equations predict roughness as a function of traffic that has accumulated since the road was graded.

The Brazilian study was based on data from the General Motors profilometer whose output is counts per kilometer, called the quarter-car index (QI). A general interpretation of the QI values can be made by noting that a very smooth, newly constructed asphaltic concrete pavement would have a QI value

Figure 10. Rut-depth predictions for gravel-surfaced roads in Kenya.



less than 30 counts/km. Two separate regression equations were developed. The first predicts roughness as a function of time within a blading period, given the roughness immediately after blading. The second equation predicts the roughness after blading. The general nature of the equations is shown in Figure 12 for a selected set of conditions.

There are significant differences between the Kenya and Brazilian studies relative to the factors that affect roughness. The Brazilian relationships predict greater roughness increases with time during the dry season than during the wet season. In contrast, the Kenya study found no influence on rainfall (although the rainfall was less than 400/mm). In any event, the major factors influencing roughness appear to be gravel material type, traffic, road alignment, environment and the level of maintenance activity (grading) accomplished.

Table 6. Predicted rut depths (mm) in outer wheel paths for Brazilian conditions (15).

				Lateritic Wearing Course						Quartzitic Wearing Course					
				20 ADT			600 ADT			20 ADT			600 ADT		
				Cumulative Traffic			Cumulative Traffic			Cumulative Traffic			Cumulative Traffic		
Type Season	Vertical Curvature (%)	Lane	Horizontal Curvature (%)	0	2000	4000	0	30 000	60 000	0	2000	4000	0	30 000	60 000
Dry	6	Downhill	Tangent	4	7	12	10	18	31	9	15	24	22	26	32
			250	7	9	11	10	15	23	9	11	14	13	14	16
		Uphill	Tangent	4	7	12	10	18	31	9	19	41	22	30	43
			250	7	9	11	10	15	23	9	15	24	13	17	20
	1	Downhill	Tangent	4	7	12	10	18	31	9	15	24	22	26	32
			250	7	9	11	10	15	23	9	11	14	13	14	16
		Uphill	Tangent	4	7	12	10	18	31	9	19	41	22	30	43
			250	7	9	11	10	15	23	9	15	24	13	17	20
Wet	6	Downhill	Tangent	7	14		28	51		8	55		32	79	
			250	11	2		26	16		8	6		20	16	
		Uphill	Tangent	7	14		28	51		8	72		32	90	
			250	11	2		26	16		8			20	19	
	1	Downhill	Tangent	11	2		43	27		13	10		49	41	
			250	18	0		41	8		13	1		31	8	
		Uphill	Tangent	11	2		43	27		13	13		49	47	
			250	18	0		41	8		13	1		31	10	

Figure 11. Road roughness relationship with traffic for Kenyan conditions.

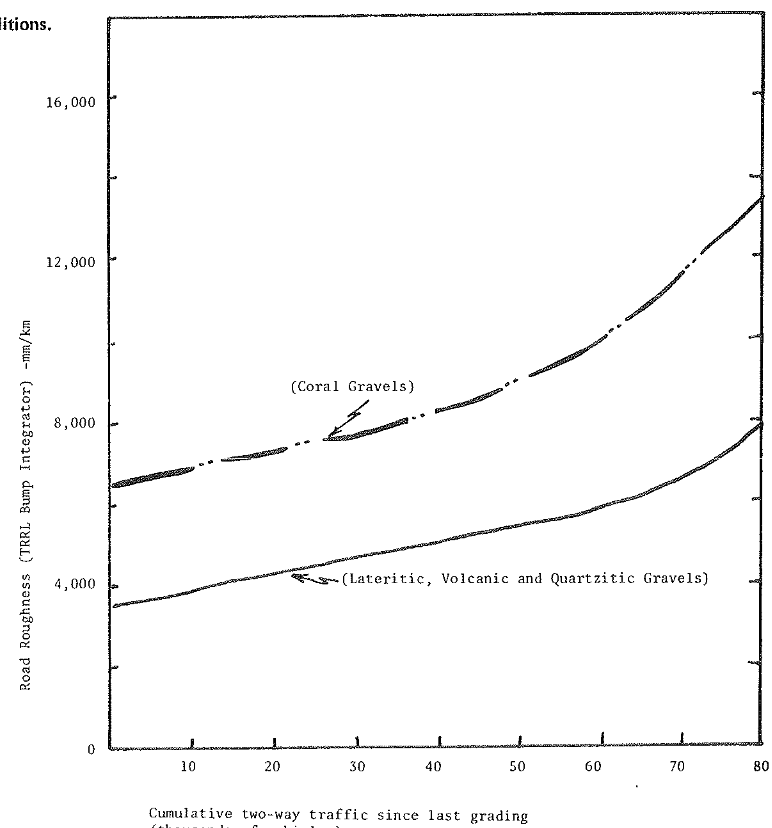
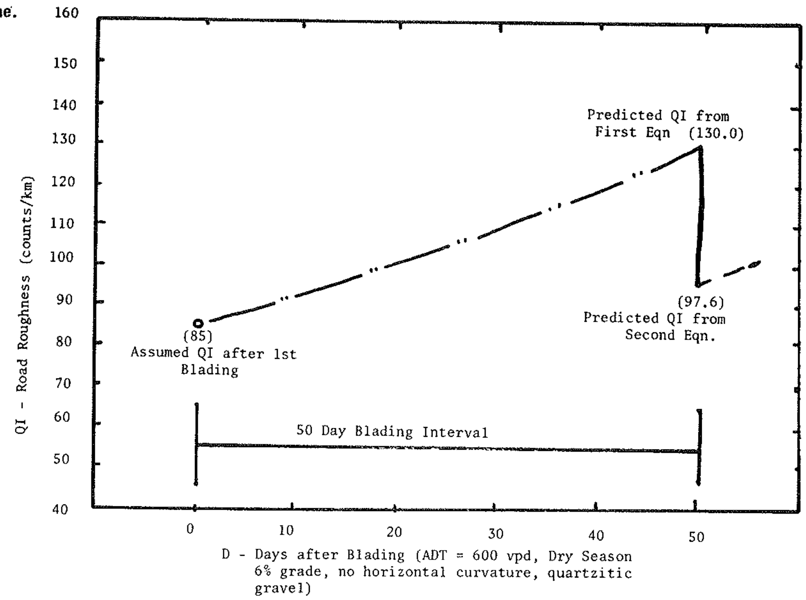


Figure 12. Predicted change in roughness with time.



CHAPTER 3

Design Philosophy and Economic Considerations

The influence of local conditions is of paramount importance in the design of low-volume roads. The design engineer must carefully evaluate locally available materials, construction equipment capabilities, availability of local labor, technical skills of individuals connected to the project, and the level of maintenance that will ultimately be part of the design, construction, and road-use cycle.

Limitations imposed by local conditions may make it impossible to construct roads whose design specifications require a given level of service over a predefined design life. In such cases, the engineering effort should be directed to determining what design life can be provided for the structure under the best of local conditions.

It should be recognized that different designers of low-volume roads will often differ in their design philosophies. For example, some believe that structural thickness is a relatively unimportant design consideration and that all engineering effort should be placed on the evaluation of pavement material properties. In contrast there are design methods that concentrate almost entirely on structural thickness and minimize the importance of materials quality. Roads can be built without any level of engineering activity, but, if resources are to be used efficiently, road design and construction must be engineered. Accordingly, the overall philosophy of this synthesis is to present a variety of engineering methodologies and fundamentals that can lead to optimum use of resources.

As a general rule, the level of engineering effort to obtain input values for design methods should be greater when increased traffic volumes and loads are expected, especially when poor subgrade conditions are likely. Thus, it is generally cost-effective to increase the engineering level as stronger structures are required.

Engineering effort should also be consistent with respect to all design variables. For example, it is not sound practice to develop expensive traffic information if soil tests will not be performed to evaluate subgrade strength.

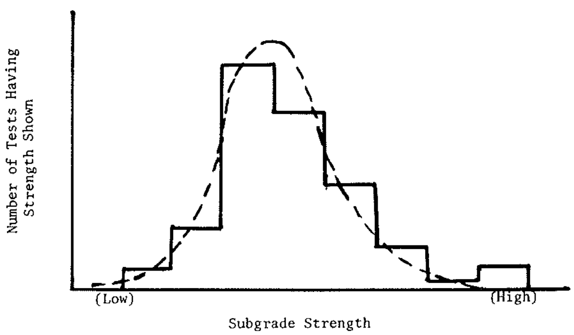
It can be expected that an increased level of engineering activity will increase design reliability and hence make the most efficient use of available material and financial resources. Design reliability is measured by the difference between the predicted performance and the actual performance of the road under traffic. Large differences indicate that the design engineer has not analyzed fully the local conditions and available resources.

Another design consideration is that all materials and construction processes have an inherent degree of variation. For example, Figure 13 shows the distribution of subgrade strength values that were obtained from laboratory tests on soil samples from an area of relatively uniform soil.

If the structural design were to be based on the lowest strength value, then virtually 100% of the road would be overdesigned and more costly than necessary. On the other hand, a design based on the highest strength value would imply that 100% of the pavement would be underdesigned and would experience premature failure. For low-volume roads it is recommended that average test values be used for design parameters such as subgrade strength.

Another important consideration is that it may be practical and cost-effective to develop a road structure in stages. The initial design can be for a less expensive structure (shorter design life) if it is planned to strengthen (or replace) the struc-

Figure 13. Influence of soil strength variability on percentage of underdesign-overdesign.



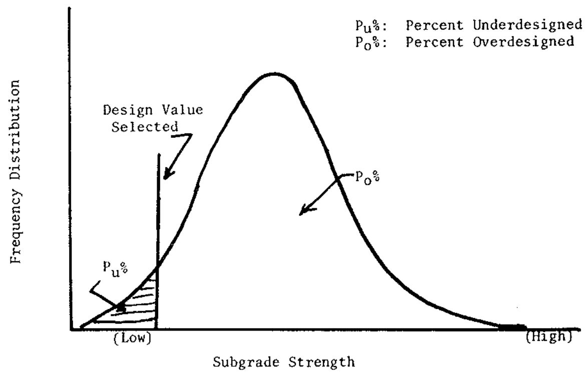


Figure 14. Principles of wconomic analysis applied to selection of design pavement section.

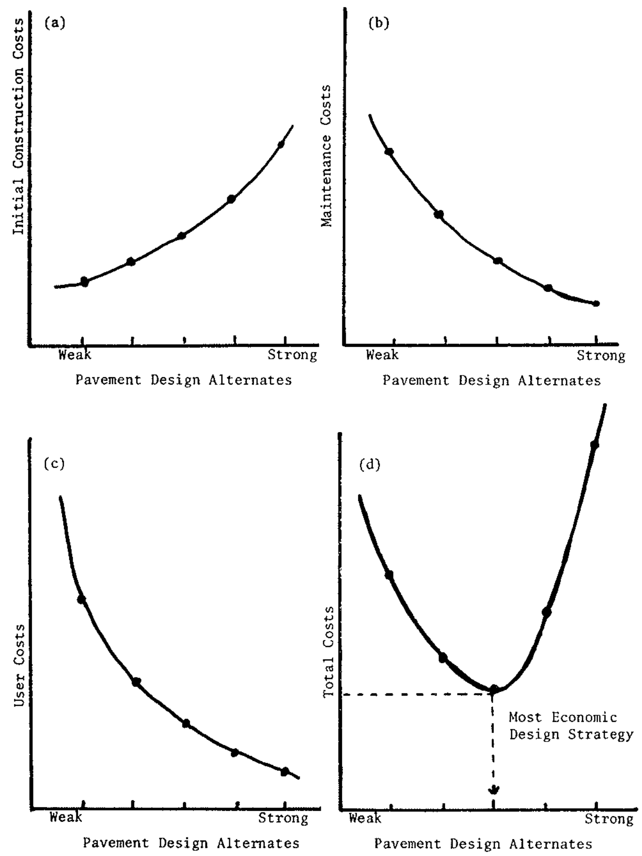


Table 7. Costs to be considered in economic analyses of alternative structural designs.

Major Group	Subgroup	Remarks
Initial cost	Pavement	All layers above subgrade
	Earthwork	Embankment/subgrade
	Site clearance	
	Drainage	
	Signposts, road markings, fencing, etc.	
Maintenance cost	Grading	Unpaved roads only
	Regraveling	Unpaved roads only
	Crack sealing	Paved roads only
	Patching	Paved roads only
	Overlays/surface dressing	Paved roads only
User costs	Travel/delay time	Added user costs
	Fuel and oil	Part of vehicle operating costs
	Vehicle parts	Part of vehicle operating costs
	Tires	Part of vehicle operating costs
	Vehicle depreciation	Part of vehicle operating costs
	Interest	
Salvage	In situ materials	May be viewed as negative cost or asset

ture no later than the time of failure for the initial structure. Additional structural strength in the second stage is generally provided through stabilization of the structural materials and/or through increased thickness of the road structure. Thus, for example, the initial design thickness might be somewhat less than required for the desired design life if additional thickness will be added in a second stage before the initial structure has failed. If warranted by traffic conditions, the second (or later) stage might involve stabilization and surface treatment for the initial surface layer. Principles of stage construction have been presented in Synthesis 2 (see the inside back cover of this publication).

One of the most significant steps in structural design is the appraisal of all costs associated with the construction and use of the structure. Methodology for economic analysis is the subject of Synthesis 5 (see the inside back cover of this publication).

Economic analysis can provide answers to questions such as the following: How can the benefits of layer stabilization be justified on a cost basis? Is it better to construct an all-weather road in frost-affected areas or to limit loads, or even to close the road system during the weak support period? What is the most cost-effective design among a variety of conventional pavement structures that have differing combinations of thickness and material quality?

All costs over the design period should be taken into account in what is called a "life-cycle cost analysis".

Four types of costs enter into the economic analysis that should be made to decide which of several alternative structural designs will be used. These costs are shown in Table 7.

As shown in Figure 14a, initial construction costs increase with designs that specify greater structural strength, but maintenance costs are expected to be less for the stronger structures (Figure 14b). User costs are also expected to be less for the stronger designs, mainly because such roads are smoother and safer than those with weak designs (Figure 14c). Finally, the total costs for some designs can be expected to be less than for other designs (Figure 14d). Thus the economic analysis can determine which designs have the least total cost. Costs that are common to all alternative designs need not be considered. For example, if only the alternatives for base stabilization are to be analyzed, then costs associated with subsurface drainage, earthwork, and subgrade preparation need not be considered since they would be common to all base alternatives.

It has been found that for nonbituminous-surfaced roads, the level of maintenance is at least as important as the initial construction standard in determining the level of service provided by the pavement system. Thus, user costs and maintenance costs are very important elements in the economic analysis.

Maintenance costs are frequently categorized into two classes: routine maintenance and major maintenance. For paved roads, crack sealing and patching frequently are of a routine preventative maintenance type. Major maintenance affects the entire pavement surface and is either a surface dressing or a bituminous overlay. Routine maintenance of an unpaved road is generally represented by grading operations. Major maintenance normally results from the need to regravel the existing road because of gravel losses, or may include the addition of gravel layers in a stage construction plan.

User costs may be categorized as those associated with travel time (to include delay) and vehicle operating costs. Vehicle operating costs are discussed in detail in Synthesis 5.

CHAPTER 4

Traffic Factors

Traffic factors for structural design are those characteristics of vehicles that lead in one way or another to structural distress and damage. Two general factors are vehicle loads and vehicle repetitions. However, the amount of distress caused by accumulated repetitions can be much different for one type of vehicle than for another type. Thus, another traffic factor is the traffic mix, that is, the relative distribution of different vehicle types and loads.

For a given vehicle type, the major factors that relate to pavement damage are the axle and wheel spacings, the magnitude of wheel load, and the tire contact pressure. General effects of tire load and pressure were discussed in Chapter 2. Chapter 4 deals with the combined effects of different vehicle types and their respective traffic repetitions. Methods of analyzing the traffic mix are also presented.

Traffic factors play an important role in structural design procedures, particularly when both loads and repetitions are relatively high. For example, much less structural thickness may be required for a design level of 100,000 repetitions than for 10,000,000 repetitions. However, the range in design thickness from 50,000 to 200,000 repetitions may be minor. Thus for low-volume roads with traffic levels less than 500 vpd, detailed and highly accurate traffic data and complex analyses of traffic effects are seldom justified. Nevertheless, an attempt should be made to obtain realistic traffic data for the specified road that is being designed, particularly if some of the vehicles are quite heavily loaded.

Two commonly accepted ways of analyzing the effect of traffic on pavement structure are (a) mixed-traffic analysis and (b) equivalent wheel load analysis. Mixed traffic analysis is the most universally accepted way of characterizing traffic. Each country or road agency may have its own techniques for obtaining and analyzing traffic data, but the same general approach is followed throughout many parts of the world.

Equivalent wheel load analysis is especially useful when designing road structures for a relatively small number of repetitions by relatively heavy vehicles. Examples include roads used for industry access, dock facilities, and hauling of timber or agricultural products. The equivalent wheel load approach generally ignores the effect of all other vehicles in the traffic mix and bases the traffic analysis entirely on the special or critical vehicle. For these reasons, this approach should be used for more specialized traffic conditions rather than as an alternate for the mixed traffic analysis method.

MIXED-TRAFFIC ANALYSIS

The basic concept of mixed-traffic analysis is that each vehicle repetition causes some structural damage (however small) and therefore consumes some part of the design life. If a particular structure is designed to convey 100 repetitions of a specific vehicle type before failure occurs, then each pass or repetition of that vehicle on the road would cause 1/100 of the life to be consumed. The damage per pass (unit damage) is thus 0.01. The same structure might have a life of 500 repetitions for a second type of vehicle. Each repetition of the second vehicle would consume 1/500 or 0.002 of the structural life. The total damage from 50 passes of the first vehicle and 200 passes of the second vehicle would thus be 50 (0.01) + 200 (0.002) = 0.90, or 90% of the design life.

In analyzing traffic mixes, it is necessary to relate the relative effect of any vehicle to that of an arbitrarily selected standard vehicle. The relative effect is called the equivalent damage factor (F) value. This value is the ratio of unit damage of a given vehicle to the unit damage that would be caused by the standard vehicle. Thus, if a given axle load has an F value of 2, each pass of this axle load is 2 times as damaging to the road as each pass of the standard axle load.

Another descriptive factor in mixed traffic relates to the lateral wandering of vehicles on the pavement surface. If the traffic is highly channelized on a narrow road, then each pass of a vehicle will result in one damage repetition at the point of maximum damage. However, if traffic movement is such that the vehicles take various lateral positions, then it may require several passes of a given vehicle to cause one unit of damage at a given point on the surface. If this factor is ignored, the resulting design will be on the conservative side and thus provide service for a somewhat longer time than the design period.

The final step in mixed traffic analysis is to determine the number of repetitions of the standard vehicle (in the design period selected) that would cause the same cumulative damage to the pavement as the actual vehicles in the traffic mix. If lateral placement variations are ignored, the number of equivalent repetitions is given by

$$N = (p_1F_1 + p_2F_2 + \ldots + p_nF_n)$$

where

N = total equivalent repetitions of the standard vehicle in the design period,

$p_1, \ldots, p_n$ = number of passes during the design period of vehicle type 1, ..., vehicle type n, and

$F_1, \ldots, F_n$ = equivalent damage factor for vehicle type 1, ..., vehicle type n.

In the remainder of this section a number of techniques are presented for calculating F-values and equivalent repetitions.

The standard vehicle that is almost universally used in mixed traffic studies is the 18,000-lb (8200 kg) single axle load. Thus, the relative damaging effects of all other vehicles in the traffic mix are related to this axle load.

Equivalent damage factors (F-values) for different axle loads and axle types can be computed directly from the fundamental definition of the F value. Factors that are widely used throughout the world are those that were developed from analysis of data from the AASHO Road Test that was conducted in the United States during the late 1950s and early 1960s.

Table 8 shows F-values that are used throughout the remainder of this synthesis. As can be seen, the values are shown for both single-axle and tandem-axle truck configurations. The F-value for a 30,000-lb single-axle load is shown to be F = 10.03. Thus, this axle load is about 10 times as damaging as the 18,000-lb single-axle load. Thus it requires about 10 repetitions of an 18,000-lb single-axle load to cause as much damage as one repetition of the 30,000 lb single-axle load.

Table 8. Summary of equivalent damage factors.

Single-Axle Load				Tandem-Axle Load			
lb (000s)	kg	kN	F-Value	lb (000s)	kg	kN	F-Value
2	910	8.9	0.0002	10	4 540	44.5	0.01
4	1 810	17.8	0.003	12	5 440	53.4	0.02
6	2 720	26.7	0.01	14	6 350	62.3	0.03
8	3 630	35.6	0.04	16	7 260	71.2	0.05
10	4 540	44.5	0.08	18	8 160	80.0	0.08
12	5 440	53.4	0.18	20	9 070	89.0	0.12
14	6 350	62.3	0.34	22	9 980	97.9	0.17
16	7 260	71.2	0.60	24	10 890	106.7	0.24
18	8 160	80.0	1.00	26	11 790	115.6	0.34
20	9 070	89.0	1.59	28	12 700	124.5	0.46
22	9 980	97.9	2.44	30	13 610	133.4	0.62
24	10 890	106.7	3.62	32	14 520	142.3	0.82
26	11 790	115.6	5.21	34	15 430	151.2	1.07
28	12 700	124.5	7.31	36	16 320	160.1	1.38
30	13 610	133.4	10.03	38	17 230	169.0	1.75
32	14 520	142.3	13.51	40	18 140	177.9	2.19
34	15 430	151.2	17.87	42	19 070	186.8	2.73
36	16 320	160.1	23.30	44	19 980	195.7	3.36
38	17 230	169.0	29.95	46	20 880	204.6	4.11
40	18 140	177.9	38.02	48	21 790	213.5	4.98

Note: F-values shown are AASHO factors for flexible pavements whose structural number is 2.0 and for a terminal serviceability index of 2.0.

The significance of heavy overloads on highways is shown by the table. For example, F = 38.02 for the 40,000-lb single-axle load. This implies that one pass of the 40,000-lb axle load causes the same pavement damage as about 38 passes of an 18,000-lb single-axle load. In other words, if there were 500 vpd of an 18,000-lb single-axle load, only 13 vpd of the 40,000-lb axle load would cause the same degree of pavement damage.

In contrast to the significant effect of heavy-axle loads, the relatively insignificant effect of light vehicles such as passenger cars can also be observed. Although not shown in Table 8, a passenger vehicle with a single-axle load of 500 lbs has an F-value of about 0.0000001. Thus, it would take approximately 10,000,000 repetitions of this passenger vehicle axle-load to equal the destructive effect of 1 pass of the standard 18,000-lb single-axle load.

For this reason passenger cars are generally ignored in the traffic mix analysis and only trucks or commercial vehicles are considered. It is generally sufficient to include only those vehicles whose single-axle loads are greater than the 2000-lb value shown in Table 8.

These examples of the range of the relative damaging effect due to passenger vehicles and heavily loaded axles clearly illustrate that pavement damage is not linearly related to axle load. Studies have shown that the following relationships are applicable to the equivalent damage factors:

$$F = (L_1/18)^{4.5} \text{ and } F = (L_2/33.5)^{4.5}$$

where

L_1 = single-axle load in thousands of pounds, and
L_2 = tandem-axle load in thousands of pounds.

For example, if the F-value for a 42,000-lb tandem axle is desired,

$$F = (L_2/33.5)^{4.5} = (42/33.5)^{4.5} = 2.76.$$

This computed value of F = 2.76 agrees very well with the tabulated F = 2.73 value shown in Table 8. For many practical situations, sufficiently accurate F-values can be calculated by using a power of 4.0 instead of 4.5 and thus simplify the calculation. In the above example, use of the 4.0 power would give the approximation F = 2.47.

To make a detailed traffic analysis, quantitative values are needed for the following factors:

1. Average daily traffic (ADT),
2. Percentage of trucks (commercial vehicles),
3. Distribution of axle loads among trucks,
4. Number of lanes,
5. Design (service) life, and
6. Rate of traffic growth.

ADT and percentage of trucks can be obtained from traffic count studies on roads whose traffic conditions are assumed to be similar to those for the road being designed. The ADT is an estimate of the total number of vehicles in both directions that are expected to use the road on a typical day.

The percentage of trucks (commercial vehicles) is usually obtained by counting any vehicle having an axle load greater than about 2000 lb. The most difficult factor to evaluate is the distribution of the various axle loads of trucks within the traffic mix. This can normally be done only by weighing vehicles at roadside weigh stations or through portable weighing devices.

The number of traffic lanes on the roadway is important since the design number of equivalent repetitions is for a particular lane. Thus, if a two-lane facility is constructed, the ADT divided by 2 gives the number of vehicles per day traveling on the design lane. If only a single lane road is being constructed, the design number of vehicles per day on the design lane is the ADT value itself.

The total amount of traffic for which a road structure is designed clearly depends on the length of the design period. A design life must therefore be selected before the traffic analysis can be completed. For low-volume roads, a design period of 5 to 10 years is usually selected, perhaps coupled with a stage construction plan that will extend the life of the initial structure. In many cases, it can be expected that current traffic volumes will grow over the design period. Thus the current ADT value (and equivalent axle loads) will generally become larger in each successive year of the design period. Estimates of the annual rate of future growth can be projected from growth rates that have been observed in recent years.

Figure 15. Cumulative traffic growth factor as a function of traffic growth rate and design life.

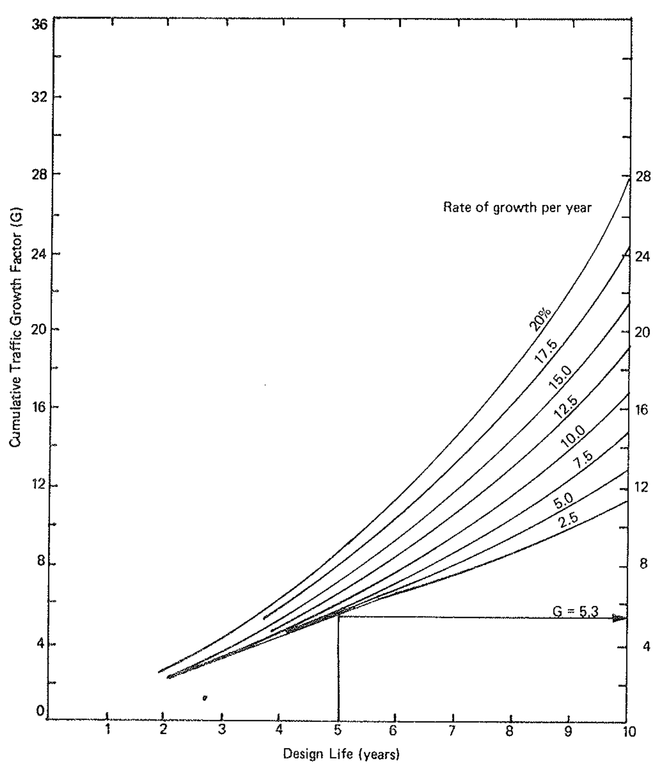


Figure 15 gives values for a growth factor G that is to be multiplied by the current ADT to give the accumulated number of vehicles to be experienced by the road structure for a given design life and annual rate of traffic growth. The figure shows, for example, that G is about 5.3 for a growth rate of 2.5% per year and a design life of 5 years.

Further information on the acquisition of traffic data is contained in Compendium 15 (see inside back cover).

Successive steps in a mixed-traffic analysis will be illustrated with the following hypothetical data:

1. Average daily traffic: ADT = 350 vehicles per day (two-way)
2. Number of lanes: 2
3. Percentage of trucks: 18%
4. Design life = 10 years
5. Axle load distribution per 100 trucks: (see Table 9)

Equivalent damage factors (F) in the next to last column of Table 9 were taken from Table 8. The final column is obtained by multiplying the F-values times the corresponding number of axle loads per 100 trucks. The final total of 161.16 is the number of 18,000-lb single-axle loads that are equivalent to the observed distribution of axle loads (per 100 trucks). It should be noted that the distribution refers only to the 18% of total ADT that are trucks and that 72% of the vehicles are ignored in the calculation of equivalent axle loads.

For the initial design year, the total number of equivalent 18,000-lb single-axle loads N_{18} is calculated as follows:

N_{18} (Year 1) = (350 vehicles per day/2 lanes) x (18 trucks/100 vehicles) x (161.16 equivalent axle-loads/100 trucks) x (365 days/year) = 18,530 equivalent 18,000-lb axle-loads.

From Figure 15, the growth factor for a 10-year design life and 15% traffic growth per year is about 21.8. Thus the total of N_{18} for all 10 years of the design life is

N_{18} (10 years) = 18,530 x 21.8 = 403,954 or approximately 400,000 equivalent 18,000-lb axle-load repetitions.

APPROXIMATION METHOD FOR MIXED-TRAFFIC ANALYSIS

For many low-volume road traffic analyses, available input data may be meager to nonexistent. It is also true that structural design requirements are not particularly sensitive for low N_{18} repetition values. Because of this, approximate solutions or estimates based on realistic selection of input values may be used.

In the example of the previous section, the traffic mix resulted in 161.16 equivalent N_{18}

Table 9. Hypothetical axle load distribution and equivalent axle repetitions.

Axle-Load Distribution from Weighing Study					
Axle Type	Axle Load (lb 000s)	Number of Trucks per 100 Trucks	Number of Axles per 100 Trucks	Equivalent Damage Factor per Axle Load (F-values from Table 8)	Equivalent Axles per 100 Trucks
Single	8	27	54	0.04	2.16
	16	46	92	0.60	55.20
	24	9	18	3.62	65.16
Tandem	28	6	12	0.46	5.52
	36	12	24	1.38	33.12
Total		100	200		161.16

Table 10. Traffic mix factor (M)[a].

Load Distribution (N_{18} per truck)	Percentage of Trucks		
	Low (under 15%)	Medium (15-25%)	High (more than 25%)
Light (under 0.75)	9	18	27
Medium (0.75-1.5)	23	46	69
Heavy (more than 1.5)	37	73	110

[a]Values shown are for two-lane roads and are to be doubled for one-lane roads. In many cases, local conditions are such that even on two-lane roads, major traffic is concentrated in the center of the road except for passing situations. In these cases, the designer should also double the table value.

repetitions per 100 trucks, or 1.61 repetitions per truck. This value is somewhat typical of low-volume roads that frequently carry heavy loads. Recent research (16, 17) has shown that in developing countries this ratio may range from about 0.5 to 2.0. While this is a fourfold difference, it does allow for the selection of realistic ranges that may be encountered in low-volume road design. In addition the typical range of percent trucks has been found to be 5-15%, although values up to about 50% have been observed.

By using this information, a traffic mix factor (M) can be determined for convenient categories of percent trucks (low, medium, and high) and categories of the probable axle-load distributions (equivalent repetition-truck ratio expressed in categories of light, medium, and heavy loads). Table 10 gives values for M for nine combinations of truck percentage and load category. When the M factor is multiplied by the value of ADT, the result is an approximation of N_{18} in the initial design year. For the example of the previous section, the M factor in Table 10 is found in the column for medium percent trucks (18%) and in the row for heavy-load distribution (1.61 repetitions per truck). Thus M = 73 for the example, and the approximate value for N_{18} in the initial design plan is

N_{18} (Year 1) = M x ADT = 73 x 350 = 25,550 equivalent 18,000-lb axle-loads.

By using the same growth factor as before, G = 21.8, the approximation method gives

N_{18} (10 Years) = 25,550 x 21.8 = 556,990 or approximately 557,000 equivalent repetitions.

In this example, the approximation method produced 557,000-400,000 = 157,000 more equivalent repetitions than did the method based on the observed axle-load distribution. In general, however, the approximation method gives sufficiently accurate results for the design of low-volume roads.

EQUIVALENT SINGLE-WHEEL LOAD ANALYSIS

In some special design situations, traffic may consist of low frequency but extremely heavy loads and may include special tire configurations that do not lend themselves directly to the use of typical single-axle or tandem-axle equivalent damage factors. In these situations, an equivalent single-wheel load (ESWL) approach may be used for structural design. This approach is particularly recommended when the structure is likely to experience excessive shear displacements that are due to the heavy or special vehicle. Since details for the ESWL approach are relatively complex and lengthy, they will not be presented in this synthesis. The interested reader will find full details in other works (4, 18, 19).

CHAPTER 5

Soils and Subgrades

This chapter presents basic concepts for the description and classification of soils and for the measurement of soil strength. It must be recognized, however, that behavior of subgrade soil depends greatly on (a) the environmental conditions in which the road is constructed and (b) the level of construction activity used to prepare natural soil for use as a pavement subgrade. These two factors are discussed in Chapters 6 and 8, respectively.

Soil in its basic form may be viewed as a combination of differing amounts (or percentages) of separate soil size grains. The size of individual grains may vary from coarse (gravels and sands) to fine (silts and clays). Early in soils engineering, it was recognized that the distribution of soil sizes, called grain size distribution or soil gradation, was an important physical property of a soil. Later experience showed, however, that other properties such as the plasticity characteristics of the fine portion were equally as important.

As a result, it is not uncommon for the engineer to encounter two distinctly different soil classification schemes. In one scheme soil types (e.g., gravel, sand, silt, and clay) are defined solely by ranges in soil particle sizes. This approach leads to a textural soil classification that is based on the relative percentages of sand, silt, and clay sized particles in the soil.

Although many important inferences can be made from such a classification scheme, the best scheme for engineering purposes is one that takes into account both gradation and plasticity characteristics. In general, grain size distribution is an important property for coarse-grained soils, and plasticity is the most important property for fine-grained soils. These principles have led to the establishment of engineering soil classification systems. The two most widely used systems of this type are the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO, formerly AASHO) system.

Table 11. Summary of soil size nomenclature.

Soil Size	Grain Size (mm) ASTM 0422	AASHO T88	MIT	USCS	International
Gravel	75-4.75		100-2.0	2.0-1.0	
Coarse sand	4.75-2.0	2.0-0.425	2.0-0.6	1.0-0.5	2.0-0.5
Medium sand	2.0-0.425		0.6-0.2	0.5-0.25	0.5-0.2
Fine sand	0.425-0.075	0.425-0.075	0.2-0.06	0.25-0.05	0.2-0.1
Silt size	0.075-0.005	0.075-0.002	0.06-0.002	0.05-0.005	0.02-0.002[a]
Clay size	0.005-0.001	0.002-0.001	<0.002	<0.005	0.002-0.0002
Colloids	<0.001	<0.001			<0.002

[a] Formerly Swedish classification; has additional soil-size category called "Mo" (0.1 - 0.02 mm).

Table 12. Standard U.S. sieve properties.

Size (in)	Size Opening (mm)	Sieve No.	Opening (mm)
4	101.6	No. 4	4.76
3	76.1	No. 8	2.38
2	50.8	No. 10	2.00
1	25.4	No. 20	0.841
3/4	19.0	No. 40	0.420
1/2	12.7	No. 100	0.149
3/8	9.51	No. 200	0.074
1/4	6.35	No. 270	0.053
		No. 400	0.037

Table 13. Illustrative soil gradation analysis[a].

Sieve Size	Size Opening (mm)	Weight Retained (g)	Retained (%)	Passing (%)
1 in	25.4	0.0	0.0	100.0
3/4 in	19.0	100.0	10.0	90.0
1/2 in	12.7	110.0	11.0	79.0
No. 4	4.76	250.0	25.0	54.0
No. 10	2.00	150.0	15.0	39.0
No. 20	0.841	100.0	10.0	29.0
No. 40	0.420	60.0	6.0	23.0
No. 100	0.149	80.0	8.0	15.0
No. 200	0.074	50.0	5.0	10.0
Pan		100.0	10.0	0.0

[a] Total weight = 1000 gm.

SOIL SIZE AND GRADATION

Ever since the treatment of soils as a science, various agencies have attempted to describe the general ranges of soils that comprise the major types of soils (gravels, sands, silts, and clays). A summary of various size classifications is shown in Table 11. As can be seen, there is no universal agreement on the soil sizes that are associated with particular soil types.

Within a particular soil deposit the distribution of grain size (gradation) may be found by a combination of mechanical (sieve) analysis and hydrometer testing. Mechanical sieving is practical for sizes generally greater than 0.074 mm (No. 200). Hydrometer analysis, based on Stoke's law for particle flow in a viscous fluid, is generally applicable for soil sizes finer than the 1.0 mm-0.5 mm range. For most engineering classifications the distribution of grain size for silt and finer soils is not required. Use of mechanical techniques for grain size is usually sufficient for engineering practice. Only if classification by a textural system is desired is there a need to conduct hydrometer studies.

Mechanical analysis of soils is accomplished by placing the soil through a series of stacked sieves (usually 5 to 8) and weighing the amount of soil retained on each sieve. Table 12 lists the sieve nomenclature and size of the opening for several common sieves. Various intermediate-sized sieves are commercially available. The same size opening for a sieve may have different designations in different countries. For example, a sieve with an opening of 2.00 mm is equivalent to a No. 10 U.S. sieve, a No. 8 British standard sieve, and a No. 45 French sieve. Although sieve sizes smaller than a No. 200 sieve are shown, these sieves are not generally used.

Table 13 illustrates the results and computations of a mechanical analysis test for a given material. After sieving is completed, the weight retained on each sieve (and pan) is determined. By using the total weight of the original sample, the percentage retained and the total percentage passing a given sieve can be easily computed. It is common for the grain size distribution results to be plotted in terms of percentage passing versus log size opening. Figure 16 is such a plot developed for the example shown in Table 13.

Both the data in Table 13 and the plot of Figure 16 indicate that there is a relatively equal percentage of soil sizes between adjacent sieves. In this case the material is said to be well-graded. Figure 17 shows several gradations that the engineer may encounter. Soil A is the well-graded soil of Figure 16. For soil B there is a general deficiency of material between the No. 4 and the No. 100 sieves. This gradation is said to be a gap-graded material to denote that a number of soil sizes are not present in the soil. The gradation curve for soil C shows that almost all of this soil is within a relatively small size range. Such a distribution represents a uniform-graded material. The term poorly graded is generally used to refer to all materials that are not well-graded.

Specific points on the grain size curve may be used to determine if a particular soil is well-graded or poorly graded. For example, the coefficient of uniformity (C_u) is defined by $C_u = D_{60}/D_{10}$ where D_{60} is the grain size for which 60% of the total material is finer, and where D_{10} is the grain size or which 10% of the total material is finer. To have a well-graded material, the C_u should be greater than 4 for gravelly soils and greater than 6 for sandy soils. For the soil shown in Figure 16, D_{60} = 6.2 mm and D_{10} = 0.074 mm. Thus C_u = 6.2 mm/0.074 mm = 83.8.

Another coefficient that characterizes soil grading is $C_c = (D_{30})^2/(D_{10}D_{60})$.

The D_{30} value is the grain size at which 309% of the material is finer or passes the sieve. If the soil is well-graded, the value of C_c is generally between 1 and 3. Poorly graded materials result in large C_c values. For the soil shown in Figure 16, D_{30} = 0.9 mm and $C_c = (0.9)^2/(0.074)(6.2) = 1.77$.

Because C_u is greater than 6 and C_c is between 1 and 3, the grain size distribution curve shown in Figure 16 represents a well-graded material.

Figure 16. Typical soil grain size distribution curve.

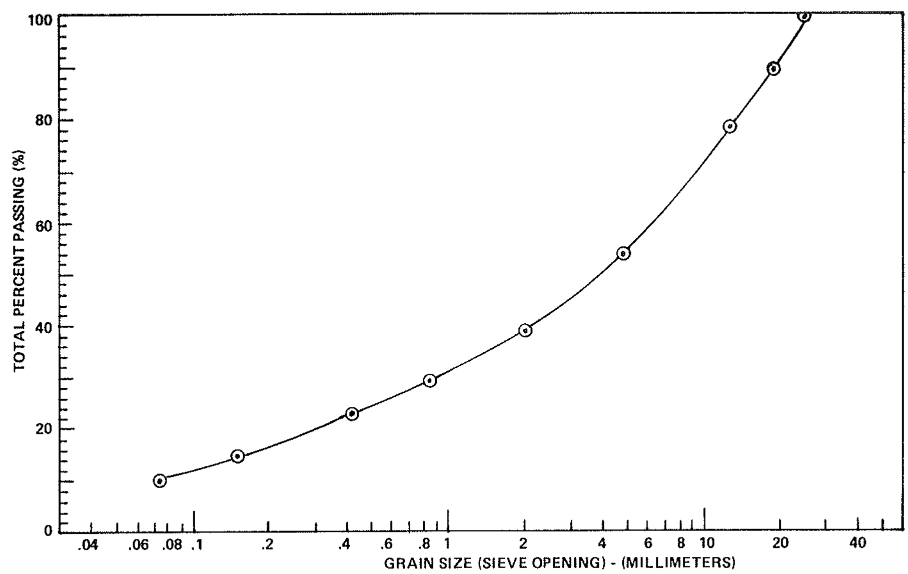
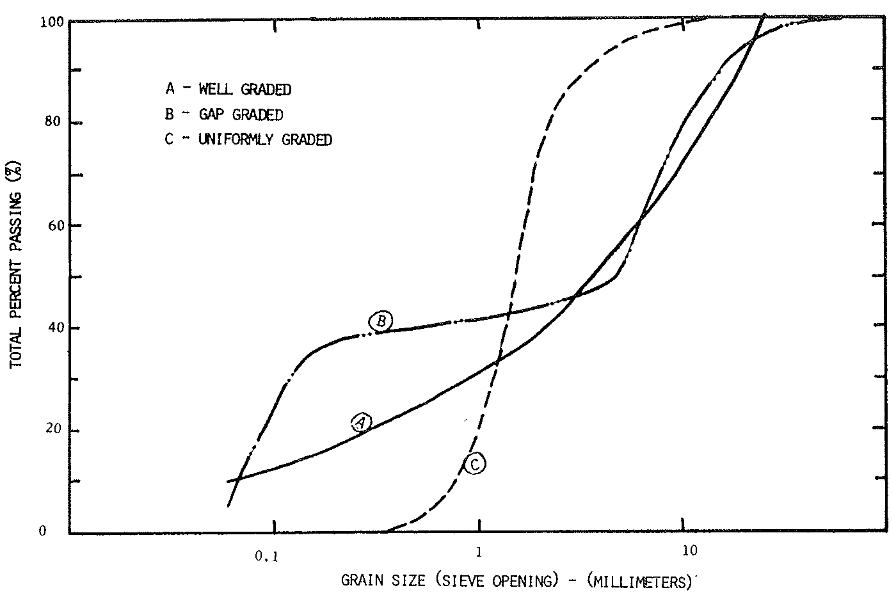


Figure 17. Typical types of soil gradations.



SOIL CONSISTENCY

Simple, but effective, laboratory procedures have been developed to delineate silty (noncohesive) soils from clayey (cohesive) soils. These procedures were developed by the Swedish agronomist, Atterberg, at the turn of the century. Although Atterberg developed several indices, only three are widely used by soils engineers today. These parameters are called the Atterberg Limits and are defined as follows: w_λ (or LL) = Liquid Limit, w_p (or PL) = Plastic Limit, and I_p (or PI) = Plasticity Index = $w_\lambda - w_p$, where w is the soil moisture content at the two limits, respectively. Moisture content is defined by w = 100 (weight of wet soil - weight of oven dry soil)/(weight of oven dry soil). Thus moisture content is the ratio of the weight of water driven off in the drying process to the weight of the soil in its dry state, expressed as a percent.

The Atterberg limits are based on the change in soil consistency as moisture (water) is added to the soil. Soil, when dry or slightly wet, exists in a semi-solid to solid state. On adding water, the soil tends to exhibit a more plastic consistency in that remolding can be easily accomplished. Finally, if enough water is added, the plastic state will be transformed into a viscous condition where soil flow may be expected to occur. The Atterberg limits represent soil moisture contents that delineate the three major phase changes in consistency. The recommended procedure for determining these values is highly standardized, highly empirical, but highly reproducible for any soil. It should be noted that Atterberg limits are only conducted on material passing the No. 40 sieve.

The Plastic Limit test (ASTM D-424 or AASHTO T-90) defines the lowest moisture content at which the soil is in a plastic state. The test involves rolling a thread of soil into a diameter of 1/8 in so that cracks are not visible in the soil thread. The soil moisture content at which this occurs is called the Plastic Limit and is denoted by w_p (or PL).

The Liquid Limit test (ASTM D-423 or AASHTO T-89) defines the moisture content at which the soil changes from a plastic state to a more fluid or liquid state. Specific details of the test procedure and lab equipment may be found in the test specifications.

The Plasticity Index (I_p or PI) describes the range of moisture contents over which the soil will be in a plastic state. Thus, from an engineering viewpoint, the larger the I_p value, the more clayey or plastic is the material. For example, if a soil has a w_λ = 18%, and w_p = 12%, then I_p =18 - 12 = 6%. This range of moisture contents over which the soil is in a plastic state can be observed to be significantly lower than a more plastic soil with w_λ = 70%, w_p = 22%, and I_p = 48%.

ENGINEERING CLASSIFICATION SCHEMES

In engineering classification schemes, gradation properties are deemed most important for granular soils but not important for fine-grained soils. For fine-grained soils, plasticity properties are most important but gradation properties are not. For mixed-type soils, both gradation and plasticity properties are important.

Thus the influence of soil size distribution becomes less significant and the effect of plasticity characteristics more significant as the soil type changes from a coarse-grained to a fine-grained soil. Two major engineering classification schemes are used worldwide. They are the USCS and the AASHTO systems.

The USCS system was developed originally by A. Casagrande and was later modified by the U.S. Army Corps of Engineers. The following symbols are used:

Coarse-Grained Soils
- G-Gravel
- S-Sand

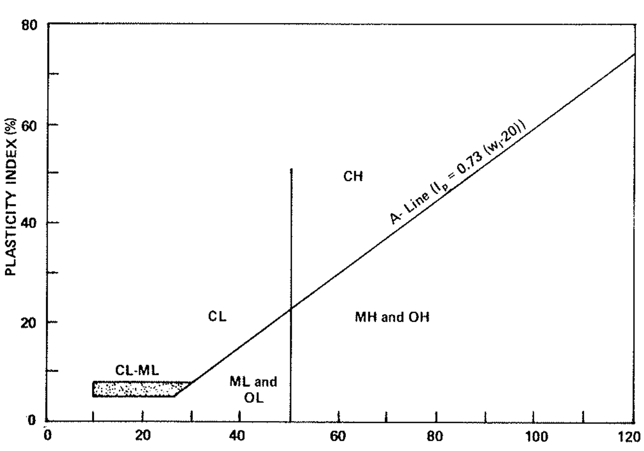
Fine-Grained Soils
- M-Silt
- C-Clay
- O-Organic
- Pt-Peat

Gradation Properties
- W-Well-Graded
- P-Poorly Graded

Liquid Limit (Compressibility Properties)
- H-High-Compressibility
- L-Low-Compressibility

Coarse-grained soils (G or S) are those with less than 50% passing a No. 200 sieve, while fine-grained soils (M,C,O) are those for which more than 50% pass

Figure 18. Plasticity chart for use with USCS.



a No. 200 sieve. If the soil is fine-grained, use is made of the plasticity chart shown in Figure 18 to completely classify the soil. This is accomplished by plotting the w_λ and I_p values of the soil and noting the area in which these values lie. The A line separates engineering clays from engineering silts in that any point above the A line is designated by a C or clay prefix. A liquid limit value of 50 separates fine-grained soils into high (H) and low (L) compressible zones. In general, organic and peaty materials (O and Pt) are classified on the basis of visual appearance and odor. If the soil is coarse-grained, the first order of subdivision is based on the No. 4 sieve that is used to delineate gravel from sand sizes. If the greatest proportion of the coarse-grained material is retained on the No. 4 sieve, then the soil is a Gravel (G). Likewise, if the greatest percentage of the coarse-grained material is within the No. 4 to No. 200 sieves, then the material is classified as a Sand (S).

Various subgroups of either G or S soils are then made on the basis of the amount of fine-grained material in the soil (passing No. 200). If 5% or less pass a No. 200 sieve, the G or S symbols are modified to reflect whether or not the soil is well or poorly graded (i.e., GW, GP, SW, or SP) based on the C_u and C_c values previously discussed in this chapter. If the soil contains more than 12% fines (passing No. 200), it is desirable to modify the symbol to reflect whether the fine-grained portion is clayey (C) or silty (M) in nature. For this case, the plasticity properties are used (Atterberg limits) along with Figure 18 to determine if the material lies above or below the A line. Thus possible USCS symbols for this category are: (GM, GC, SM, SC). Finally, if the percentage passing the No. 200 sieve is intermediate (between 5% and 12%), both the characteristics of the gradation and plasticity properties of the fine-grained soil portion are noted by the use of a dual symbol. A classification symbol of GW-GC would be indicative of this group. The USCS classification scheme is shown in Table 14.

The present AASHTO classification scheme evolved from the U.S. Bureau of Public Roads (BPR) procedure that was introduced in 1929. In this system, coarse-grained soils are separated from fine-grained soils on the basis of 35% passing the No. 200 sieve. It is apparent that, if a soil has between 35% and 50% passing a No. 200 sieve, the AASHTO classification results in a fine-grained soil symbol; the USCS system would necessitate a coarse-grained notation. These subtle differences should be kept in mind when trying to infer soil performance from classification schemes.

The AASHTO scheme uses a subdivision procedure based on the relative influence of gradation and plasticity characteristics. Eight major soil groups are designated by the symbols A-1 through A-8. The symbols are arranged so that A-1, A-2, and A-3 soils are coarse-grained, A-4 and A-5 are silts, A-6 and A-7 are clays, and A-8 is organic. Thus, soils A-4 through A-8 are fine-grained. Within the A-1, A-2, and A-7 groups, subgroups are based on the Atterberg limits (w_λ and I_p) of the fine-grained soil portion. Table 15 shows the classification scheme for the AASHTO system. In general, the technique is to start at Group A-1 and determine the first group (or subgroup) that satisfies the gradation and plasticity limits specified in Table 15.

One additional feature of the AASHTO system is that it provides a numeric indicator for soil relative to its probable performance as a subgrade soil. This is accomplished by the Group Index (GI) parameter where a GI = 0 indicates an excellent

Table 14. USCS scheme (20).

Major Division		Group Symbol	Laboratory Classification Criteria: Finer than 200 Sieve (%)	Supplementary Requirements	Soil Description
Coarse-grained (more than 50% by weight coarser than No. 200 sieve	Gravelly soils (more than half of coarse fraction larger than No. 4)	GW	0-5[a]	D_{60}/D_{10} greater than 4, $D_{30}^2/(D_{60} \times D_{10})$ between 1 and 3	Well-graded gravels, sandy gravels
		GP	0-5[a]	Not meeting above gradation for GW	Gap-graded or uniform gravels, sandy gravels
		GM	12 or more[a]	PI less than 4 or below A-line	Silty gravels, silty sand gravels
		GC	12 or more[a]	PI over 7 and above A-line	Clayey gravels, clayey sandy gravels
	Sandy soils (more than half of coarse fraction finer than No. 4)	SW	0-5[a]	D_{60}/D_{10} greater than 4, $D_{30}^2/(D_{60} \times D_{10})$ between 1 and 3	Well-graded, gravelly sands
		SP	0-5[a]	Not meeting above gradation requirements	Gap-graded or uniform sands, gravelly sands
		SM	12 or more[a]	PI less than 4 or below A-line	Silty sands, silty gravelly sands
		SC	12 or more[a]	PI over 7 and above A-line	Clayey sands, clayey gravelly sands
Fine-grained (more than 50% by weight finer than No. 200 sieve)	Low compressibility (liquid limit less than 50)	ML	Plasticity chart		Silts, very fine sands, silty or clayey fine sands, micaceous silts
		CL	Plasticity chart		Low plasticity clays, sandy or silty clays
		OL	Plasticity chart, organic odor or color		Organic silts and clays of low plasticity
	High compressibility (liquid limit more than than 50)	MH	Plasticity chart		Micaceous silts, diatomaceous silts, volcanic ash
		CH	Plasticity chart		Highly plastic clays and sandy clays
		OH	Plasticity chart, organic odor or color		Organic silts and clays of high plasticity
Soils with fibrous organic matter		Pt	Fibrous organic matter; will char, burn, or glow		Peat, sandy peats, and clayey peat

[a]For soils having 5 to 12% passing the No. 200 sieve, use a dual symbol such as GW-GC.

Table 15. AASHTO soil classification scheme (20).

Group	Subgroup	Percent Passing U.S. Sieve No. 10	40	200	Character of Fraction Passing No. 40 Sieve: Liquid Limit	Plasticity Index	Group Index No.	Soil Description	Subgrade Rating
A-1			50 max	25 max		6 max	0	Well-graded gravel or sand; may include fines	Excellent to good
	A-1-a	50 max	30 max	15 max		6 max	0	Largely gravel but can include sand and fines	
	A-1-b		50 max	25 max		6 max	0	Gravelly sand or graded sand; may include fines	
A-2[a]				35 max			0 to 4	Sands and gravels with excessive fines	
	A-2-4			35 max	40 max	10 max	0	Sands, gravels with elastic silt fines	
	A-2-5			35 max	41 min	10 max	0	Sands, gravels with elastic silt fines	
	A-2-6			35 max	40 max	11 min	4 max	Sands, gravels with clay fines	
	A-2-7			35 max	41 min	11 min	4 max	Sands, gravels with highly plastic clay fines	
A-3			51 min	10 max		Nonplastic	0	Fine sands	
A-4				36 min	40 max	10 max	8 max	Low-compressibility silts	Fair to poor
A-5				36 min	41 min	10 max	12 max	High-compressibility silts, micaceous silts	
A-6				36 min	40 max	11 min	16 max	Low to medium-compressibility clays	
A-7				36 min	41 min	11 min	20 max	High-compressibility clays	
	A-7-5			36 min	41 min	11 min[b]	20 max	High-compressibility silty clays	
	A-7-6			36 min	41 min	11 min[b]	20 max	High-compressibility, high-volume-change clays	
A-8								Peat, highly organic soils	Unsatisfactory

[a]Group A-2 includes all soils having 35% or less passing a No. 200 sieve that cannot be classed as A-1 or A-3.
[b]Plasticity index of A-7-5 subgroup is equal to or less than LL-30. Plasticity index of A-7-6 subgroup is greater than LL-30.

subgrade and a GI = 20 indicates a very poor subgrade soil. The basic equation used to determine this value is

$$GI = 0.2a + 0.005ac + 0.01bd$$

where

a = percentage passing the No. 200 screen minus 35 (min = 0, max = 40),

b = percentage passing the No. 200 screen minus 15 (min = 0, max = 40),

c = Liquid limit minus 40 (min = 0, max = 20), and

d = Plasticity Index minus 10 (min = 0, max = 20).

It is customary to place the GI value in parentheses after the AASHTO symbol. Thus, a proper classification symbol would be A-7-6 (20).

Use of both the USCS and AASHTO classification systems will be illustrated for two different soils whose gradation and plasticity values are given below.

	Soil 1	Soil 2
% passing Sieve No. 4	51	100
% passing Sieve No. 10	43	98
% passing Sieve No. 40	32	94
% passing Sieve No. 100	22	75
% passing Sieve No. 200	4	61
Liquid Limit w_λ	NP	48.3
Plastic Limit w_p	non-plastic	23.1

USCS Classification of Soil 1 (see Table 14)

(1) % Passing No. 200 = 4%, so must be G or S.

(2) % Gravel = (%Ret.No. 4)/(%Ret. No. 200) =

(100-51)/(100-4) = 49/96 = 0.51, greater than 50% so must be G.

(3) Since % Passing No. 200 is less than 5%; use W or P to describe gradation.

(4) Plot grain size curve (not shown) and determine

D_{60} = 10.0 mm
D_{30} = 0.33 mm
D_{10} = 0.09 mm

(5) Compute C_u and C_c values.

(6) $C_u > 4$ but $C_c < 1$, use P for poor gradation.

(7) Classification is GP.

USCS Classification of Soil 2 (see Table 14)

(1) % Passing No. 200 = 61%, so must be C or M.

(2) Go to Figure 18 with Atterberg data

w_λ = 48.3%
$I_p = w_\lambda - w_p = 48.3 - 23.1 = 25.2\%$.

(3) Plot is in area marked CL, so classification is CL. Final classification is CL.

AASHTO Classification of Soil 1 (see Table 15)

(1) % Passing No. 200 = 4%, so must be A-1, A-2, or A-3.

(2) % Passing No. 10 = 43.
% Passing No. 40 = 32.
Cannot be A-3 as % Passing No. 40 is less than 51 minimum; Cannot be A-1-a as % Passing No. 40 is greater than 30 maximum.

(3) Must be either A-1-b or A-2; look at Atterberg data.

(4) As soil is NP (I_p = 0) cannot be A-2.

(5) Soil classification symbol must be A-1-b.

(6) Compute GI. a = b = c = d = 0, so GI = 0

(7) Final Classification A-1-b (0).

AASHTO Classification of Soil 2 (see Table 15)

(1) % Passing No. 200 = 61% so must be A-4, A-5, A-6, or A-7.

(2) Look at I_p value = 25.2%; cannot be A-4 or A-5.

(3) Look at w_λ = 48.3%; cannot be A-6; must be A-7 group.

(4) Look at A-7 subgroup (A-7-5 or A-7-6); I_p of A-7-6 must be greater than w_λ = 30; since I_p = 25.2% > $w_\lambda - 30$ = 48.3 - 30 = 18.3%, the soil must be A-7-6.

(5) Compute GI; a = 61 - 35 = 26; b = 61 - 15 = 46 (use 40 maximum); c = 48.3 - 40 = 8.3; d = 25.2 - 10 = 15.2; then GI = 0.2(26) + 0.005(26) (8.3) + 0.01(40)(15.2) = 12.4.

Further information on soil exploration and classification is contained in Compendium 6 (see inside back cover).

SOIL STRENGTH

For granular-surfaced roads, the selection of a design subgrade strength value is a very significant step in the design process. The selection is complicated because of (a) inherent soil variability, (b) effect of climatic and subsurface drainage conditions on soil strength, and (c) uncertainty of quality control during road construction.

In many cases, designs must be developed with very limited descriptions of the subgrade soil. Wherever possible, attempts should be made to conduct actual strength tests on the typical subgrade soil. Many different methods have been developed to measure soil strength, but in this synthesis only the California Bearing Ratio (CBR) method will be discussed. The CBR method has worldwide acceptance as a proven method of soil strength and design.

In the absence of laboratory strength test data, approximate or indirect correlations of soil strength with soil classification information may be used. Figure 19 shows general strength correlations of the AASHTO and USCS soil classes to the California Bearing Ratio (CBR) value. It can be seen that for any given soil class, a rather wide range of probable CBR values is shown. The sensitivity of thickness design to subgrade strength (CBR) is much more critical at very low strengths than for high-strength materials. For example, changing the design CBR from a value of 20 to 40 may result in thickness reductions of the structure by no more than 1 in (2 to 3 cm). However, the difference between a design CBR = 2 and CBR = 4 may result in structural designs that vary by as much as 10 in (25 cm) of granular material.

The CBR test was originally developed in the 1920s by the California Division of Highways and was adopted by the U.S. Army Corps of Engineers in the early 1940s. While the test may be conducted in situ, it is normally performed as a laboratory test on remolded specimens. In essence, the test is an empirically developed penetration test that relates the resistance of the tested soil to the resistance of a standard crushed stone material at given penetration levels.

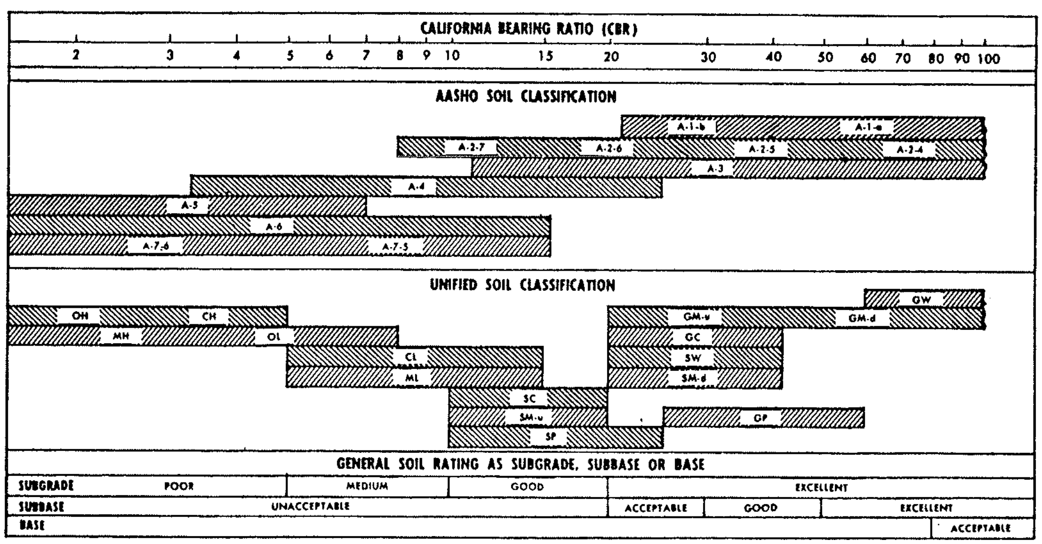
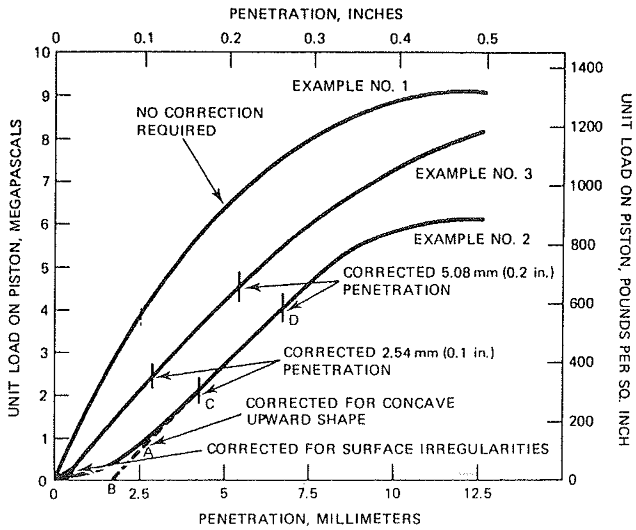


Figure 19. General soil classification: strength correlation (21).

Figure 20. Typical load-penetration curves for CBR test.



The CBR test value is extremely dependent on (a) the moisture-density of the sample after laboratory compaction and (b) whether or not sample soaking is allowed before testing.

The test is conducted on a soil sample that has been remolded and compacted to a particular density-moisture state that is intended to stimulate in situ field conditions. A 6-in (15.25-cm) diameter specimen is usually used. Before testing, surcharge weights are placed on top of the soil within the mold to simulate the anticipated overburden pressure due to pavement layers above the material being tested. A load piston and deflection gages are used to read the necessary piston load that is required to achieve a preset piston penetration. Loads are generally read at the following penetrations:

Penetration	Penetration
0.6 mm (0.025 in)	4.4 mm (0.175 in)
1.3 mm (0.050 in)	5.0 mm (0.200 in)
1.9 mm (0.074 in)	6.4 mm (0.250 in)
2.5 mm (0.100 in)	7.5 mm (0.300 in)
3.2 mm (0.125 in)	10.0 mm (0.400 in)
3.8 mm (0.120 in)	12.5 mm (0.500 in)

The piston has a standard cross-sectional area of 3 in^2 (1935.5 mm^2) and is penetrated at a uniform rate of 0.5 in/min (1.27 mm/min). The load and penetration data are then plotted as shown in Figure 20. Load data are converted to unit piston pressures by dividing the loads by the cross-sectional area of the piston.

As noted in the figure, it may be necessary to correct the smooth curve drawn through the data. The need for data correction can only be visually determined from the original pressure-penetration curve. For this reason, it is highly important that the results be plotted before any calculations of CBR are made. If corrections are required, the procedure is to find a new corrected zero penetration point. Adjusted pressures, using the corrected penetration values, are then used to calculate the CBR value.

The CBR value is determined by multiplying 100 times the unit piston pressure on the tested soil at a given penetration, then dividing by the unit piston pressure on the standard crushed stone at the same penetration.

Standard loads (and pressures) for the crushed stone material at different penetrations are shown below:

Penetration		Standard Load		Standard Pressure	
(mm)	(in)	(N)	(lb)	(MPa)	(psi)
2.5	0.1	13345	3000	6.89	1000
5.0	0.2	20017	4500	10.34	1500
7.5	0.3	25335	5700	13.10	1900
10.0	0.4	30693	6900	15.86	2300
12.5	0.5	34696	7800	17.93	2600

For each test, it is usual to compute CBR values at the 2.5-mm and 5.0-mm (0.1-in and 0.2-in) penetrations. Usually, the 2.5-mm penetration will have a larger CBR. However, if this does not occur, then the test should be rerun. If similar results are obtained on the retest the largest CBR value should be recorded.

As an example, suppose the CBR curve shown in Example No. 1 (Figure 20) represented the design soil condition. From this curve, the following unit loads are noted:

Penetration (in)	Unit Pressure (psi)
0.1-in	580 psi
0.2-in	950 psi

Therefore:

CBR(0.1 in) = 100 X 580 psi/1000psi = 58%
CBR(0.2 in) = 100 X 950 psi/1500psi = 63%

The CBR of the soil material would be 63% provided that a retest gives similar results for the 0.2 in penetration.

Further information on soil strength is contained in Compendium 10 (see inside back cover).

CHAPTER 6

Environmental Factors

For the types of structures considered in this synthesis, major environmental factors are the influences of moisture and frost on the strength and volume of structural materials. Only moisture effects will be discussed since frost action is seldom encountered in the great majority of developing countries.

MOISTURE-STRENGTH CONSIDERATIONS

In general, subgrade strength is a function of the in situ moisture and density conditions of the soil. These variables are governed by certain fundamental relationships that are normally applicable to all soil types. Figure 21 illustrates the effect of moisture content and compaction effort on soil strength as determined from laboratory tests.

If a soil, at a given moisture content, is compacted in the laboratory within a standard volumetric mold, a certain dry density is obtained (Figure 22). At a given level of compaction effort, the dry density will increase with increased higher moisture contents until a maximum or peak density occurs. Beyond this moisture content, the density will decrease. Thus, for a given constant level of compactive effort, an optimum dry density occurs at a moisture content defined as optimum moisture. In Figures 21 and 22 three such compaction curves are shown for high, medium, and low levels of compaction energy. Each compaction curve has its own values for optimum moisture content and optimum dry density. A line connecting the optimum points on various compaction curves is sometimes referred to as the Line of Optimums. It should also be noted that as the level of compaction increases, the optimum moisture decreases and the optimum density increases. Also shown in Figure 22 is the Zero Air Voids line. This line represents the maximum possible moisture content for a given dry density and, therefore, represents a completely saturated soil.

Figure 21 indicates how soil strength (CBR) changes with moisture content for each of the three compaction levels. This strength is associated with the moisture content at which the soil specimen is compacted or molded, and is usually called the as-molded strength of the soil. Comparison of Figures 21 and 22 indicates that for each level of compaction, maximum CBR strength occurs at approximately the optimum moisture value. Furthermore, the strength decreases significantly as the moisture content is increased beyond the optimum level.

While the relationships shown in these figures clearly demonstrate the importance of moisture content, it should be recalled that the strengths obtained are associated with the as-molded or as-compacted moisture contents. Once the soil is compacted at a given moisture content, it can either gain or lose moisture before reaching its in situ or equilibrium condition. It is the in situ moisture content that will dictate the actual structural strength. It is basic, therefore, to estimate the environmental conditions that a specific subgrade soil will experience during its structural life.

Although some designers employ a conservative approach by specifying soaked-strength values for design under all climatic conditions, recent research by the Transport and Road Research Laboratory (TRRL) of Great Britain has led to the development of general guidelines for estimating the probable in situ moisture content for various environmental conditions (1, 3, 22, 23,). The fundamental concepts of this approach are as listed below.

1. The in situ density of the subgrade can be controlled within limits by compaction during construction.
2. The equilibrium or in situ moisture content is a function of the soil type, local climatic conditions, and the depth of the ground water table below the road surface.
3. The design strength should be based on a moisture content equal to the wettest moisture condition likely to occur in the subgrade after the road is open to traffic.

Table 16 may be used to estimate CBR design values for different types of soils and various ground water conditions.

Figure 21. Effect of moisture content and compaction on soil strength.

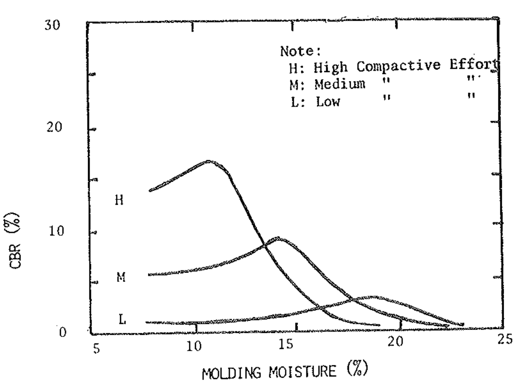


Figure 22. Effect of moisture content and compaction on dry density.

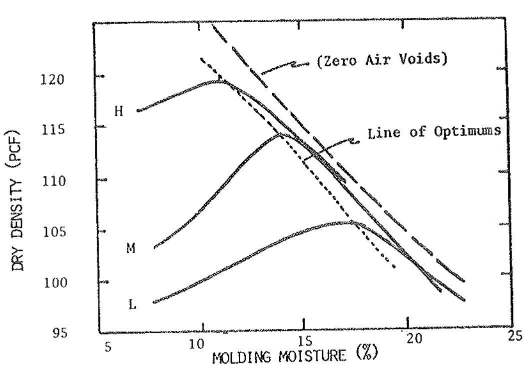


Table 16. Estimated design CBR values for subgrades compacted to at least 95 percent standard density.

Minimum Seasonal Depth of Water Table (m)	Non-Plastic Sand	Sandy Clay PI = 10	Sandy Clay PI = 20	Silty Clay PI = 30	Heavy Clay PI ≥ 40	Silt
0.6	8	5	4	3	2	1
1.0	25	6	5	4	3	2
1.5	25	8	6	5	3	Use laboratory tests
2.0	25	8	7	5	3	Use laboratory tests
2.5	25	8	8	6	4	Use laboratory tests
3.0	25	25	8	7	4	Use laboratory tests
3.5	25	25	8	8	4	Use laboratory tests
5.0	25	25	8	8	5	Use laboratory tests
7.0	25	25	8	8	7	Use laboratory tests

HIGH-VOLUME CHANGES

A worldwide problem is damage to road structures because of high-volume change in subgrade soils. High-volume changes in road structures generally result in differential heaving and can lead to excessive road roughness.

Certain conditions must occur simultaneously before the high-volume change results in excessive heave or swell. In general, these conditions include (a) presence of a potentially high-volume change soil and (b) potential for change in soil-moisture from the as-constructed phase to the in situ equilibrium phase. Soil heave can be brought about by either the natural environment or by moisture changes that are introduced during construction.

The swelling potential of a soil can be described by swell percentage, which is the amount of vertical expansion from the intial height of a soil sample. Swell pressure refers to the vertical stress that is required to hold a test specimen to zero volume change.

Studies have shown that greater volume changes can be expected with increasing colloid content and plasticity index. One of the most widely used techniques to determine the swell potential of soils is based on correlations with the Plasticity Index of the soil (24, 25). Typical values of this correlation are listed below.

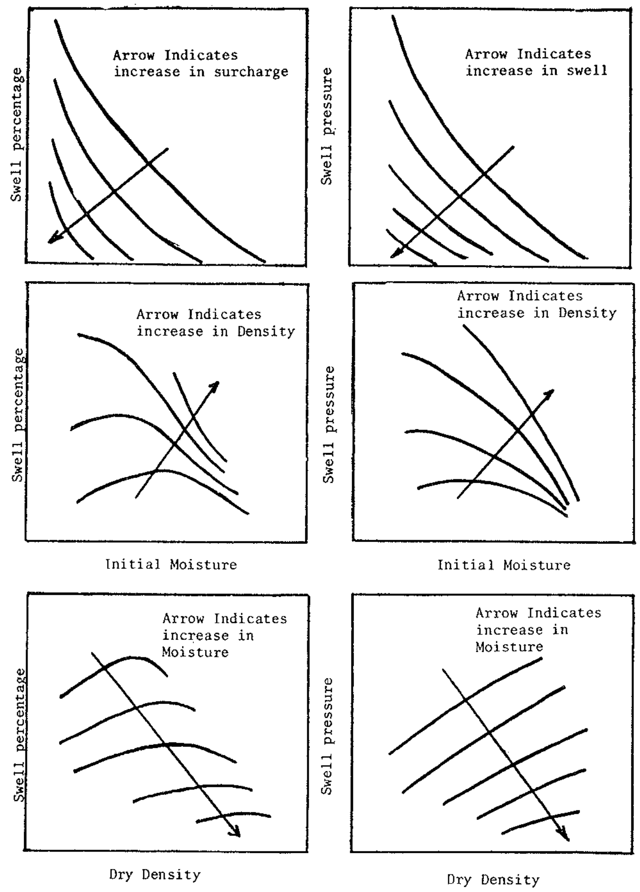
PI	Swell (%)	Degree of Swell
10	0.4- 1.5	Low
20	2.2- 3.8	Medium
30	5.7-12.2	High
40	11.8-25.0	Very high
50	20.1-42.6	Very high

Although the specific soil type is a significant factor in the magnitude of heave that can develop, the influence of initial moisture and density is of extreme importance. Figure 23 shows the effects of moisture content, dry density, and surcharge (vertical stress) on swell percentage and swell pressure. Perhaps the most significant conclusion to be drawn from the figure is that swell value can be greatly minimized if initial moisture is higher than optimum and if a low compactive effort (density) is used. This conclusion is contrary to the use of compaction for optimizing strength and minimizing soil densification due to traffic. Thus, if excessive heave is expected to be a significant distress mode, the usual compaction procedures may have to be modified.

In some design situations it may be necessary to construct the road on potentially expansive soil. In these cases there are several alternatives for the reduction or elimination of detrimental soil volume changes. One alternative is to remove and replace the existing soil. This approach depends on the economics of removal and replacement. Laboratory consolidometer-load expansion techniques can be used to estimate the depth of excavation that is needed. The depth of fill should be sufficient to constrain the swell pressures of the swelling soil that is not removed. Although partial excavation may not eliminate heave, the greatest heave tends to occur near the surface of the swelling subgrade soil. Thus, excavations of only several feet may be quite effective.

In Figure 23 it was shown that increased surcharge on an expansive soil will greatly reduce the swell, especially if used in combination with a high compaction moisture content. Therefore, the use of a pavement thickness that will cause an overburden stress equal to the presssure will eliminate

Figure 23. Schematic influence of initial soil moisture and density on high-volume characteristics.



swelling. Special studies are necessary to evaluate the necessary surcharge pressure. For example, studies on expansive soils in Israel indicate that 60 cm (24 in) is sufficient in most situations (26). However, on some highly expansive clays in Texas, computations have shown that pavement thickness to eliminate swell would have to be on the order of 950 cm (31 ft) (27).

In many situations, it may be economically feasible to use a chemical admixture to stabilize the swelling soil. Hydrated lime is generally the most effective stabilizer for reduction of swelling. Because of normal construction practices, this treatment is primarily used for the upper subgrade layers. More expensive and sophisticated deep stabilization techniques may be employed if found to be economically feasible.

By specifying compaction moisture contents above optimum the structure may be designed at a lower CBR strength value. Although pavement thickness requirements may be slightly greater for this reduced strength, the minimization or elimination of future soil heaving may be desirable.

One of the prerequisites for volume increase is the potential for movements of moisture into and out of the swelling soil. Thus, any method that will eliminate or minimize moisture flow will be beneficial. Use has been made of granular capillary cutoffs and impermeable membranes that are enveloped around the swelling soil to keep a uniform moisture content. The proper design and maintenance of sideditches and drainage are also very effective measures for attentuation of moisture flow.

CHAPTER 7

Road Surfacing Materials

The major portion of this chapter is concerned with granular materials that are used for road surfacing or as base and subbase layers for thin bituminous surface treatments. Characteristics of bituminous surface treatments are also discussed.

GRANULAR MATERIALS

Two important characteristics of granular materials are (a) gradation of the gravel-sand particles and (b) plasticity properties of the fines or silt-clay size particles. Fines may be considered to be those particles that pass through the No. 200 screen. Figure 24 illustrates granular materials at three levels of gradation.

Figure 24a shows a granular material that has no fine particles. This type of material is frequently referred to as an open or lean mix because numerous voids are present. The strength of this mix can only be controlled by the frictional component of shear strength that depends on aggregate-to-aggregate contact. For open mixes, the angularity of the granular particle has a marked effect on strength. Well-rounded gravel particles would be expected to have little strength whereas highly angular (e.g., crushed aggregate) particles would possess higher internal friction capabilities and greater strengths. Because of the rather high volume of voids present, permeability of such an aggregate gradation is high.

Figure 24. Various stages of soil aggregate gradations.

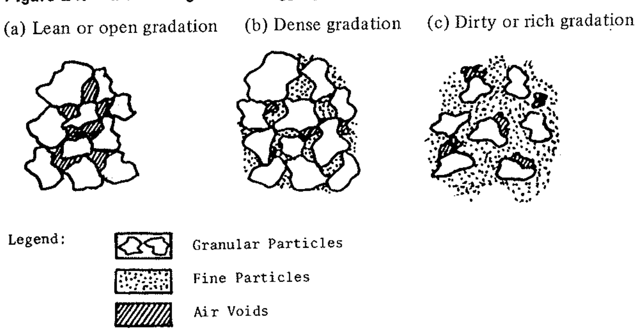
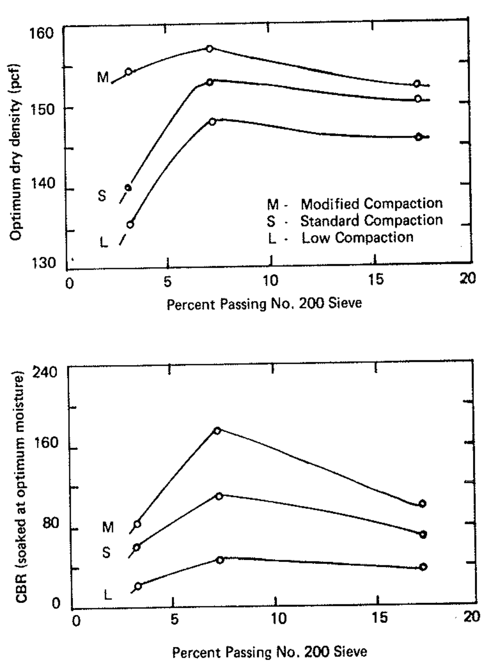


Figure 25. Influence of fines content on optimum density and CBR for unbound-graded crusher run base material.



As the percentage of fines is increased, the fine material begins to fill up the void spaces (Figure 24b). Because larger-size particles are not dislodged from each other, the increase of fines accomplishes several important functions. Since more solid particles are placed within the same total volume of material, the density of the material is increased. The shear strength of the material is also increased because of added frictional resistance and cohesion that is provided by the finer particles. Finally, the ability of water to flow through or permeate the material is drastically reduced from the lean or open case.

As fines are increased still more, the fines begin to displace the coarse particles from one another. When the fines are increased to the point as shown in Figure 24c, the granular particles float in a matrix of fine material. The material is then said to have a dirty or rich gradation.

In this case a slight density decrease will occur because fine particles have displaced coarser material. However, while change in density may not be very large, significant strength changes will occur. This is because the frictional 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. In general, the permeability of this gradation is even less than the dense gradation.

In summary, the distribution of sizes within a granular material plays a significant role on the density, strength, and permeability properties of the material. In Figure 25 the influence of percentage fines on the optimum dry density and CBR strength is shown for a graded crusher run material used as a base course. Each curve represents a given level of laboratory compactive effort.

The density relationship in Figure 25 shows that for a low percentage of fines the maximum possible density at optimum moisture is relatively low for all compactive efforts. With the addition of more fines, the density increases appreciably until a peak value occurs near 7% to 7.5% fines. Beyond this level, density decreases at a lesser rate. The values of 3%, 7%, and 17% fines correspond to cases (a), (b), and (c), respectively, in Figure 24. Regardless of the compactive level used, Figure 25 shows that the maximum possible density of the material occurs near a percentage of fines equal to 7.0% to 7.5%.

The distribution of the soaked CBR strength at optimum compaction moisture also indicates trends quite similar to those for density. For any compactive effort, the greatest CBR strength occurs when the percentage of fines corresponds to the densest possible case (i.e., at 7.0% to 7.5%). The figure also shows that strength is influenced significantly by level of compaction.

Wherever it is possible for the engineer to have some control over the gradation, the Power-Grading Law is a useful concept. This law is $p = (d/D_{max})^n \times 100$, where D_{max} = maximum aggregate size in the material, d = any particular grain size less than D_{max}, p = percentage of material whose grain size is less than d, and n = value of the power.

For example, if a granular material had a maximum aggregate size of D_{max} = 1.0 in (25.4 mm) and was graded to a power n = 0.6, the required percentage passing a No. 40 sieve (0.420 mm) would be computed as follows:

$$p = (0.420/25.4)^{0.6} \times 100 = 8.5\%$$

Figure 26 illustrates typical grading curves for various values of n when D_{max} is 1.0 in (25.4 mm). As the value of n is increased (e.g., n = 0.15 to n = 0.75), the percentage of fines decreases markedly, going from case (c) to case (a) of Figure 24. An important consequence of the power law is that the densest possible gradation and highest strength are attained when the value of n is approximately 0.50.

Figure 27 shows how the power law changes with D_{max} when n is held fixed at 0.45. The curves indicate that, as the maximum size of aggregate is increased, the required percentage of fines to achieve optimum density and strength is decreased.

The significance of extra compaction energy is clearly shown in Figure 28 for a number of granular materials. The large increase in strength from standard compaction effort to modified compaction effort is best illustrated by the fact that the average CBR at modified conditions is 2.2 times that for standard conditions.

While increased compactive effort may result in an increase in density of only several pounds per cubic foot, significant increases in strength will be achieved. Thus compaction and aggregate gradation are important factors in the assessment of local materials for use in construction.

The intrusion of fine-grained subgrade soils into the granular layers can alter the granular performance. The overall effect of this intrusion is to push granular base material into the subgrade on application of a vehicle load. The resulting surface rutting is no different than that which results from shear failure or densification of the granular material under traffic.

In general, intrusion can be prevented on open-type gradations by using a filter layer or sand blanket with a nominal particle size of about one-eighth of an inch. The thickness of this blanket course usually ranges between 2 and 4 in.

Under saturated conditions, the strength of granular material will be decreased by the presence of fines that have a high plasticity index. However, this decrease is usually insignificant up to fine (No. 40) percentages approximately equal to those required for optimum density conditions. Simple laboratory tests should be used on potential granular sources to evaluate whether plasticity levels will conform to required specification tolerances.

In practice, the acceptance or rejection of a particular material source depends on additional

Figure 26. Grain size distribution for power-graded materials.

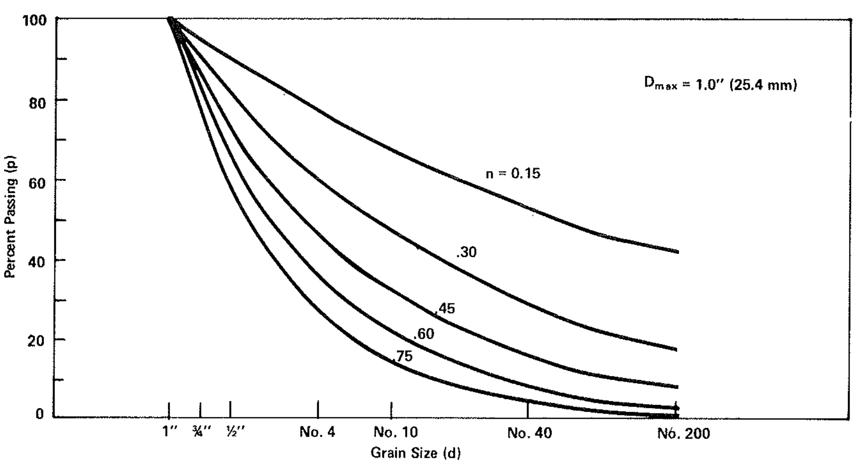
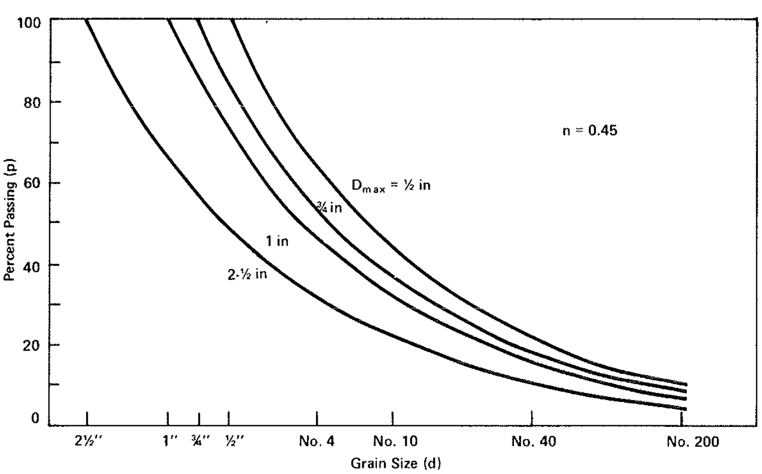


Figure 27. Effect of maximum aggregate size for dense-graded material.



factors beyond density, strength, and gradation. The material must be assessed for its soundness or ability to withstand repeated load actions. Soft aggregate is susceptible to drastic changes in gradation as it breaks up under loadings. This break-up or abrasion can occur under both construction compaction as well as under actual traffic loads. The most commonly used laboratory test to determine the relative degree of abrasion is the Los Angeles Abrasion Test (AASHO T-96). The larger the abrasion value obtained in the test, the softer the aggregate.

It is important for granular materials to remain durable in the face of environmental effects such as physical weathering. Efforts should be made to draw on previous experience with local materials.

In general, granular material requirements differ between (a) wearing surface use and (b) granular base or subbase use in bituminous-surfaced roads. Three major differences are as follows:

1. The maximum aggregate size for granular wearing surfaces is generally less than that normally used for base and subbase layers.
2. Granular wearing surface materials require a greater percentage of fine material (passing No. 200) than a base or subbase layer.
3. The fine material used in granular wearing surface layers should possess greater plasticity and larger liquid limits than for material used in base or subbase layers.

These differences are related to the different distress mechanisms that are associated with different uses. There are more potential distress mechanisms for granular surface layers than for granular materials used as base or subbase layers. While strength is an important consideration for both uses, the granular surface layer must also be evaluated for its resistance to the direct abrasive action of traffic, ravelling, corrugation, and pothole development.

Strength is the main criterion for evaluating granular material for use as base or subbase layers in a bituminous-surfaced road. For this use, the granular material should meet the gradation and plasticity requirements that have been discussed.

When a granular layer is used as a wearing surface, the confining stress near the top of the layer is relatively small. As a result, if the material has no fines it may have a very low shear strength when a load is applied. The cohesive-resistance of fines may be the only source of strength for resisting shear stresses. The development of corrugations is primarily due to the rhythmical bouncing of vehicles on the road surface. Corrugations also are more frequent in cohesionless sandy-gravelly materials. Thus control of the sand percentages in the material is important. It has been suggested (28) that coarse and fine sand content should be less than 55% of the total material to minimize corrugations. In addition, a minimum liquid limit of 20% is suggested.

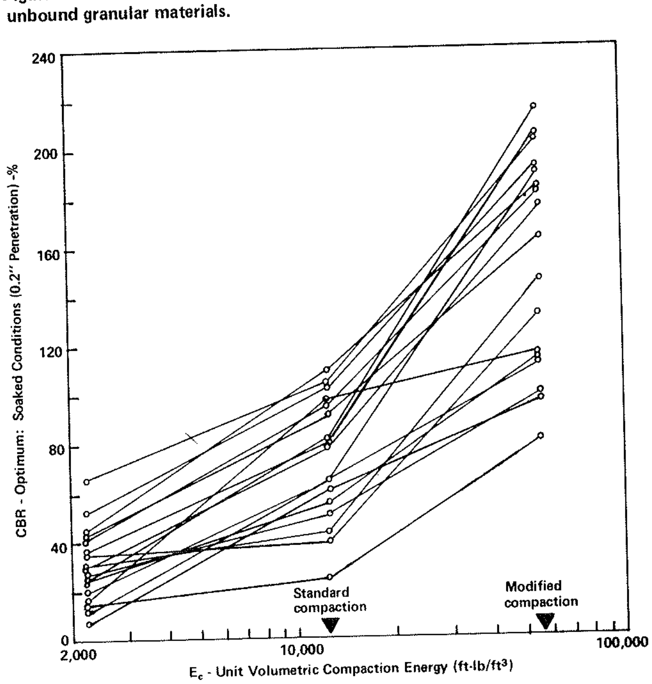
Many desirable properties that minimize corrugations will also prevent ravelling. Ravelling may be minimized by keeping the road surface in a moist condition. Minimum percentage clay contents of 6%, plasticity indexes greater than 6%, and liquid limits greater than 20% have been proposed as desirable.

The generation of fines, and their subsequent removal by vehicular traffic as dust, is obviously aggravated by the climatic environment at the road site. Very dry conditions coupled with large percentages of fine materials in the material will cause serious dusting and associated traffic hazard problems. A liquid limit greater than 20% and a sand content greater than 30% will help to minimize this problem (28).

The development of potholes is frequently associated with material properties and areas where water is allowed to stand on the road surface. From a materials viewpoint, liquid limits less than 35% and sand contents greater than 30% have been noted as desirable for the minimization of pothole development.

It is apparent that maximum performance of a granular wearing surface requires minimum and maximum values for the plasticity index. The allowable range is normally greater than that allowed for granular bases in bituminous-surfaced roads. In addition, the relative percentage of sand size material is more important to control in granular wearing surfaces than in base or subbase layers.

Figure 28. Influence of compaction energy on optimum soaked CBR for unbound granular materials.



GRADATION REQUIREMENTS

Table 17 and Table 18 summarize typical grading requirements for granular material used as granular surface layers. Table 19 and Table 20 illustrate grading requirements for base or subbase usage.

For granular wearing surfaces, the maximum aggregate size is generally 3/4 in (19 mm) to 1 in (25.4 mm), with the 3/4-in value preferred. This is in contrast to the much larger allowable maximum aggregate sizes for base or subbase use.

Regardless of the proposed use, the required percentage of fines increases with a decrease in the

Table 17. Grading requirements for granular surface course (3).

	AASHTO Grading Requirement			
Sieve Size	C	D	E	F
1 in	100	100	100	100
3/8 in	50-85	60-100	-	-
No. 4	35-65	50-85	55-100	70-100
No. 10	25-50	40-70	40-100	55-100
No. 40	15-30	25-45	20-50	30-70
No. 200[a]	8-15	10-25	8-20	8-25

[a]Minimum 8 percent pass No. 200.

maximum aggregate size. This is in general agreement with the principles that have been discussed for the power grading law.

Requirements that have been given by two agencies for the plasticity of surface course materials are listed in Table 21. A similar compilation is given in Table 22 for granular material used as base or subbase layers in a bituminous-surfaced road.

In general, the maximum allowable values of both PI and LL are greater for granular materials when used as a wearing surface than when used as a base or subbase course.

For granular wearing surface materials, Table 21 shows minimum PI values greater than zero. On the other hand PI = 0 is an allowable condition in Table 22 for base or subbase material. Thus it is necessary for the wearing surface material to possess some degree of binding ability or cohesive action that resists the abrasive action of traffic.

In general, there is a regional climatic influence on the allowable maximum PI for both functional uses of the granular material. As the environment becomes drier a larger maximum PI value is allowed. Although not specifically shown in either table, traffic level also affects the allowable plasticity requirements. As the design traffic level is increased, the allowable LL(max) and PI(max) should be decreased.

Table 18. Grading requirements for granular surface courses (29).

	Maximum Aggregate Size (in)		
Sieve Size	3/4	3/8	3/16
3/4 in	100	100	100
3/8 in	80-100	100	100
3/16 in	60-85	80-100	100
No. 7	45-70	50-80	80-100
No. 36	25-45	25-45	30-60
No. 200	10-25	10-25	10-25

Table 19. Grading requirements for granular base or subbase (3).

	AASHTO Grading Requirement					
Sieve Size	A	B	C	D	E	F
2 in	100	100	100	100	100	100
1 in	-	75-95	100	100	100	100
3/8 in	30-65	40-75	50-85	60-100	-	-
No. 4	25-55	30-60	35-65	50-85	55-100	70-100
No. 10	15-40	20-45	25-50	40-70	40-100	55-100
No. 40	8-20	15-30	15-30	25-45	20-50	30-70
No. 200	2-8	5-20	5-15	10-25	6-20	8-25

Table 20. Grading requirements for granular base or subbase (22, 29).

	Maximum Aggregate Size (in)				
Sieve Size	3	1.5	3/4	3/8	3/16
3 in	100	100	100	100	100
1.5 in	80-100	100	100	100	100
3/4 in	60-80	80-100	100	100	100
3/8 in	30-65	40-75	80-100	100	100
3/16 in	25-55	30-60	50-85	80-100	100
No. 7	20-45	25-50	35-70	50-80	80-100
No. 36	10-30	15-30	15-35	25-50	25-55
No. 200	5-15	5-15	5-15	10-25	10-25

Because strength is a major requirement, the higher-quality material should be placed as near to the surface as possible. Lower-quality materials are acceptable in flexible pavement structures as subbase material. Thus, higher PI and LL values are allowed for granular materials when used as subbase than when used as base courses.

Minimum strength (CBR) requirements for granular base or subbase materials are shown in Table 23. Minimum strength requirements are not given for granular wearing surface layers. Although the strength of wearing course materials is important, specifications on gradation and plasticity properties are more relevant to good performance of the wearing course material. In general, a granular wearing course should have the highest available strength among those materials that meet the grading and plasticity specifications.

Abrasion requirements for coarse aggregate are generally based on the Los Angeles Abrasion Test after 500 revolutions. In general, maximum allowable LA values of 50% are specified for typical base or subbase use, although values of 60% may be allowed on low-volume facilities that have a bituminous surface course. When granular material is used as a wearing surface, the material should be less abrasive because of its direct contact with the vehicular traffic. Under these conditions, typical LA loss ranges are 40% to 50%.

Further information on granular materials is contained in Compendium 7 (see inside back cover).

AGGREGATE BLENDING

It is often not possible to locate a specific source of granular material that satisfies the required specifictions and that is economical to use. In this case, surveys should be made to learn if materials from several different local sources can be blended so that their combined properties meet desirable specifications. In areas where sandy deposits predominate, the search can be made for fine soil deposits (silt or clay) that would provide the required fines and meet plasticity requirements. In areas where clays predominate, surface deposits that have appreciable amounts of angular particles can frequently be found and combined with the fine soils.

Blending involves separate checks to see if both gradation and plasticity requirements are met. Figure 29 may be used to check whether or not two different materials can be combined to meet gradation specifications.

As an illustration of the use of this diagram, suppose that the desired gradation for a granular surface course material is given by gravel, 55-70%; sand, 25-40%; and fines, 5-15%. In Figure 29 this gradation specification range is shown as a shaded region. Any specific gradation falling within this area will satisfy the desired specification limits.

Table 21. Plasticity requirements for granular surface courses.

Agency	Plasticity Properties			
1. AASHTO (3)		LL = 35_{max}	PI = 4 - 9	
2. TRRL (30)		LL_{max}	PI	Linear Shrinkage
	Moist temperature/wet tropics	35	4-9	2.5-5
	Seasonally wet tropics	45	6-20	4-10
	Arid/semi-dry	55	15-30	8-15

For this example problem, assume that a gravel source (A) and two sand sources (B and C) have been located as candidate sources for the granular surface material. From a laboratory grain-size distribution study, the individual gradations of these three material types are found to be

Surface Material	Source A	Source B	Source C
Percentage Gravel	65	12	15
Percentage Sand	20	63	80
Percentage Fines	15	25	5

The respective gradations are plotted in Figure 29 as points for sources A, B, and C. It can be seen that all three points fall outside the shaded area that defines the gradation specification. However, if the line that connects any two of the source points passes through the shaded area, then specifications can be met by blending the materials from these sources. Since the dotted line that connects Sources A and C passes through the shaded region, the specifications can be met by blending materials from Sources A and C. The dotted line from Source A to Source B does not pass through the shaded area, so specifications cannot be met by blending materials from these sources.

The following steps are necessary to determine the relative amounts of material to be blended from Sources A and C.

1. Select a convenient point X that lies on the dotted line joining Sources A and C and that lies within the shaded specification area. This point corresponds to Blend X that will be made with materials from Sources A and C.
2. Use any arbitrary measuring scale to determine the distances from Source A to Source C, from Source A to Blend X, and from Source C to Blend X. For Figure 29 these distances are
   Source A to Source C: AC = 35 units
   Source A to Blend X: AX = 6 units
   Source C to Blend X: CX = 29 units
3. Find the percentage of Source A material and Source C material to be used:
   % Source A material = (CX/AC) 100 = (29/35) 100 = 82.9%
   % Source C material = (AX/AC) 100 = (6/35)100 = 17.1%
   Thus for every 100 kg of the blend, about 83 kg will come from Source A and about 17 kg from Source C.
4. Gradation for the blended material can be calculated as follows:
   % Gravel = (82.9% Source A x 65% Gravel) + (17.1% Source C x 15% Gravel) = 56.5% Gravel
   % Sand = (82.9% Source A x 20% Sand) + (17.1% Source C x 80% Sand) = 30.2% Gravel
   % Fines = (82.9% Source A x 15% Fines) + (17.1% Source C x 5% Fines) = 15.3% Fines.

The foregoing procedures can be used to design an aggregate blend that will meet specifications for both granular surface courses and future base course specifications. This is accomplished by superimposing both specification gradations on the trian-

Table 22. Plasticity requirements for granular base or subbase.

Agency	Plasticity Properties		
1. AASHTO (3)	LL = 25_{max}	PI = 6_{max}	
2. TRRL (30)	LL = 25_{max}	PI = 6_{max}[a]	Linear shrinkage = 4_{max}

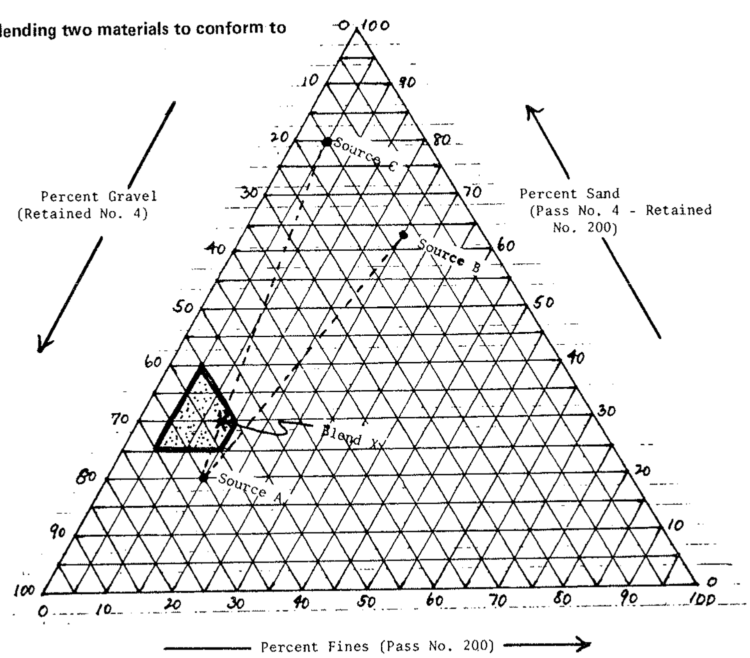
[a]If dry area (250 mm or less rainfall); PI = 12_{max}.

Table 23. Strength (CBR) requirements for granular base or subbase.

	Minimum CBR Value	
Agency	Base Course	Subbase
1. TRRL (22, 30)	80	25
2. Brazil: U.S.A.I.D. Study (31)	50-60[a]	25-40[a]
3. South Africa (28)	50-60[b] 60-70[b]	35-45[b]

[a]Use lower value if design life is for less than 250 000 equivalent 18 000-lb single-axle loads.
[b]Use lower values if subgrade CBR >25.

Figure 29. Triangular gradation chart for blending two materials to conform to gradation specifications.



gular chart. The area of overlap between the two specification ranges therefore represents a gradation range that satisfies both uses of the granular material. This approach should be used when there is a possibility of future upgrading that will use the granular base course for a bituminous surface course.

A final consideration is that the plasticity characteristics of the blended material should also comply with specifications. This can be accomplished by running liquid limit and plastic limit tests on the blended material. If plasticity requirements are not met, adjustments in the blend percentages may be necessary.

LATERITIC GRAVELS

Lateritic soils and lateritic gravels are widespread and frequently occurring in all tropical areas of the world. Because of its somewhat peculiar engineering behavior as a road building material, lateritic materials are treated separately.

Lateritic gravels are frequently used for subbase, base, and even surfacing material on unpaved granular surfaced roads. Comprehensive studies have been made of laterites and other problem soils of the tropics (31, 32).

Laterite is defined as a highly weathered, red subsoil, or material rich in secondary oxides of iron, aluminum, or both. It is nearly void of bases and primary silicates and may contain large amounts of quartz and kaolinite. It develops in a tropical or forested warm to temperate climate and is a residual or end product of weathering. Laterite is capable of hardening after exposure or on being subjected to wetting and drying. When it forms a hardened crust or layer, it is frequently called ironstone.

Lateritic gravel is composed of nodules or concretions in an unconsolidated matrix. The concretions are mainly accumulations of iron or aluminum oxide around some nucleus such as a quartz grain. When appreciable quantities of quartz are present in the parent bedrock, the weathering profile, including the lateritic gravel horizon, will contain quartz particles.

In Africa, regional pedologic mapping has been based on the French classification system. In this system, red tropical soils are grouped into one of three units: Ferruginous, Ferrallitic, and Ferrisols. The Ferruginous category includes soils developed in low rainfall areas (less than 1830 mm) with pronounced dry seasons. Ferralitic soils are developed in humid areas (greater than 1500 mm rainfall) with dense vegetation. In the last category, Ferrisols develop under intermediate to high rainfall conditions (1250-2750 mm) but natural profile development is hindered by high erosion capability.

In South America, the FAO-UNESCO system is used. Most of the red tropical soils fall under the Ferralsol category, with Arenosols, Acrisols, and Luvisols being common and Nitosols and Cambisols present but restricted in distribution. Attempts have been made to correlate the French and FAO systems. While this has not led to complete agreement, some evidence exists that the following groups are somewhat similar:

1. Ferruginous-Luvisols
2. Ferralitic-Ferralsols, Arenosols, Acrisols
3. Ferrisols-Cambisols, Nitosols.

As with any granular deposit, a wide range in quality of lateritic gravels (ironstones and concretionary gravels) may occur within and between pit sources. It is therefore difficult to generalize material quality by any particular pedologic classification. Each specific material source must be evaluated for its adequacy as subbase, base, or granular surfacing individually.

One of the more significant findings of laterite studies (31, 32) is the fact that several commonly used methods to evaluate material durability are either too severe or lack good correlation with field performance. This finding was especially true for the Los Angeles abrasion test method and generally accepted specifications. The test method that showed the most promise was the Slake Durability test, commonly used as a rock test for sedimentary rocks with significant clay contents (e.g., shales, siltstones, etc.). The specific details of the test procedure for lateritic gravel evaluation are found in reference (31). The Slake Durability Index, for good performing lateritic gravels is a minimum of 97. Materials with values below 94 demonstrated poor performance. For lateritic gravels used as granular surfacing layers, a minimum value of 95 is recommended.

Based on studies in South America and Africa, several general relationships between various soil and compaction properties have been established. These correlations are shown in Table 24 and can be used as a first-order estimate of probable material quality for red tropical soil materials.

Figure 30 shows the general range of soaked CBR values for red tropical soils of South America as a function of the AASHO soil classification system. From this figure, it can be observed that A-2 soils (A-2-4 and A-2-6) generally exhibit soaked CBR values from about 65 to 150. This, of course, is within the satisfactory strength requirements for base layer quality. Subbase type materials (CBR range 25 to 50+) are shown to correlate with lateritic material with A-4 and A-5 classifications.

Another important characteristic of lateritic gravels is illustrated in Figure 31. This diagram

Table 24. Summary of red tropical soil relationships (31, 32).

	South America		Africa	
Item	Relationship	Pedologic Group	Relationship	Pedologic Group
1. Atterberg limits	PI = 0.45LL - 3.50	Ferrasols	PI = 0.71LL - 8.50	Ferruginous
	PI = 0.47LL - 3.80	Acrisols	PI = 0.57LL - 3.62	Ferrallitic
	PI = 0.82LL - 13.95	Arenosols	PI = 0.50LL - 1.50	Ferrisols
	PI = 0.75LL - 12.70	Luvisols		
	PI = 0.23LL - 7.96	Nitosols		
2. Compaction properties	OMC = 0.61, PL = 84 or OMC = 0.34(2μ + 6.16)		MDD = 160, = 2.78 OMC	

Note: PI = plasticity index, LL = liquid limit, PL = plastic limit, OMC = optimum (max) moisture content, MDD = maximum dry density (modified AASHTO), and μ = clay context.

Figure 30. Correlation of AASHO soil classification with CBR for South American red tropical soils (31).

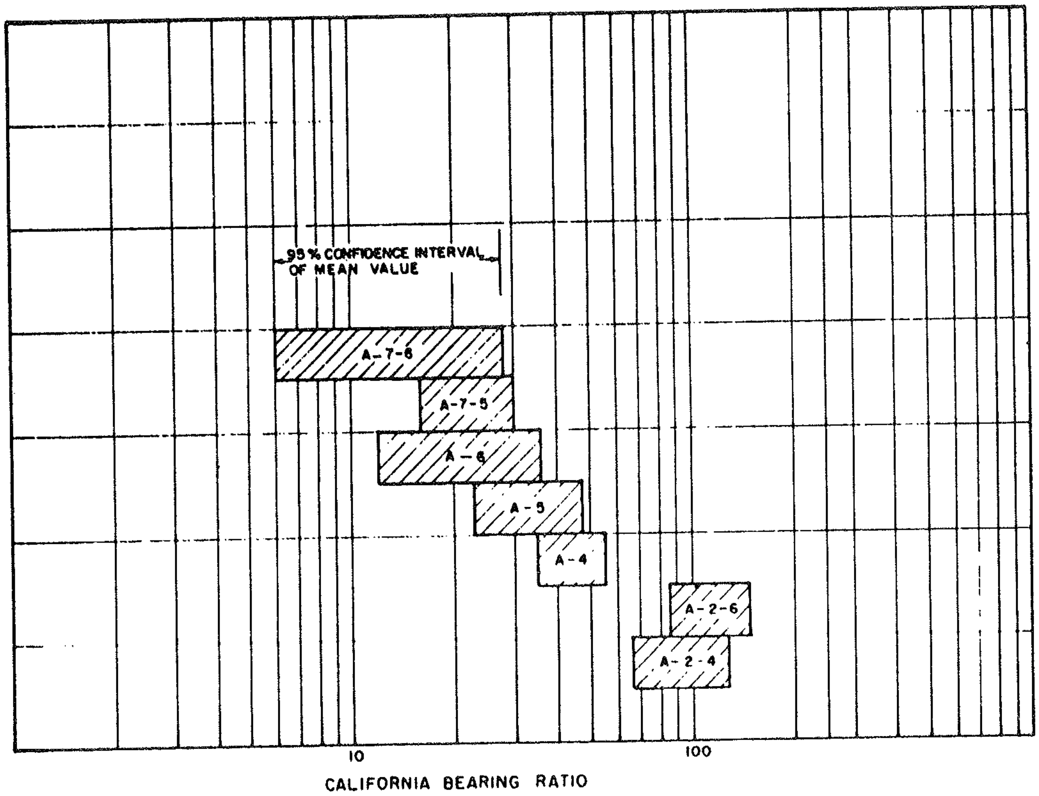


Figure 31. Relationship of CBR to granulomatric modulus (31).

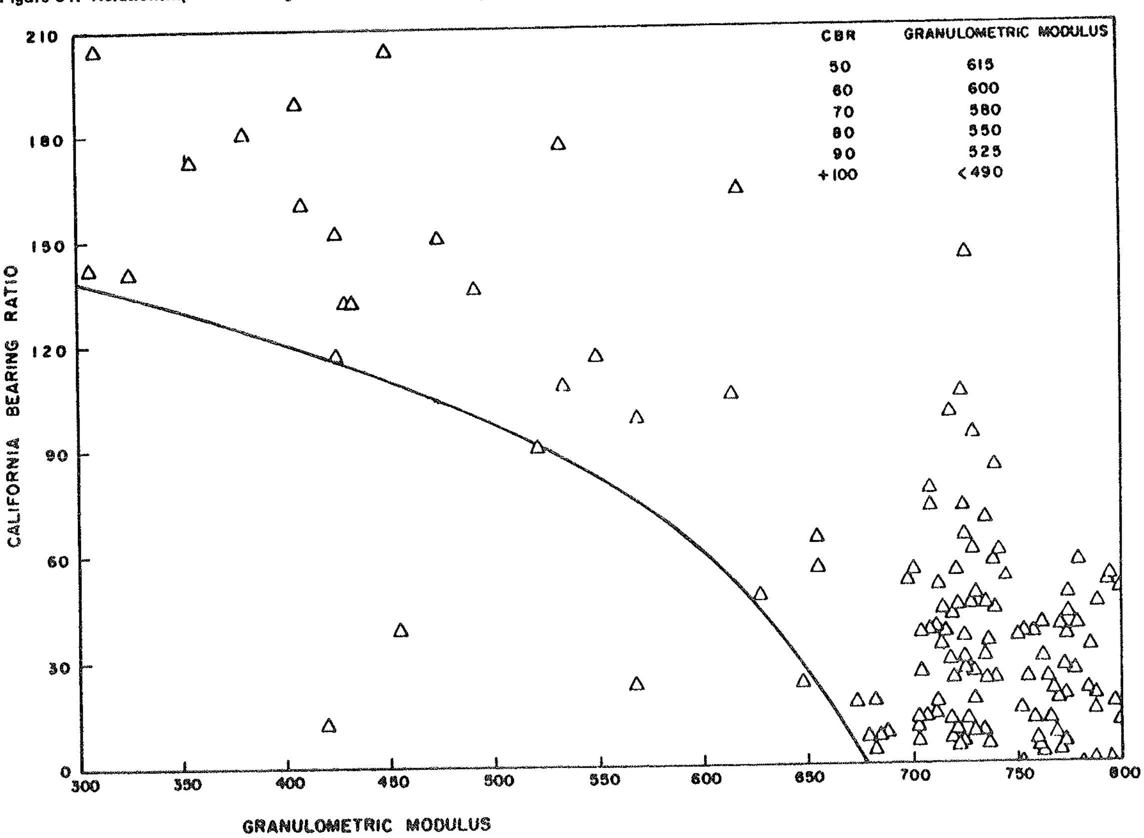


Table 25. Illustrative granulometric modulus computations.

Sieve Size	Coarser Material (% passing)	Finer Material (% passing)
1 in	100.0	100.0
3/4 in	95.0	100.0
1/2 in	71.0	100.0
3/8 in	63.0	100.0
No. 4	52.0	100.0
No. 10	44.0	98.0
No. 40	29.0	92.0
No. 200	13.0	61.0

Notes: Granulometric modulus = 467 and 751, respectively.
Estimated minimum CBR = 100+ and = 0+, respectively.

shows the relationship between the soaked CBR value and the granulometric modulus of the material. The curve shown in Figure 31 represents the minimum probable CBR of the material as a function of the granulometric modulus. The granulometric modulus is defined to be the sum of the percentages that pass the following sieves: 1 in, 3/4 in, 1/2 in, 3/8 in, No. 4, No. 10, No. 40, and No. 200. Table 25 shows illustrative computations of the granulometric modulus for a coarse-grained and a fine-grained material. As noted, the minimum CBR for the coarser material (GM = 467) is 100+, while the finer material may be simply noted to have a CBR value that is greater than zero.

SURFACE TREATMENTS

A bituminous surface treatment provides a low-cost and all-weather surfacing material that can increase performance appreciably for light to medium traffic conditions. Surface treatments are generally less than 1 in (25 mm) in thickness. Thus the bituminous layer adds little direct strength increase to the pavement structure. However, the indirect benefits of this type of surface are increased life and performance.

A fundamental design concept is that the granular structure must by itself have sufficient load-bearing capacity when surface treatments are used. For this reason the material quality requirements for base or subbase courses must be met.

The probable service life of surface treatments varies considerably. Ranges of 5 to 10 years of major maintenance free service life are generally obtained. The successful performance of surface treatments depends heavily on climate and construction control as well as on proper selection of materials.

The term surface treatment is applied to a wide variety of bituminous applications that include the types listed below.

1. Spray and chip coats
   a. Single-surface treatment
   b. Multiple-surface treatment
2. Mixed-in-place surface treatment
3. Plant mix (seal) surface treatment
4. Seal coats
   a. Fog seal
   b. Slurry seal
   c. Sand seal
5. Bitumen spray coats
   a. Dust pallative
   b. Prime coat
   c. Tack coat

Plant mix seals or surface treatments are generally high type and high-quality layers that are used primarily to improve or upgrade the skid resistance of existing asphaltic surfaces. The open-graded friction course is typical of this category. Seal coat surface treatments are used on existing asphaltic surfaces to improve the surface texture, to seal small cracks, and to improve the skid resistance of the pavement. Because both of these categories of surface treatment deal with maintenance or rehabilitation of existing bituminous surfaces, they are beyond the scope of this synthesis.

Bitumen spray coats involve the spraying of low-viscosity bitumen on an existing layer without the addition of any aggregate. For example, slow curing or slow setting liquid asphalts are frequently sprayed on existing granular-surfaced roads as a temporary means of minimizing dust problems associated with traffic. The function of a prime coat is to serve as a binding layer between a granular base material and an asphaltic layer that is to be placed on top of the granular material. Penetration of the bitumen into the granular base is highly desirable. A tack coat is intended to bind two bituminous layers together.

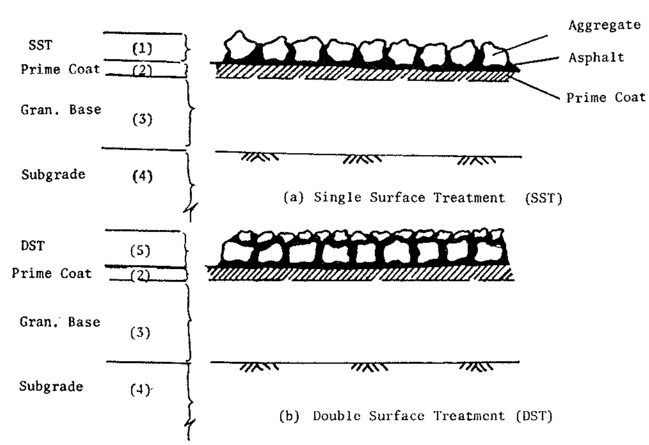
The spray and chip coat and the mixed-in-place surface treatment are two major construction techniques that are used for low-cost wearing courses. Spray and chip coats can be either single or multiple, depending on the number of layers that are applied. Multiple surface treatments can all be applied during initial construction or can be applied at successive time intervals after construction. The spray and chip treatment is characterized by (a) the application of bituminous material to the existing surface and (b) immediate application of a single size aggregate. The aggregate becomes seated in the asphalt layer and thus provides a wearing surface for the traffic.

Figure 32 is a schematic diagram of single and double surface treatments. A prime coat is usually necessary to bind the surface treatment to the unbound granular base layer. It is important to note that in surface treatment spray and seal work the asphalt is placed first and then the aggregate is spread on top of the asphalt layer. It is necessary for aggregate gradings to be relatively one-sized so that aggregate-tire contact is maintained. In the multiple surface treatment pavement, each successive layer normally employs a nominal aggregate size that is approximately one half of the lower layer aggregate size. This process leads to seating of aggregates within each successive layer so that denser mix of asphalt and aggregate is attained. A well-designed surface treatment will generally have about 70% of the available void volume filled with asphalt.

The use of surface treatments provides several advantages that are listed below.

1. Surface-treated roads provide a low-cost

Figure 32. Schematic diagram of surface-treated pavements.



alternative for an all-weather road surface.

2. Surface treatments act as an effective seal or barrier against the detrimental effects of surface water.
3. Surface-treated roads allow some reduction in pavement cross-section requirements and may provide economic savings.
4. Multiple surface treatments are conducive to stage construction.
5. The use of a bituminous layer will greatly reduce or minimize the severity of distress modes such as aggregate loss or dusting.
6. If properly constructed, surface-treated roads will reduce the level of road maintenance relative to granular-surfaced roads.

The foregoing advantages should be balanced against the potential disadvantages or limitations listed below.

1. Surface-treated roads require a more careful design analysis to ensure that adequate load-bearing capacity exists.
2. Maintenance of these roads may be more expensive than for granular surfaced roads.
3. Specialized equipment and skilled operators are necessary for proper construction of surface-treated roads.
4. Surface treatments generally require higher-quality aggregate material in both the surface-treated layer and the unbound granular base course.

Before application of the prime coat, it is necessary that the surface be broomed to eliminate loose material on the existing surface. Weak areas in the granular base should be replaced with new base material before the surface treatment is applied. In cases where the existing granular material is highly distressed, it may be advisable to completely scarify and recompact the granular material before the bituminous construction proceeds.

Application of the asphalt layer is a highly important construction step. The two most important considerations are to ensure a uniform spray application quantity of the asphalt and to ensure that the proper viscosity is attained during the spray process. These requirements can only be met by a truck-mounted asphalt distributor with heating capabilities. The recommended viscosity range for spraying is 20-120 centistokes (approximately 10 to 60 Saybolt Furol seconds). The spray temperature should be based on the specific temperature-viscosity relation of the asphalt used (33). Care must be taken to ensure that spray temperatures are within the safe range relative to the specific flash point of the material used.

The spreading of the aggregate must be accomplished in a uniform manner, generally by mechanical spreaders. In order to achieve proper binding of the aggregate within the asphalt layer that has been sprayed, aggregate must be placed on the asphalt in less than 1 min.

Immediately after the aggregate has been spread, it is desirable to roll the aggregate with pneumatic tire rollers. Steel rollers should be avoided because high-contact stresses may fracture and degrade the aggregate. Within several days light brooming may be desirable to remove loose or excess aggregate from the surface.

The best results for surface treatment will occur in hot and dry conditions. Bonding between the bituminous and granular materials is greatly improved with warmer air temperature.

Aggregates used as surfacing material must be of high quality. Desirable characteristics of surface treatment aggregates are listed below.

1. High abrasion resistance (low Los Angeles abrasion values)
2. Crushed or fractured faces
3. Cubical in shape
4. Relatively one-sided
5. Clean or free of deleterious fine-grained material
6. Chemically compatible with the bituminous material.

As a general rule, typical LA abrasion values for coarse aggregate retained on a No. 8 sieve should be less than 40 to ensure adequate abrasion resistance. Use of relatively one-sized aggregate that is primarily cubical in shape ensures a relatively constant plane surface of exposed aggregate. Flat and elongated particles should not be used.

Because of the direct contact of the aggregate with the vehicle tire, the skid qualities of the pavement are directly influenced by surface texture. Crushed particles (stone, gravel, or slag) increase friction capabilities and provide greater strength by interlock of the surfacing layer. Typical specifications require that greater than 60% of the aggregate particles (by weight) possess two or more fractured surfaces.

Table 26 illustrates typical one-sized aggregate

Table 26. Gradation requirements for one-size aggregates (34).

Size Designation	Nominal Size Square Openings[a]	Amounts Finer Than Each Laboratory Sieve (Square Opening), Percentage by Weight							
		1	3/4	1/2	3/8	No. 3	No. 4	No. 8	No. 200
A	3/4 to 1/2	100	85 to 100	0 to 20	0 to 7			0 to 1	0 to 0.5
B	1/2 to 3/8		100	85 to 100	0 to 30	0 to 7		0 to 1	0 to 0.5
C	3/8 to No. 3			100	85 to 100	0 to 25	0 to 10	0 to 1	0 to 0.5

[a]In inches, except where otherwise indicated. Numbered sieves are those of the U.S. Standard Sieve Series.

Table 27. Gradation requirements for graded aggregates (34).

Size Number	Nominal Size Square Openings[a]	Amounts Finer Than Each Laboratory Sieve (Square Opening), Percentage by Weight							
		1	3/4	1/2	3/8	No. 4	No. 8	No. 16	No. 50
6	3/4 to 3/8	100	90 to 100	20 to 55	0 to 15	0 to 5			
7	1/2 to No. 4		100	90 to 100	40 to 70	0 to 15	0 to 5		
8	3/8 to No. 8			100	85 to 100	10 to 30	0 to 10	0 to 5	
9	No. 4 to No. 16				100	85 to 100	10 to 40	0 to 10	0 to 5

[a]In inches except where otherwise indicated. Numbered sieves are those of the U.S. Standard Sieve Series.

Table 28. Types of asphalt for surface treatments (33).

	Asphalt Cements		Liquid Asphalts								
			Rapid Curing (RC)				Medium Curing (MC)				
Types of Construction	120/150	200/300	70	250	800	3000	30	70	250	800	3000
Surface treatments with cover aggregates	X	X	X	X	X	X			X	X	X
Seal coats	X	X	X	X	X	X		X	X	X	X
Slurry seal											
Fog seal											
Tack coat			X								
Prime			X	X			X	X	X		
Dust laying							X	X			

[a]SS grades can be used when sand is used for cover. [b]Water diluted.

Table 29. Quantities of asphalt and aggregate for single-surface treatments and seal coats (33).

Line No.	Size of Aggregate	Size No.	Pounds of Aggregate per Square Yard[a,b]	Gallons of Asphalt per Square Yard[a,c]	Hot Weather (80° F +) Hard Aggregate	Hot Weather (80° F +) Absorbent Aggregate	Cool Weather (up to 80° F) Hard Aggregate	Cool Weather (up to 80° F) Absorbent Aggregate
1	3/4 to 3/8 in	6	40-50	0.40-0.50	120-150 RC3000, RS2, CRS-1, CRS-2	RC3000, RS2, RS-2K, RS-3K CRS-1, CRS-2	RC800, RS2, RS-2K, RS-3K, CRS-1, CRS-2	RC800, RS2, RS-2K, RS-3K, CRS-1, CRS-2
2	1/2 in to No. 4	7	25-30	0.25-0.30	200-300[d] RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, RS-2K, RS-3K, CRS-1, CRS-2	RC250, 800, RS1, RS2, RS-2K, RS-3K, CRS-1, CRS-2	RC250, 800, RS1, RS2, RS-2K, RS-3K, CRS-1, CRS-2
3	3/8 in to No. 8	8	15-20	0.15-0.20	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2
4	1/4 in to No. 8	9	10-15	0.10-0.15	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2	RC250, 800, RS1, RS2, CRS-1, CRS-2
5	Sand		10-15	0.10-0.15	RC250, 800, RS1, RS2, CRS-1, CRS-2, SS-1, CSS-1	RC250, 800, RS1, RS2, CRS-1, CRS-2, SS-1, CSS-1	RC250, 800, RS1, CRS-1, CRS-2, SS-1, CSS-1	RC250, 800, RS1, CRS-1, CRS-2, SS-1, CSS-1

Notes: These quantities and types of materials may be varied according to local conditions and experience.
Single-surface treatments. The maximum size aggregate should not be more than 1/2 in. Use line 2. For lighter surface treatments, use line 3 or 4: however, lines 3 and 4 are more for light seal coats. For sand seals use line 5.
Double-surface treatments. The maximum size can be up to 3/4 in. First course, use line 1; second course, use line 3 or 4. For lighter double-surface treatments, use for first course, line 2; for second course, line 3 or 4.
Triple-surface treatments. The maximum size aggregate is usually 1/4 in. The following is recommended; first course, line 1; second course, line 2; third course, line 3 or 4. For most situations, the best probably is lines 1, 2, and 4 for the three courses.

[a]The lower application rates of asphalt shown in the table should be used for aggregate having gradings on the fine side of the limits specified. The higher application rates should be used for aggregate having gradings on the coarse side of the limits specified.
[b]The weight of aggregate shown in the table is based on aggregate with a specific gravity of 2.65. In case the specific gravity of the aggregate used is less than 2.55 or more than 2.75, the amount shown in the table above should be multiplied by the ratio that the bulk specific gravity of the aggregate used bears to 2.65.
[c]Under certain conditions, MC liquid asphalts may be used satisfactorily.
[d]In some areas, persistent difficulty in retaining aggregate has been experienced with 200-300 penetration asphalt cements.

gradations used in surface treatments. The maximum aggregate size has a major effect on ride smoothness and ride noise. Experience has shown that a 1/2-in maximum aggregate is best for all-around performance. Selection of maximum sizes should be based on the anticipated number of surface treatments that may be used in the life of the pavement. When multiple surface treatments are used, the nominal size of each aggregate layer is generally reduced by one-half so that the aggregates of each succeeding layer will be well seated or nested in the lower layer.

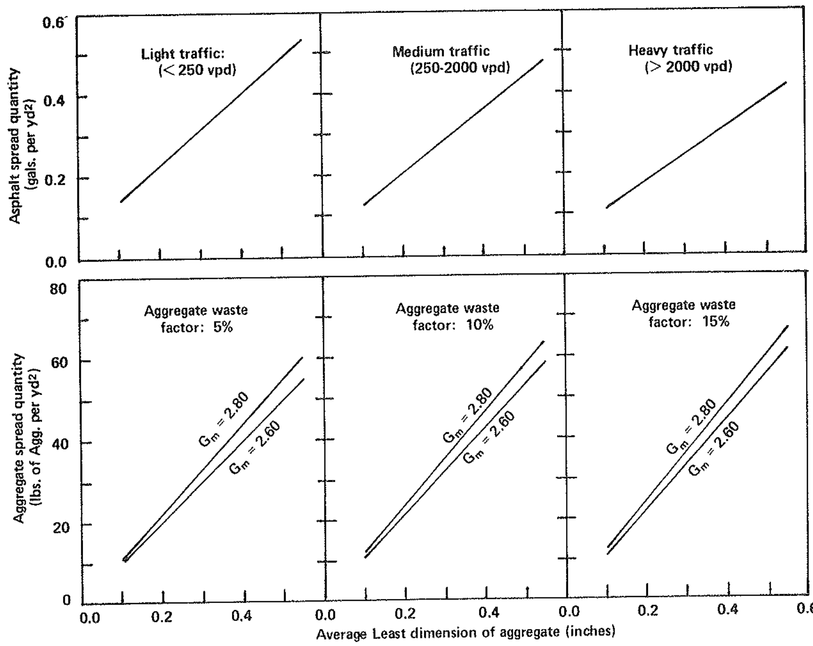
Economic availability of one-sized aggregates may be impossible. Graded aggregates have been successfully used provided that the gradations are strictly controlled in accordance with grading specifications shown in Table 27. For single surface treatments the maximum aggregate size should be 1/2 in (No. 7 size). If very light traffic conditions are anticipated, maximum sizes of 3/8 in or 1/4 in may be used (size No. 8 or No. 9). For double surface treatments, aggregate size No. 6 (first layer) and No. 8 (second layer) are used for light to medium traffic, while No. 7 and No. 9 can be used for lighter traffic.

Clean and chemically compatible aggregates are necessary for strong bonds between the aggregate and bituminous materials. Unlike high-quality bituminous plant mixes, surface treatments derive their strength from the bond rather than through internal friction mechanisms. Lack of adhesion of bituminous material to the aggregate will lead to premature ejection of aggregate particles from the bituminous layer. Washing of the aggregate to free the material from fine silt or clay coatings may be required. Aggregate that is highly acidic or silaceous may not give good coating and adhesion results unless special bituminous additives (anti-stripping agents) are employed. The highest degree of adhesion will occur when clean, hot, and dry aggregates are used. If slightly dusty aggregates are used, adhesion may be improved by using damp aggregates. The use of kerosene mist on aggregates at the rate of about 1 gal of kerosene per ton of aggregate has also been found to promote binding and coating (33).

Asphalt cements and liquid asphalts that can be used for surface treatments are shown in Table 28. It is beneficial to have the bitumen remain as fluid as possible after spraying and before application of the aggregate layer so that a good bond develops between the two materials. After the aggregate is placed, rolled, and the pavement is opened to traffic, it is desirable to have the bitumen harden

Slow Curing (SC)				Emulsified (Anionic)					Emulsified (Cationic)					
70	250	800	3000	RS-1	RS-2	MS-2	SS-1	SS-1h	CRS-1	CRS-2	CMS-2s	CMS-2	CSS-1	CSS-1h
		X	X	X	X				X	X				
				X	X	X	X[a]	X[a]	X	X			X[a]	X[a]
							X	X					X	X
								X[b]						X[b]
				X[b]			X[b]	X[b]	X[b]				X[b]	X[b]
X	X						X[b]						X[b]	
X														

Figure 33. Single surface treatment design curves for one-size aggregate.



as quickly as possible so that subsequent traffic will not displace the embedded aggregate. For these reasons, RC liquid asphalts, rapid set (CS, CRS) anionic and cationic emulsions, and high-penetration (low-viscosity) asphalt cements are generally preferred.

The design quantity of asphalt is selected on the assumption that the percentage of the total voids to be filled with asphalt varies from about 80% for very light traffic conditions to around 60% for heavy traffic conditions. Table 29 may be used to estimate quantities of materials that are needed for single surface treatments and seal coats.

Design curves for single surface treatments (SST) using one-sized aggregate are shown in Figure 33. Aggregate and asphalt spread quantities are shown as functions of the average least dimension of the aggregate used.

The average least dimension is defined to be the average of the smallest dimension (length, width, height) of a sample of aggregate particles. This value can be determined by caliper measurement of a number of individual aggregate particles.

The aggregate spread quantity in pounds of aggregate per square yard is a function of the bulk specific gravity of the aggregate (G_m). The aggregate spread quantity also depends on percentage of aggregate that may be lost through material handling, brooming and traffic.

The asphalt spread quantity, gallons of asphalt per square yard, is a function of the anticipated traffic level as well as the average least dimension of the aggregate.

The curves in Figure 33 apply only to surface treatments on primed based material. If the base is a very porous bituminous surface, the asphalt spread quantities in Fig. 33 should be increased by 0.03 to 0.05 gallons per square yard. If asphalt is being applied to a very porous bituminous surface, then the tabular values should be reduced by 0.05 to 0.10 gallons per square yard.

Use of Figure 33 is illustrated by the folowing example:

<u>Assumptions</u>

Average least dimension = 0.4 in
Bulk specific gravity = 2.70
Aggregate Waste Factor = 10%
Traffic volume = 500 vpd

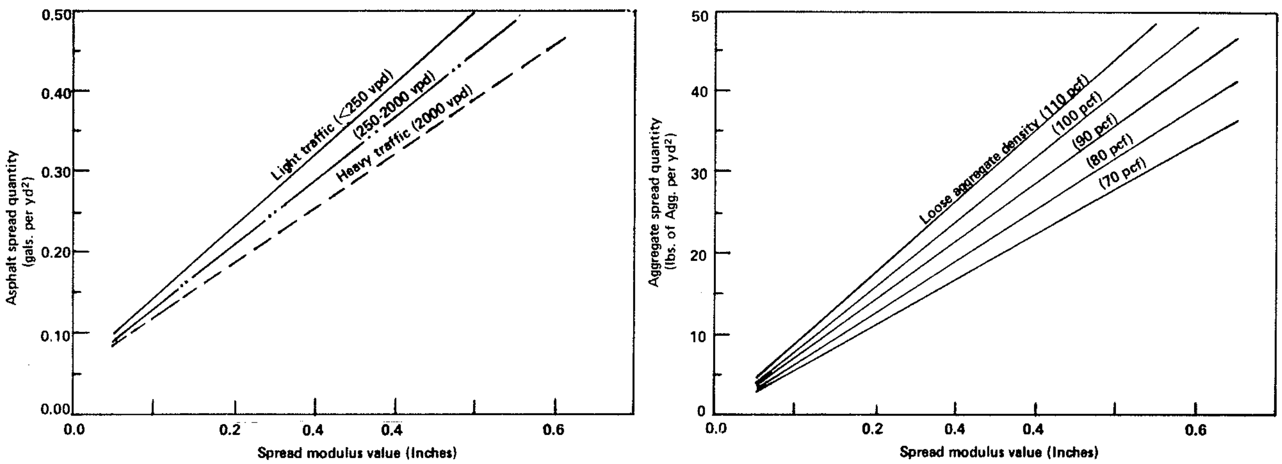
<u>Quantities from Figure 33</u>

Aggregate spread quantity = 45 lbs/yd^2
Asphalt spread quantity = 0.36 gals/yd^2.

Single surface treatment design curves for the use of graded aggregate are shown in Figure 34. In this case, the aggregate spread quantity is a function of the unit weight of the loose aggregate and the spread modulus. The asphalt spread quantity is a functon of the spread modulus and design traffic level in vehicles per day.

The spread modulus represents the mean particle

Figure 34. Single-surface treatment design curves for graded aggregate.



diameter of a graded aggregate and is calculated by the following formula.

Spread modulus $= 0.2(D_{100} + D_{80})/2 + 0.6(D_{80} + D_{20})/2 + 0.2(D_{20} + D_0)/2$,

where

D_{100}, D_{80}, D_{20}, and D_o are the aggregate sizes for which 100%, 80%, 20%, and 0% of the total material has size less than the given size (in inches).

These D values can be quickly read off the aggregate grain size distribution curve that shows percentage passing each size.

If a graded aggregate was found to have the following gradation.

$D_{100} = 0.500$ in, $D_{80} = 0.320$ in, $D_{20} = 0.200$ in, and $D_0 = 0.040$ in, then the spread modulus would be $0.2(0.820)/2 = 0.6(0.520)/2 + 0.2(0.240)/2 = 0.082 + 0.156 + 0.024 = 0.262$ in.

Use of Fig. 34 is illustrated by the following example.

Assumptions
- Spread modulus = 0.36 in
- Loose aggregate density = 100 pcf
- Traffic estimate = 800 vpd

Quantities from Figure 34
- Aggregate spread quantity = 28 lbs/yd^2
- Asphalt spread quantity = 0.33 gals/yd^2.

Further information on surface treatments is contained in Compendium 12 (see inside back cover).

CHAPTER 8

Improvement of Material Quality

Two primary methods are available for improving the quality of granular materials in low-volume road structures. The first is compaction, the second is chemical stabilization.

COMPACTION

Benefeits from compaction are (a) increased material strength (CBR), (b) improved resistance to densification under traffic loadings, and (c) reduced ability of moisture to flow through the material.

In the laboratory, the compactive effort applied to a given material is controlled by keeping the unit volumetric energy used to compact a specimen at a constant magnitude. Laboratory compaction is generally achieved by dropping a hammer a fixed distance into soil placed within a mold of known volume. Besides the hammer weight and height of fall, the number of soil layers and the number of blows per layer will determine the total energy used to compact the soil.

At present there are two major compaction tests that are widely used throughout the world in pavement engineering. These are (a) the Standard (or Proctor) test and (b) the Modified Compaction test.

Values for the test variables are given below for each type of test.

Compaction Variable	Standard	Modified
Hammer weight	5.5 lb	10 lb
Hammer fall distance	12 in	18 in
Number of soil layers in mold	3	5
Number of hammer blows per layer	25	25
Volume of compaction mold	(1/30)ft^3	(1/30)ft^3

For each type of test, a unique compaction curve of moisture content versus density can be obtained by compacting several specimens at differing moisture contents. In Fig. 21 (Chapter 6) curve M was developed under standard compaction effort; while curve H represents the modified compaction test.

In the field, compaction is normally controlled by specifications that require a minimum acceptable level of density. The variable used for compaction control is called percent compaction. Percent compaction (PC) is defined by

PC = 100x(dry density of field-compacted material)/(optimum dry density of the same material when compacted in the laboratory at optimum moisture content).

The value of the denominator must be determined by developing a density-moisture curve for the material, as shown in Fig. 21, for either the standard or modified test. The numerator dry density must be determined by whichever compaction method has been used for the denominator.

To illustrate the control procedure, assume that a subgrade soil has the laboratory compaction characteristics shown in Figure 22 and is being compacted in the field. Moisture content tests indicate that the field moisture is 12%. Density tests show that the in-place field density is 128.8 pcf. The field dry density is therefore (128.8)/(1+0.12) = 115.0 pcf.

Fig. 22 shows that the optimum dry density for the modified compaction test (curve H) is 119.0 pcf. Thus, PC (modified) = 100(115.0)/(119.0) = 96.6%.

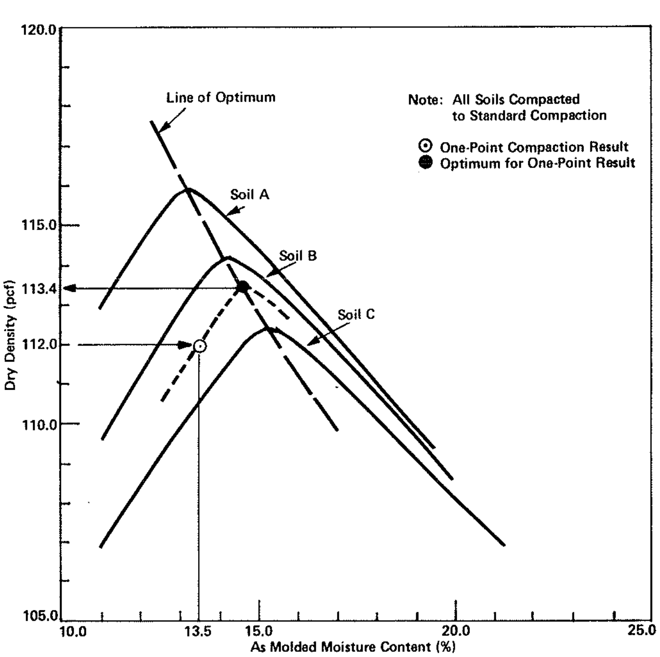
If a percent compaction had been based on the standard compaction test, the optimum dry density would be 114.0 pcf from curve M of Figure 21. in this case, PC (standard) = 100(115.0)/(114.0) = 100.9%.

Field compaction is controlled by measuring the dry density of samples of the field-compacted soil. A common method for evaluating the field density is the sand cone test in which compacted material is extracted and the resulting hole is filled with sand. The extracted material is weighed and the volume of the hole is derived from the amount of sand used to fill the hole.

As an illustration, suppose that the extracted soil weighed 4.74 lb, the volume of sand used was 0.0368 ft^3 and that the field moisture content was 12%. The field wet density should then be (4.74)/(0.0368) = (128.8)/(1+0.12) = 115.0 pcf. The corresponding percent compaction would be calculated as shown above.

Quite often, several different soils may be encountered over the area of a construction project.

Figure 35. Typical one-point compaction problem.



When this occurs, laboratoiry moisture-density curves should be developed for each soil and for whichever compaction test is being used. The result is a family of curves as shown in Fig. 35. Each soil in the family has a different set of optimum moisture and optimum density values.

The procedure used to establish optimum density for the soil being tested is to use what is called a one-point compaction test. Suppose, for example, that the compacted soil sample has a field moisture content of 13.5%. Suppose also that, when the sample is compacted in a mold to standard compaction, the ensuing dry density is 112.0 pcf. The moisture content and dry density values correspond to the circled point in Figure 35. By interpolating a compaction curve that is parallel to the curves for soils B and C and that passes through the circled point, the optimum dry density is estimated to be 113.4 pcf. This value then becomes the denominator for evaluating percent compaction.

For subgrade materials, minimum requirements of either 95% (standard) or 90% (modified) for the upper 8 in (20 cm) to 12 in (30 cm) are usual. Unbound granular materials normally have requirements of 100% (standard) or 95% (modified) (22, 28, 35, 36).

Further information on the compaction of roadway soils is contained in Compendium 10 (see inside back cover).

CHEMICAL STABILIZATION

The use of chemical additives can be a very efficient way to improve the properties of almost all road building materials. Stabilizers will normally improve the immediate and long-term strength of the materials. Additives can also alter the plasticity properties of the soil portion and thereby minimize the potential influence of high-volume changes. Recent evidence shows that the use of stabilizers may increase the resistance of granular material to degradation under service conditions.

Benefits derived from additives are generally proportional to the amounts used and therefore to the cost of the additives. Even small amounts of stabilizer (2%-3%) can modify and improve the soil material.

In addition to increased costs, there are other limitations on the use of stabilizing additives. In general, the use of stabilizers implies a higher level of technical skills, engineering effort, and availability of special construction equipment. Thus, even though the use of additives may be potentially beneficial, the use may not be economic or physically possible.

The major types of additives are (a) lime, (b) cement, and (c) bitumen, including road tars.

Because of lime reactions with aluminates and silicates, lime is probably the most beneficial additive for clay soils or gravel soil mixtures that have clay fines. Lime cannot be used to stabilize organic soils, sands, or granular materials with poor (open-type) gradations. Thompson (37) suggests a minimum clay content of 10%, while the TRRL (22) suggests a minimum of 15% passing the No. 40 sieve and a coefficient of uniformity greater than 5 as conditions for the use of lime stabilization. The decision as to the acceptability of lime as a stabilizer should be made on the basis of laboratory tests.

From a practical viewpoint, addition of lime to soils will tend to reduce the soil PI and increase the shrinkage limit. Both of these results will tend to reduce the high volume change potential of plastic clay. Both the expansion and shrinkage potentials may be effectively controlled by this stabilization technique.

The percentage of lime additive to use depends on the type of soil and the potential reason for stabilizing. If long-term high strengths are the ultimate objective, then the lime percentage will be high. However, even small percentages of lime may reduce the potential for high volume change and increase immediate strength. In general, the range of lime percentage for most soils will vary from about 2 to 10%. If strength is desirable, the lime percentage will be in the upper range but most lime soil modifiers will be less than 4%. Higher lime percentages are needed for clays with higher plasticity indexes. Less lime is normally needed to stabilize clay gravels than to stabilize fine-grained soils. Minimum percentages of 2-3% are normally used to ensure uniformity of mixing in the soil.

Cement-stabilized soils have been used in pavement construction for more than 50 years to improve road performance. In many respects, cement stabilization accomplishes improvements similar to those for lime stabilization. When combined with moisture, the cement hydrates to form a cemented product. The degree of cementation, and hence degree of improvement, is directly proportional to the quantity of portland cement that has been added. Thus, even small amounts of cement will improve the strength and properties of the soil.

With the exception of organic materials, cement can be used effectively on soil types that range from plastic clays to high-quality crushed stone.

Because of the lack of dependence of soil type on the degree of hydration, the major benefit of cement stabilization is increased strength. Decreased soil plasticity and increased resistance to high volume change are also important benefits. The percentage of cement to be used depends primarily on the soil type and the intended objective of the stabilization effort. The cement percentage generally decreases as the stabilized material goes from a clay to a gravel. Increases in clay plasticity require greater stabilization percentages that often prove impractical and uneconomical. Typical values are 15%-20% for plastic clays down to 3%-5% for gravels (38).

Although there are many potential advantages for using cement stabilized soils, several limitations should be recognized. As for lime, the high strengths associated with cement will generally create a very rigid or stiff material. Although rigidity provides increased resistance to shear deformation, increased tensile stresses due to loads may develop and cause fatigue cracking. Another potential problem with cement-treated materials is the possibility that polygonal shrinkage cracks will develop throughout the layer depth. Surface water can then seep through the openings and reduce the in situ strength of the subgrade.

Cement-stabilized soil differs greatly from lime-stabilized soil in that the rate of strength gain is very fast and the magnitude of strength improvement is generally greater. The rapidity of the hydration necessitates compaction within a few hours of initial mixing. Compaction after initial setting will radically alter the strength gain and destroy the hydration process.

In many locations throughout the world, bituminous stabilization can be economically and efficiently employed to improve material quality. Unlike lime and cement, benefits from bituminous stabilizers are derived from the internal adhesive and cohesive forces of the bitumen itself.

Almost all soils have the potential for being stabilized with bituminous materials. Experience has shown that bituminous stabilization is most effective for (a) clays, (b) sandy soils, (c) sand gravels, and (d) crushed stone.

For clays, the major role of the bituminous material is to waterproof the clay particles and allow the full strength of the compacted soil to be mobilized. Thus bitumen tends to counter the strength reductions that are brought about by moisture increases in an unsaturated soil. Clay-bitumen stabilization is generally recommended only for clay materials that have (a) liquid limits less than 40%, (b) plastic indexes less than 18%, and (c) less than 50% passing the No. 200 sieve size.

The major application of bituminous stabilizers is for sandy soils. In these cases the bitumen can markedly improve the strength of local materials. As for all bituminous-aggregate mixtures, strength is increased by providing a cohesive component to the shear strength. Pure granular materials lack this attribute and depend on confining stresses to develop full friction. An optimum bitumen content will always exist because increased addition of the stabilizer will eventually cause the individual sand particles to be displaced from a grain-to-grain contact position. Such a condition then effectively reduces (eliminates) the potential friction of the granular mass.

Not all sands and sandy soils are conducive to bitumen stabilization. The materials should not have PI values greater than 12, and no more than 25% should pass the No. 200 sieve. Thus dirty sands may be difficult to stabilize effectively. Similarly, minimal strength increases can be expected with open-graded or uniform sands because this material possesses little internal friction. In many practical cases, these sands can be blended with fine-grained soil to achieve better gradation before bituminous stabilization is attempted.

In general, the range of bitumen percentage for sand stabilization is 4-12%. For both soil-bitumens and sand-bitumens a primary objective is to achieve uniform mixture of the bitumen and soil material. Proper field compaction is necessary if quality is to be optimized. For soil-bitumen compaction, optimum moisture should be used; sand-bitumens are normally compacted at moisture contents of less than 5%. Bituminous materials that may be used for various soil types are listed below.

Soil Type	Bituminous Material
Soil-bitumen	MC 250/800; SC 250/800 SS-1; SS-1h CSS-1; CSS-1h
Sand-bitumen (clean)	RC 70/250/800; MC 250/800 MS-2; MS-2h CMS-2; CMS-2h
Sandy soils	RC 250/800; MC 250/800 SS-1; SS-1h CSS-1; CSS-1h

Heavy road oils sprayed on the surface of granular-surfaced roads will produce a waterproofing and dust-control effect. Because maximum bituminous penetration is highly desirable, slow-cure cutbacks or slow-set emulsions are normally used. Before application, the surface should be bladed and moistened to increase the effectiveness of the bituminous material. About 1 gal/yd^2, applied in two to three separate applications, is generally quite satisfactory for most conditions. Care should be exercised to prevent excessive quantities of road oil in localized areas. When this occurs, sand may be applied to blot up excess bituminous material.

Further information on soil stabilization is contained in Compendium 8 (see inside back cover).

CHAPTER 9

Structural Design Methods

This chapter presents structural design methods that have been developed by (a) the U.S. Army Corps of Engineers (USACE), (b) the United Kingdom Transport and Road Research Laboratory (TRRL), and (c) the U.S. Forest Service (USFS). Additional methods might have been included but those presented contain most of the basic features of current methodology. Moreover, no current design procedure has universal acceptance or applicability. It is therefore important for the designer to acquire performance feedback for any method that has been used and to modify the design procedures in the light of performance experience. Any design method should be carried out in consonance with the many structural principles that have been presented in the previous chapters of this synthesis.

USACE DESIGN PROCEDURES

The USACE has accumulated much experience on low-volume road design and performance (4, 18, 19). Although major concern has been for surface mobility of military vehicles and aircraft, the USACE experience includes earth roads, granular-surfaced roads, and roads that have bituminous surface treatments.

The USACE procedures are based on equations that give required thicknesses for material that is to be placed over underlying material of a given strength (CBR), provided that the placed material has greater CBR strength than the underlying material.

The term "required" refers to a thickness that will withstand a specified number of axle loads before the structure reaches a level of deformation that corresponds to low serviceability. In the design equations, axle loads are characterized by equivalent single wheel loads (lb) and by tire contact areas (in^2). Thus, the basic USACE design equations can be stated in terms of standard 18,000-lb single axle loads that were discussed in Chapter 4.

Required thicknesses for various CBR values and for various numbers of equivalent 18,000-lb single axle load repetitions (N_{18}) are shown in Figure 36. The thickness scale on the left side of the figure is for bituminous surface treatment (BST) structures, the scale on the right is for granular-surfaced roads. For example, if the subgrade CBR = 8% and if the design life is N_{18} = 10,000 repetitions of the standard axle load, the required thickness for a BST structure is nearly 10 in. For the same conditions, the required thickness for a granular surface is about 7.5 in. In either case the material to be placed on the subgrade must have CBR strength greater than 8%.

It will be found that the granular surface thicknesses are all about 78% of the corresponding BST thickness. This difference arises mainly because the design equation for granular surfaces permits greater deformation at failure than does the BST design equation.

The design curves in Figure 36 can be used to determine thicknesses for multilayer structures in which each successive layer has greater strength than the preceding layer. For example, for N_{18} = 1,000,000 repetitions, a total thickness of about 35 in is required for a BST structure over a subgrade whose CBR = 2%. If a granular subbase with CBR = 20% were placed over the subgrade, Figure 36 shows that about 7 in of still stronger material would be required to protect the subbase layer. Thus the design cross section could be 35 - 7 = 28 in of subbase (CBR = 20%), 6 in of base (CBR > 20%) and 1 in of BST.

The foregoing USACE design procedures do not provide thickness adjustments for the CBR strengths of the granular materials. Thus, in the example given above, a total thickness of 35 in is required, irrespective of the amounts by which the CBR values of the granular layers exceed the subgrade CBR. For granular-surfaced roads, the USACE has developed a design equation, called the rut-depth model, that takes into account the CBR strength of the granular surface material.

Selected points from curves that represent the rut-depth model are shown in Table 30. Entries within the table give required CBR strength for various combinations of equivalent single axle loads, granular surface thickness, and subgrade CBR. For example, if N_{18} = 100,000 equivalent single-axle loads, if the granular surface is to be 12 in thick, and if the subgrade CBR is 6%, then Table 30 shows that the granular surfacing material should have CBR strength of 63%.

Table 30 can also be used to estimate the required thickness of granular surfacing material whose CBR strength is known. For example, if the granular material has CBR = 50% and is to be placed

Figure 36. Thickness design curves for surface-treated roads and granular surface roads (USACE analysis).

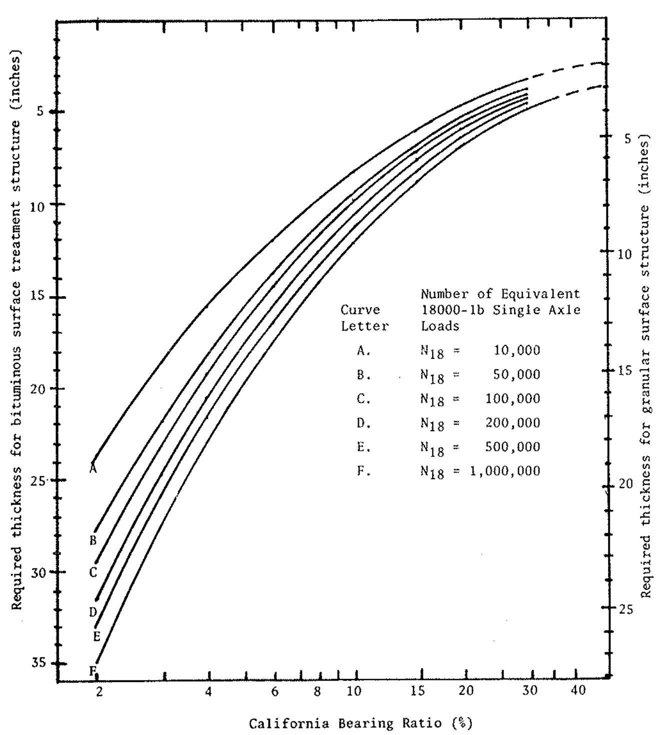


Table 30. Required CBR strength of granular surfacing material (18).

Number (000s) of Equivalent 18 000-lb Single-Axle Loads (N_{18})	Subgrade CBR (%)	Thickness of Granular Surface (in) 6	9	12	15	18	21	24	27	30
10	2	96	62	48	40	34	31	28	26	24
	4	78	50	38	32	28	25	23	21	20
	6	69	44	34	28	25	22	20	19	17
	8	63	41	31	26	23	20	18	17	16
	10	59	38	29	24	21	19	17	16	15
	15	52	33	26	21	19	17	15	14	13
	20	48	31	24	20	17	15	14	13	12
50	2	147	95	73	61	53	47	43	40	37
	4	119	77	59	49	43	38	35	32	30
	6	105	68	52	43	38	34	31	28	27
	8	96	62	48	40	35	31	28	26	24
	10	90	58	45	37	32	29	26	24	23
	15	79	51	39	33	28	25	23	21	20
	20	73	47	36	30	26	23	21	20	18
100	2	178	114	87	73	63	57	52	48	45
	4	143	92	71	59	51	46	42	39	36
	6	126	82	63	52	45	41	37	34	32
	8	116	75	57	48	41	37	34	31	29
	10	108	70	54	46	39	35	32	29	27
	15	95	62	47	39	34	31	28	26	24
	20	87	56	43	36	31	28	26	24	22
500	2	270	175	134	111	97	87	79	73	68
	4	219	141	108	90	78	70	64	59	55
	6	194	125	96	80	69	62	57	52	49
	8	177	115	88	73	64	57	52	48	45
	10	166	107	82	68	59	53	48	45	42
	15	146	94	72	60	52	47	43	40	37
	20	134	86	66	55	48	43	39	36	34
1000	2	325	210	161	134	116	104	95	88	82
	4	263	170	130	108	94	84	77	71	67
	6	233	150	115	96	83	75	68	63	59
	8	213	138	106	88	76	68	62	58	54
	10	199	129	99	82	71	64	58	54	50
	15	176	114	87	72	63	56	51	48	44
	20	161	104	80	66	58	52	47	44	41

on subgrade whose CBR = 2% for a design life of N_{18} = 10,000 equivalent single axle loads, then about 12 in of the granular material would be required. However, if the granular material had CBR = 35%, then nearly 18 in thickness would be required for subgrade CBR = 2% and N_{18} = 10,000.

If Figure 36 is used, a thickness of about 18 in of granular surface is required for subgrade CBR = 2% and N_{18} = 10,000. Thus for these conditions, the thickness required by Figure 36 corresponds to the thickness required by Table 30 when the granular material has a CBR = 35%. The design equations represented by Figure 36 and Table 30, respectively, are based on somewhat different assumptions and are not necessarily consistent. For example, for CBR = 2% and N_{18} = 100,000, Figure 36 implies that the required thickness of granular surface material is 23 in, provided that its CBR is greater than 2%. On the other hand, if the granular material has CBR = 30%, then Table 30 indicates that more than 30 in of thickness is required. It is recommended that Figure 36 be used to determine granular surface thickness and that Table 30 be used secondarily as a tool to estimate strength requirements.

In summary, the following steps should be taken as USACE design procedures are used for bituminous surface treatment structures or for granular-surfaced structures.

1. Analyze traffic factors according to the information presented in Chapter 4. The result should be a design value for N_{18}, the number of equivalent 18,000-lb axle load repetitions that are expected for the design life of the structure.
2. Analyze the subgrade material and available granular materials according to information presented in Chapters 5, 6, and 7. The result should be estimated values for the CBR strength of each material.
3. Use Figure 36 to determine required thicknesses for the total structure and for individual layers of granular material in a multilayer structure.
4. Specify quality and strength for granular materials in accordance with principles presented in Chapter 7. For granular surface structures, use Table 30 to estimate strength requirements for granular material.
5. For BST structures, specify materials and procedures for the bituminous surface according to information given in Chapter 7.

TRRL DESIGN PROCEDURE

The TRRL of the United Kingdom has developed a design procedure for bitumen-surfaced roads in tropical and subtropical countries (22). 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 bituminous surface treatment (BST) structures are shown in Figure 37. Required thicknesses for BST structures are shown at left for various levels of subgrade strength (CBR) and axle load repetitions (N_{18}). The TRRL curves are similar to the USACE design curves (Figure 36), but required thicknesses in Figure 37 average about 10 percent less than the corresponding thicknesses in Figure 36. On the other hand, the TRRL procedure recommends a minimum base thickness of 6 in, and a minimum value of CBR = 80% for the strength of the base material. If a subbase is used, minimum values for the subbase material are 4 in of thickness and CBR = 25% at the expected field moisture-density conditions.

As an example of the use of the TRRL method,

Figure 37. Thickness design curves for surface-treated roads (TRRL).

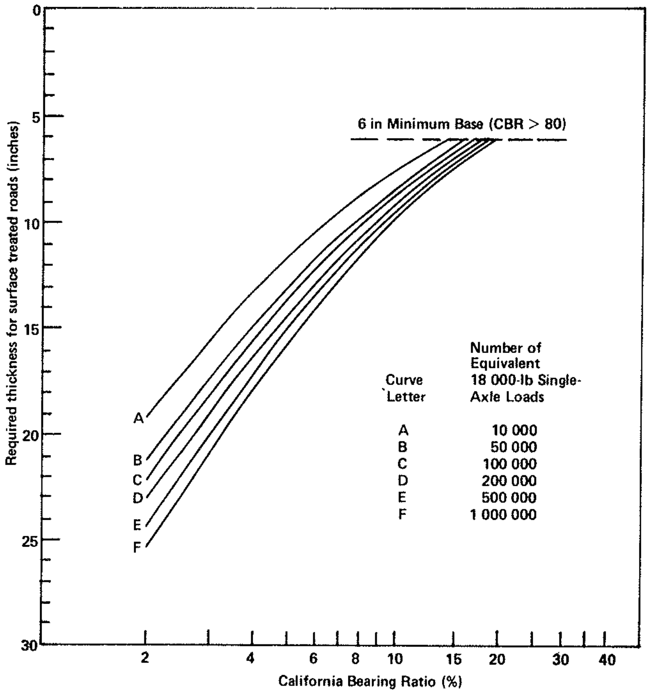
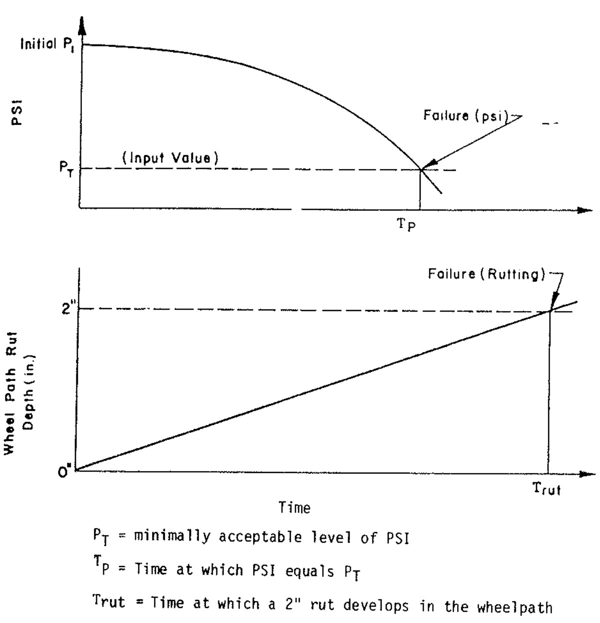


Figure 38. U.S. Forest Service failure criteria.



suppose a BST structure is being designed for subgrade CBR = 6% and for N_{18} = 1,000,000 equivalent 18,000-lb axle load repetitions. Figure 37 shows that the required pavement thickness is about 14 in. Assuming that the surface treatment is about 1 in thick and that the 6 in minimum base thickness is used, then about 14-6-1 = 7 in of subbase material is required.

If the 78% factor is applied to the TRRL curves as for the USACE curves, then the required thickness for granular-surfaced roads would be 0.78 times the corresponding thickness given by Figure 37 for BST roads. Thus, if subgrade CBR = 6% and N_{18} = 1,000,000, the 14 in thickness given by Figure 37 would be multiplied by 0.78, and about 11 in of granular material would be required for the surface layer.

Table 31. Correspondence between subgrade CBR strength, soil suport value (SS), or group index (GI).

Subgrade Strength (CBR %)	Soil Support Value (SS)	Group Index (GI)
2	2.2	
3	3.0	20
4	3.6	17
5	4.0	14
6	4.3	11
8	4.9	5
10	5.3	4
15	6.1	1.8
20	6.7	1.3
30	7.4	0.6
40	8.0	0.0

U.S. FOREST SERVICE PROCEDURES

The USFS is responsible for the design and operation of a large network of paved and unpaved roads and has developed comprehensive procedures for the structural design of roads (39). These procedures are being revised in terms of a system design approach that is based on minimization of total life cycle costs (40, 41, 42). The new procedures, however, will not be discussed in this synthesis.

Failure criteria used in the USFS design procedures are shown in Figure 38. The first criterion is present serviceability index (PSI) that begins at an initial point, P_I, and reaches a failure level, P_T, after a period of traffic and time T_P. In the remainder of this section, initial serviceability will be assumed to be P_I = 4.0 for both bituminous surface treatment roads and for granular-surfaced roads. Terminal serviceability will be assumed to be P_T = 2.0 for bituminous-surfaced roads and P_T = 1.5 for granular-surfaced roads.

The second criterion in Figure 38 is for rutting and refers only to granular-surfaced roads. Under this criterion, failure occurs when rut-depth reaches a specified design value, say 2 in. The design life is then the time (T_{rut}) required for failure to occur.

In addition to design values for serviceability index or rut-depth, the following three factors are basic to the USFS design procedure.

1. Soil Support (SS) = an empirical soil strength parameter that is not measured directly but that has correlation with CBR strength and group index values as shown in Table 31. The table shows that SS ranges from about 2.2 when subgrade soil CBR = 2% to about 8.0 when subgrade CBR = 40%.
2. Structural Number (SN) = $a_1D_1 + a_2D_2 + \ldots$ 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 32.

Suppose, for example, that a two-layer granular structure has D_1 = 6 in of surfacing whose CBR = 60% over a granular subbase whose thickness is D_2 = 10 in and whose CBR = 30%. Table 32 shows that the respective coefficients are a_1 = 0.126 and a_2 = 0.109. The structural number would therefore be $SN = 0.126(6) + 0.109(10) = 1.846$.

For bituminous surface treatments whose maximum aggregate size is as least 1 in, the first term of the structural number will be a_1D_1 where $a_1 = 0.25$ and D_1 is the thickness of the bituminous layer (e.g., 1 in).

3. Design Life (W_T) = number of equivalent 18,000-lb single axle loads to be experienced during the design period. Thus, W_T is the accumulation of equivalent axle loads between the times that PSI = P_I and PSI = P_T (see Figure 38). Methods for estimating W_T have been discussed in Chapter 4 where the notation N_{18} was used for equivalent 18,000-lb single axle load repetitions.

Other elements of the USFS design procedure include factors to account for serviceability loss through environmental effects only and a factor for adjusting W_T to account for environmental variations among regions that have quite different climates. These environmental factors must be evaluated through engineering judgment and will not be discussed in this synthesis.

The basic design factors are brought together in Table 33, which gives SN values for various combinations of SS values and equivalent single axle loads (W_T). The upper portion of the table is for terminal serviceability $P_T = 2.0$, the lower portion is for $P_T = 1.5$, and all values are for initial serviceability $P_I = 4.0$. It is recommended that the upper portion ($P_T = 2.0$) be used for BST roads and that the lower portion ($P_T = 1.5$) be used for granular-surfaced roads.

Table 32. Correlation between CBR strength of granular materials and structural number 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	

Note: For a bituminous surface layer, $a_1 = 0.25$. If the layer thickness is at least 1.0 in (or) for a bituminous surface layer at least 1.0 in thick, $a_1 = 0.25$.

To illustrate the use of Table 33 for a BST pavement structure, suppose that the subgrade CBR = 10% and that the design life is W_T = 100,000 for $P_T = 2.0$. Table 31 shows that the soil support value is SS = 5.3 and Table 33 shows that the required structural number is about 1.93. Suppose also that the structure will have about 1 in of bituminous surface treatment with a maximum aggregate size of 1.0 in and that a 4-in base course and a subbase course will be used. Thus, the structural number has the form: $SN = a_1D_1$ (surfacing) + a_2D_2 (base) + a_3D_3 (subbase). On substitution of SN = 1.93, $a_1 = 0.25$, and $D_1 = 1$ in, the equation becomes $1.93 = 0.25(1) + a_2D_2 + a_3D_3$.

If CBR = 80% for the base material and CBR = 40% for the subbase material, Table 32 shows that $a_2 = 0.136$ and $a_3 = 0.120$. On substitution, the equation becomes $1.93 = 0.25(1) + 0.136(4) + 0.120 D_3$, or $1.136 = 0.120 D_3$. Thus $D_3 = 9.5$ in for the subbase thickness, and the overall structure will have 1 in (surface) + 4 in (base) + 9.5 in (subbase) = 14.5 in (total).

To illustrate the procedures for one-layer granular-surfaced roads, suppose the granular material has CBR = 65%, is to be placed on a subgrade with a CBR = 6%, and is to have a design life of W_T = 200,000 equivalent 18,000-lb single axle loads. Table 31 shows that the soil support value is SS = 4.3 and interpolation in Table 32 gives $a_1 = 0.129$ as the structural number coefficient for the granular surface material. Interpolation in Table 33 gives a structural number of SN = 2.44 for $P_T = 1.5$, W_T = 200,000, and SS = 4.3. The structural number equation is therefore $SN = a_1D_1$, or $2.44 = 0.129 D_1$. Thus the required thickness of surfacing material is $D_1 = 2.44/0.129 = 18.9$ in.

In summary, the following steps are to be taken when using the USFS design procedure for the PSI criterion.

1. Analyze traffic factors according to the information presented in Chapter 4. The result should be a design value for W_T, the number of equivalent 18,000-lb axle load repetitions that are expected before PSI is at the terminal serviceability value, P_T. Use $P_T = 2.0$ for bituminous surface treatment roads and $P_T = 1.5$ for granular surfaced roads.
2. Analyze the subgrade material and available granular materials according to information presented in Chapters 5, 6, and 7. The results should be an estimated value for the CBR strength of each material.
3. Specify quality and strength for granular materials in accordance with principles presented in Chapter 7.

Table 33. Structural number (SN) values for bituminous or granular-surfaced structures (USFS PSI criterion).

Serviceability Index	Number (000s) of Equivalent 18 000-lb Single-Axle Loads (W_T)	Soil Support Value (SS) 2	3	4	5	6	7	8	9	10
$P_I = 4.0$, $P_T = 2.0$	10	2.10	1.82	1.57	1.34	1.14	0.95	0.78	0.62	0.48
	20	2.36	2.05	1.77	1.53	1.30	1.10	0.92	0.75	0.60
	50	2.73	2.38	2.07	1.79	1.55	1.32	1.11	0.93	0.76
	100	3.07	2.66	2.34	2.01	1.74	1.50	1.28	1.08	0.90
	200	3.46	2.98	2.59	2.25	1.96	1.70	1.46	1.24	1.04
	500	4.31	3.51	3.02	2.62	2.27	1.98	1.71	1.48	1.25
	1000	4.75	4.09	3.39	2.98	2.54	2.22	1.93	1.67	1.43
$P_I = 4.0$, $P_T = 1.5$	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
	1000	4.04	3.60	3.19	2.83	2.49	2.19	1.91	1.66	1.42

Table 34. Structural number (SN) values for granular-surfaced structures (USFS rut-depth criterion).

Number (000s) of Equivalent 18 000-lb Single-Axle Loads (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
1000	4.68	3.19	2.51	2.09	1.78	1.26	0.91

4. Use Table 31 to obtain the subgrade soil support value (SS). Use Table 32 to find a structural number coefficient (a_i) for each granular material that will be used.
5. Enter Table 33 with values for P_T, W_T, and SS to determine the required structural number, SN.
6. Form the equation SN = $a_1D_1 + a_2D_2 + \ldots$ wherein a_1D_1 represents the topmost structural layer, a_2D_2 represents the second layer, etc.
7. Substitute values for $a_1, a_2, \ldots$, and SN from steps 3 and 4 into the equation, leaving the thicknesses D_1, D_2, etc., to be determined.
8. Determine a set of values for D_1, D_2, etc., that satisfy the equation resulting from Step 7. In general there will be many possible solutions. The final choice should take into account any specifications for minimum thicknesses. If the structure has more than one layer, the thickness choices should also be based on relative unit costs of the different materials.

For granular-surfaced roads, the rut-depth criterion may be used as an alternative to the PSI criterion (see Figure 38). The USFS design equation for a failure criterion of 2 in rut-depth is represented by Table 34. Entries in Table 34 are structural numbers required for various combinations of subgrade CBR and equivalent 18,000-lb single-axle load repetitions.

For single-layer granular surfacing, Table 34 gives structural numbers for the equation SN = a_1D_1. Thus the required thickness is D_1 = SN/a_1, where values of a_1 are given in Table 32 for materials with various CBR strengths. For example, if the granular material has CBR = 80%, is to be placed on a subgrade with a CBR = 6%, and is to have a design life of W_T = 50,000, then Table 34 shows that a structural number of SN = 2.03 is required. Table 32 shows that a_1 = 0.136 for a granular material whose CBR = 80%. Thus 2.03 = 0.136 D_1 and D_1 = 14.9 in of granular surfacing material is required.

In this example, if only 6 in of the CBR = 80% material were used in the top layer of a two-layer granular-surfaced road and if granular material in the second layer had CBR = 40%, then Table 32 shows that a_2 = 0.120 for the second layer. In this case SN = $a_1D_1 + a_2D_2$, or 2.03 = 0.136(6) + 0.120D_2, and D_2 = 10.1 in. Thus, the structure would have a total thickness of 6 in + 10 in, or 16 in.

CHAPTER 10

Design Examples

The main purpose of Chapter 10 is to provide further examples of the use of design procedures that were presented in Chapter 9. Secondary aims are to exemplify principles and methods that were given in earlier chapters. The first five examples relate to granular-surfaced roads, three additional examples are presented for roads with bituminous surface treatments. In most of the examples, two or more of the USACE, TRRL, and USFS procedures are illustrated.

Example 1. Granular-Surfaced Road, Minimal Design Data.

A two-lane granular-surfaced road is to be designed for a five-year period. It is estimated that traffic will average to be about 100 vpd and that a low percentage of the traffic will consist of trucks having relatively high axle loads. The only available information on the subgrade soil is that it is composed of about 80% sand, 15% silt, and 5% clay. The design problem is to determine a thickness and minimum strength for the granular surface material.

All three design procedures given in Chapter 9 require estimates for (a) the number of equivalent 18,000-lb axle loads (N_{18}) that will be experienced and (b) the strength (CBR) of the subgrade soil.

The approximation method for mixed-traffic analysis (Chapter 4) can be used to estimate N_{18}. Table 10 shows that M = 37 for low percent trucks and heavy load distribution per truck. If a growth factor of 10% is assumed, then Figure 15 gives G = 6.2 for the assumed growth rate. Thus, the estimate for N_{18} is N_{18} = M x ADT x G = 37 x 100 x 6.2 = 23,000 (approximately).

Subgrade strength may be estimated indirectly through the USCS soil classification system (Chapter 5). If the subgrade soil is considered to be a silty sand (SM in Table 14), then Figure 19 shows that the subgrade CBR may be as low as 10.

If the USACE design procedure is used to determine the required thickness of granular surface, then Figure 36 is entered with CBR = 10% and N_{18} = 23,000 to give an approximate thickness of 7 in.

Table 30 may be used to estimate required strength for the granular material. For subgrade CBR = 10% and surface thickness 7 in, Table 30 shows that the required CBR is about 52% if N_{18} = 10,000 and about 79% if N_{18} = 50,000. The required CBR for N_{18} = 23,000 is about one-third of the distance between 52% and 79%. Thus the estimated required CBR is approximately 60% for the granular material.

If the USFS procedure (PSI criterion) is used, then Table 31 shows that the soil support value (SS) = 5.3 for subgrade CBR = 10%. Interpolation in Table 33 gives SN = 1.48, when P_T = 1.5, W_T = 23,000 and SS = 5.3. Since SN = a_1D_1, then a_1D_1 = 1.48.

There are many combinations of values for a_1 and D_1 whose product will give the required structural number. If, for example, the surface material has CBR = 70%, then Table 32 shows that a_1 = 0.132. It follows that $0.132D_1$ = 1.48 and that D_1 equals about 11 in of surfacing material.

If the USFS rut-depth criterion is used, interpolation in Table 34 for subgrade CBR = 10% and W_T = 23,000 gives SN = 1.34. If the granular material has CBR = 70%, then a_1 = 0.132 as above, and D_1 = 1.34/0.132 or about 10 in of surfacing material.

At least for this example, either USFS procedure leads to a greater thickness requirement than does the USACE procedure. Much of the difference arises because the USACE procedure permits greater deformation at failure, namely, from 2-3 in of rut depth.

<u>Example 2.</u> <u>Granular-Surfaced Road, Axle Load Data, Two-Layer Structure.</u>

A two-lane granular-surfaced road is to cross an alluvial flood plain area having a high ground water table. The subgrade soil has PI = 22 and has a saturated CBR value of 5%. This material is uniformly distributed within the design area. Good quality granular subbase material (CBR = 30%) and crushed stone base material (CBR = 100%+) are readily and economically available. The road is to have a design life of 10 years and has an ADT of 420 vpd (two-way). Traffic is expected to grow at a rate of 8% per year. Trucks represent 30% of the traffic volume. The general distribution of heavy axle loads is as follows:

Axle Load and Type	No. 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 required thicknesses for one-layer surfacing by using either of the available materials, and to determine thicknesses for each material when used together in a two-layer structure.

In this example, only the given axle load distribution will be used to determine equivalent 18,000-lb single-axle load repetitions. All lesser loads will be ignored in the mixed traffic analysis (see Chapter 4). The first step is to determine equivalent 18,000-lb axle loads (N_{18}) per 100 trucks. Equivalence factors (F), found in Table 8 are 0.18, 3.62, and 1.38, respectively, for the three types of axles for which data are given above. Thus, for each 100 trucks N_{18} = (0.18) (102) + (3.62)(54) + (1.38)(44) = 275 per 100 trucks, or 2.75 per truck. The total number of trucks in one lane during the first year of traffic is (420 vpd/2) x 30% x 365 days = 23,000 trucks (rounded). Thus, for the first year of traffic N_{18} = 2.75 x 23,000 = 63,250 equivalent axle loads. For a growth rate of 8% per year over a 10-year period, Figure 15 gives the growth factor, G = 15. Thus the final design value for equivalent axle loads is N_{18} = 63,250 x 15 = 950,000 (rounded).

If the USACE procedure is used for a one-layer granular surface, then Figure 36 shows that the required total thickness of granular surfacing is about 15 in when subgrade CBR = 5% and N_{18} = 950,000. Table 30, however, shows that the granular material would have to have minimum strength somewhat greater than CBR = 100%. Thus, only the crushed stone base material would qualify for the one-layer granular surface.

If the USFS procedure (PSI criterion) is used for a one-layer design, then Table 31 shows that the soil support value for subgrade CBR = 5% would be 4.0. Table 32 shows that a_1 = 0.140 for the base material (CBR = 100%+). If the granular material with CBR = 30% is used as surfacing material, a_1 = 0.093, but a_1 = 0.109 if this material is used as subbase in a two-layer structure. Table 33 shows that the required structural number (P_T = 1.5) is about SN = 3.16. Thus, if the subbase material is used as surfacing 3.16 = $0.093D_1$, and the required thickness is D_1 = 3.16/0.093 = 34 in. If the base material is used for surfacing, then 3.16 = $0.140D_1$ and the required thickness is D_1 = 3.16/0.14 = 23 in.

If the USFS rut-depth criterion is used it will be found (by using Table 34) that about 20-in thickness is required if the crushed stone base is used.

Both the USACE and USFS procedures give multiple solutions when both materials are used in a two-layer design. To simplify the example, suppose that D_1 = 6 in of the stone base material (CBR = 100%+) will be used in the top layer. Since it was determined above that a total thickness of 15 in is required by the USACE procedure, then the USACE two-layer design would be 15 - 6 = 9 in of subbase and 6 in of crushed stone surfacing.

If the USFS procedure is used in connection with the rut-depth criterion, interpolation in Table 34 gives a required structural number of about SN = 2.84. Since SN = a_1D_1 + a_2D_2, substitution for a_1, a_2, and D_1 gives 2.84 = (0.14)(6) + $(0.093)D_2$. The required subbase thickness is therefore D_2 = 21 in, and the total thickness is 6 + 21 = 27 in for the two layer structure. It is noted that the design is based on a failure criterion of 2 in rut-depth whereas the USACE design may permit as much as a 3 in rut-depth by the end of the design period.

<u>Example 3.</u> <u>Granular-Surfaced Road, Variable Subgrade CBR.</u>

A granular-surfaced road will receive N_{18} = 200,000 equivalent 18,000-lb single axle loads over a 7-year design period. The subgrade soil is a residually weathered clay from sedimentary limestones and shales. Numerous laboratory CBR tests that have been made on this soil have produced the following distribution of CBR values:

CBR Value (%)	3	4	5	6	7	8
Percentage of Tests	0	11	17	25	28	19
Cumulative Percentage	0	11	28	53	81	100

The problem is to select a design value for the subgrade CBR and then to determine the thickness and strength requirements of the surfacing material.

The distribution of CBR strength values reflect natural variability in the subgrade soil. If a design value of CBR = 4% is selected, then an estimated 89% (100%-11%) of the road structure would be overdesigned since less thickness would be required for CBR > 4% than for CBR = 4%. At the other extreme, if CBR = 8% is selected as a design value, then an estimated 81% of the total structure would be underdesigned since more thickness is required for CBR < 8% than for CBR = 8%. The simplest compromise would be to select a design value such that 50% of the test values were below the design value. It is suggested, however that (a) the selected value be such that 60% of the test values exceed the design value and (b) 40% of the test values are less than the design value. For the distribution above, the CBR value that exceeds 40% of the test values is about 5.5%. Thus, if the structure is designed for subgrade CBR = 5.5%, it is expected that 60% of the road will be overdesigned and 40% will be underdesigned.

If the USACE procedure is used for subgrade CBR = 5.5%, then Figure 36 gives a required thickness of about 13 in. Table 30 indicates that the surfacing material should have CBR strength of about 70%. The same procedure gives thickness requirements of about 10 in when subgrade CBR = 8% and about 16 in when subgrade CBR = 4%.

With respect to the distribution of test values, the 13-in thickness represents underdesign for CBR values of 3% and 4%, and overdesign for CBR values of 7% and 8%.

Example 4. Granular-Surfaced Road, Prediction of Distress.

Assume that the 13-in granular surface in Example 3 has been constructed with lateritic material and that ADT = 150 vpd (two-way). The average vertical grade is 15 m/km, and the average annual rainfall is 20 in. Use information that was presented in Chapter 2 to estimate the degree to which various types of distress will be evident at the end of the first year of traffic operations.

According to the Kenya experience represented in Figure 7, the depth of loose material remains at about 1 mm for lateritic gravel and does not change with accumulated traffic. Annual gravel loss is shown in Figure 8 for various combinations of annual traffic, annual rainfall (R_L), vertical curvature (VC), and gravel types. For this example, annual traffic = 150 vpd x 365 or about 55,000 vehicles per year. Annual rainfall is R_L = 20 in = 51 cm, or about 0.5 m. At this level of traffic, Figure 8 shows that about 30 mm of lateritic gravel will be lost when R_L = 1 m and VC = 10 m/km. Neighboring curves indicate that the 30 mm value should be adjusted downward for R_L = 0.5 m, and adjusted upward for VC = 15 m/km, and that the two adjustments will be similar in magnitude. Thus, a rough estimate of gravel loss is about 30 mm, or perhaps between 1.25 in and 1.50 in per year.

Rut depth can be estimated from Figure 10. For annual traffic of 55,000 vehicles, Figure 10 shows that the expected rut depth will be about 23 mm, or approximately 1 in.

Expected road roughness as measured by the TRRL Bump Integrator is given by Figure 11. The figure shows that about 5500 mm/km can be expected after one year of traffic. This level of roughness corresponds to a serviceability index of around 2.5.

In general, it can be stated that the road will be in fair condition at the end of the first year, but that blading is needed to reduce rut-depth and roughness. Moreover, the estimated gravel loss indicates that as much as half the original thickness may be lost within four years of traffic operation. An additional layer of perhaps 2 in of gravel will be needed during the second year of operation. It is therefore implied that the original design (Example 3) was less than adequate to prevent failure before the 7 year design period is completed.

Example 5. Granular-Surfaced Road, Alternative Granular Materials.

A granular-surfaced road is to be constructed on a subgrade having CBR strength of 8%. The road is to withstand 100,000 equivalent 18,000-lb axle-load repetitions before rut-depth reaches 2 in. The problem is to determine required thicknesses for three different granular materials whose CBR strengths are 40%, 80%, and 100%, respectively.

The most direct approach to this problem is the USFS procedure with rut-depth criterion. Table 34 shows that a structural number (SN) of 1.78 is required. Table 32 shows that SN coefficients for the three materials are a_1 = 0.107 (CBR = 40%), a_1 = 0.136 (CBR = 80%), and a_1 = 0.140 (CBR = 100%). Since SN = a_1D_1, the three required thicknesses are D_1 = 16.6 in, 13.1 in, and 12.7 in. Thus, for practical purposes 13-in thickness is needed for either of the two stronger materials, and three to four additional inches are needed if the weakest material (CBR = 40%) is used.

Example 6. Upgrading Granular-Surfaced Road to BST Road.

As part of a stage construction program, the granular-surfaced road in Example 1 is to be upgraded to a bituminous-surface-treated (BST) road after the initial 5-year period. The BST road is to last an additional 15 years and will carry an estimated traffic volume of N_{18} = 500,000 equivalent single-axle loads.

Assume that the USACE design thickness of 7 in has been used for granular surfacing material having CBR = 65%, liquid limit (LL) = 30%, and plasticity index (PI) = 10%. Also assume that the gradation of this material is as follows:

Size	Passing
3/4 in	100%
No. 4	58%
No. 200	23%
0.005 mm	6%

Finally, assume that gravel loss has been 1.5 in over the five-year period of service and that the remaining thickness of granular material is 5.5 in.

The first step is to decide whether the existing granular material is suitable (with increased thickness) as a base course for the bituminous surface treatment. For a 3/4-in maximum size aggregate, it is generally recommended that no more than 15% should pass the No. 200 sieve. Moreover, a high-quality base course should have a minimum value of CBR = 80%; it should also have maximum values for LL = 25% and PI = 6%. Thus it is clear that the existing material is not suitable as a base course, even if greater thickness is provided. The existing material may, however, be used as a subbase course.

If the USACE procedure is used, Figure 36 shows that a total thickness of about 11.5 in is needed for subgrade CBR = 10% and N_{18} = 500,000. Thus 11.5 - 5.5 in = 6 in of granular base material is needed for the upgraded structure. The base material should have minimum CBR = 80%, and about 7%-8% of the material should pass the No. 200 sieve. The bituminous surface treatment should be designed in accordance with information presented in Chapter 7.

If the TRRL procedure is used, Figure 37 shows that the total thickness of the structure need be only 9.5 in. However, the TRRL procedure requires a 6-in minimum base thickness, so the same design would be used as in the USACE procedure above.

Example 7. Environmental Effects on BST Design.

A BST road is to be built on a subgrade soil that is relatively uniform in all characteristics except for ground water conditions. One part of the road has a ground water table close to the road surface; a second part has a deep ground water table but is subject to relatively high rainfall of more than 10 in per year. The third stretch of road will be over a deep ground water table and within an arid climate. The in-place CBR values for these three conditions are 3%, 5%, and 8%, respectively.

If the BST road is designed to accommodate N_{18} = 500,000 equivalent 18,000-lb axle-load repetitions, find the total thickness required for each of the three climatic conditions.

If the USACE procedure is used, then Figure 36 gives required thicknesses as shown in the second column below.

In-Place CBR (%)	USACE Thickness (in)	TRRL Thickness (in)
3	25.7	20
5	18.8	15
8	13.6	11

If the TRRL procedure is used, then Figure 37 gives required thicknesses as shown in the third column above. The TRRL procedure results in lesser thickness requirements than does the USACE procedure. However, it must be remembered that the TRRL procedure requires a minimum of 6-in base thickness with CBR greater than 80%, whereas the USACE procedure does not demand this base strength.

The main point of this example is that nearly twice as much thickness is required for the worst of the environmental conditions (CBR = 3%) than for the best condition (CBR = 8%). The use of soaked strength design values (e.g., CBR = 3%) for all conditions is clearly inefficient when in-place strength data are available.

Example 8. Economics of Granular Materials for a BST Structure.

A BST road is to accommodate 1.0 million equivalent 18000-lb single-axle loads during a 10-year design life. A single surface treatment with maximum aggregate size of 0.75 in will be used. It is assumed that the serviceability index will be 4.0 initially and will reach a failure level of P_T = 2.0 at the end of the design period. The subgrade soil has CBR = 5%.

Base material will be a high-quality crushed stone with CBR = 110%+, and 8.0 in of this material will be used. Two possible materials are available for the subbase layer. The first has CBR = 20%, and the second has CBR = 40%. If 1.0 is the unit price of the first subbase material, then the second material has unit price 1.2. The problem is to determine what thickness is required for the most economical subbase layer.

For the given conditions, the most direct design approach is the USFS procedure with PSI criterion. Table 31 shows that the soil support value for CBR = 5% is SS = 4.0. Table 33 shows that the required structural number is SN = 3.39 for W_T = 1000000 and SS = 4.0.

Structural number coefficients in Table 32 are 0.142 (extrapolated) for the base material, 0.095 for the CBR = 20% subbase material, and 0.120 for the CBR = 40% subbase alternative. Since the surface treatment will be less than 1 in thick, its structural number coefficient is equal to that for the base material, i.e., $a_1 = 0.142$.

The structural number equation is $SN = a_1D_1$ (surface) + a_2D_2 (base) + a_3D_3 (subbase) from which $a_3D_3 = 3.39 - 0.142(0.75) - 0.142(8.0) = 2.15$.

For the first subbase material, $D_3 = 2.15/0.095 = 22.6$ in, and for the second material, $D_3 = 2.15/0.120 = 17.9$ in. Thus, if unit prices were equal, the second material should be used since less thickness would be required than for the first material.

By using the actual unit costs (1.0 and 1.2), the relative costs of the two alternatives are as follows: first subbase material, 22.6 x 1.0 = 22.6 cost units; and second subbase material, 17.9 x 1.2 = 21.5 cost units.

Thus, the second material gives a more economical design even when the unit costs are taken into account. The final design would have nearly 27 in total thickness as shown below:

Layer	Thickness (in)	CBR Strength (%)
Surface treatment	0.75	110+
Base	8.0	110+
Subbase	17.9	40
Subgrade		5

If the USACE procedure were used, Figure 36 shows that a total thickness of about 20 in would be required, and only 11-12 in of subbase would be used.

The TRRL procedure requires a total thickness of only about 16 in (Figure 37), including about 7 in of the stronger subbase. The first subbase material (CBR = 20%) would not meet the TRRL minimum of CBR = 25% for subbase material. This final example again points out that different design procedures can lead to quite different thickness requirements. It can be assumed that much of the variation is attributable to differences in failure criteria, and therefore to different levels of performance expectations for the design period.

As was stated at the outset of Chapter 9, it is essential to observe the performance of roads that have been designed by particular procedures and to modify the procedures as needs become apparent.

References

1 Millard, R. "The Craft of Highway Engineering", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

2 Yoder, E.J., "Introduction", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C., 1975.

3 Yoder, E.J. and Witczak, M.W. Principles of Pavement Design, 2nd Edition, J. Wiley and Sons, New York, 1975.

4 Ahlvin, R.G. and Hamitt, G.M., "Load-Supporting Capability of Low-Volume Roads", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C. 1975.

5 Greenstein, J. and Livneh, M., "Design Thickness of Low-Volume Roads", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

6 Carey, W.N. and Irick, P.E., "The Pavement Serviceability Performance Concepts", HRB, Highway Research Bulletin 250, Washington, D.C. 1960.

7 Hudson, W.R., "Outline of a Generalized Road Roughness Index for Worldwide Use", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Board Record 702, TRB, Washington, D.C., 1979.

8 Haas, R.C.G. and Hudson, W.R., Pavement Management Systems, McGraw-Hill, Inc., New York, 1978.

9 Carmichael, R.F., Hudson, W.R., and Sologuren, C., "Evaluation of Highway Roughness in Bolivia", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Board Record 702, TRB, Washington, D.C., 1979.

10 Rolt, J., "An Investigation into Road Deterioration in Kenya", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C., 1975.

11 Robinson, R., "The Kenya Road Transport Investment Model", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C., 1975.

12 Hide, H., "Investigation of Vehicle Operating Costs in Kenya", Proceedings, Low-Volume Roads, Transportation Research Bord, Special Report 160, Washington, D.C., 1975.

13 Faiz, A. and Staffini, E., "Engineering Economics of the Maintenance of Earth and Gravel Roads", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

14 Abaynayaka, S.W., "Techniques for Measuring Vehicle Operating Cost and Road Deterioration Parameters in Developing Countries", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C., 1975.

15 Visser, A.T., deQueiroz, C.A., and Moser, B., "A Preliminary Evaluation of Paved and Unpaved Road Performance in Brazil", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

16 Ellis, C.I.,"Axle-Load Distribution on Roads Overseas. Survey on Roads in West Malaysia", TRRL Report LR 187, Crowthorne, England, 1968.

17 Ellis, C.I. and Potochi, F.P., "Axle-Load Distribution on Roads Overseas: Abu Dhabi and Qatar 1970-1971", TRRL Report LR 572, Crowthorne, England, 1973.

18 U.S. Army Engineers, "Thickness Requirements for Unsurfaced Roads and Airfields-Bare Base Support", Technical Report S-70-5, USAE-WES, Vicksburg, Mississippi, July 1970.

19 Barber, V.C., Odom, E.C., and Patrick, R.W., "The Deterioration and Reliability of Pavements", Technical Report S-78-8, USAE-WES, Vicksburg, Mississippi, July 1978.

20 Sowers, G.F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, Fourth Edition, MacMillan, New York, 1979.

21 The Asphalt Institute, "Soils Manual for the Design of Asphalt Pavement Structures", Manual Series No 10 (MS-10) College Park, MD, 1978

22 Transport and Road Research Laboratory, "A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries", TRRL Road Note 31, 2nd Edition, London, England, 1977.

23 Transport and Road Research Laboratory, "A Guide to the Structural Design of Pavements for New Roads", TRRL Road Note 29, 2nd Edition, London, England, 1970.

24 Seed, H.B., Richard, W.J., and Raymond, L., "Prediction of Swelling Potential for Compacted Clays", ASCE Journal of Soil Mechanics, Div. 88, SM3, 1962.

25 Seed, H.B., Mitchell, J.K., and Chan, C.K., "Studies of Swell and Swell Pressure Characteristics of Compacted Clays", HRB, Bulletin No. 313, Washington, D.C., 1962.

26 Kassiff, G., Livneh, M., and Wiseman, G., Pavements on Expansive Clays, Jerusalem Academic Press, Jerusalem, Israel, 1969.

27 McDowell, C., "Interrelationship of Load, Volume Change, and Layer Thicknesses of Soils to the Behavior of Engineering Structures", Proceedings, Highway Research Board, Vol. 35, 1956.

28 Mitchell, M.F., Petzer, E.C.P., and Vander Walt, N., "The Optimum Use of Natural Materials for Lightly Trafficked Roads in Developing Countries", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

29 Odier, L., Millard, R.S., dos Santos, P. and Mehra, S.R., "Low Cost Roads-Design, Construction and Maintenance", Butterworth and Co. Publishers Ltd., London, England, 1971.

30 Ellis, C.I., "Pavement Engineering in Developing Countries", TRRL Supplementary Report No. 537, Crowthorne, England, 1979.

31 Morin, W.J., and Todor, P.C., "Laterite and Lateritic Soils and Other Problem Soils of the Tropics", U.S. Agency for International Development Report AID/CSD 3682, Washington, D.C., 1975.

32 Lyon Associates, Inc., "Laterite and Lateritic Soils and Other Problem Soils of Africa", U.S. Agency for International Development Study AID/CSD-2164, June 1971.

33 The Asphalt Institute, "Asphalt Surface Treatments and Asphalt Penetration Macadam", Manual Series No. 13 (MS-13), Second Edition, College Park, MD, 1969.

34 The Asphalt Institute, "The Asphalt Handbook", Manual Series No. 4 (MS-4), College Park, MD, 1968.

35 National Institute for Transport and Road Research, "Structural Design of Road Pavement: Draft TRH 4", Pretoria, South Africa, 1978.

36 Erickson, L.F., "Low-Volume Road Pavements", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C. 1975.

37 Thompson, M.R., "Soil-Lime Mixtures for Construction of Low-Volume Roads", Proceedings, Low-Volume Roads, Transportation Research Board, Special Report 160, Washington, D.C., 1975.

38 Robbins, E.G. and Packard, R.G., "Soil-Cement: A Construction Material", Proceedings, 2nd International Conference: Low-Volume Roads, Transportation Research Record 702, TRB, Washington, D.C., 1979.

39 U.S. Forest Service, "Transportation Engineering Handbook," Chapter 50, Pavements, U.S. Department of Agriculture, Washington, D.C., 1974.

40 Roberts, F.L., McCullough, B.F., Williamson, H.J., and Wallin, W.R., "A Pavement Design and Management System for Forest Service Roads A Working Model", University of Texas, Austin, Research Report No. 43, Final Report Phase II, February 1977.

41 McCullough, B.F. and Luhr, D.R., "LVR Pavement Design and Management System for Forest Service Roads - Implementation", University of Texas, Austin, Research Report No. 60, Final Report Phase III, January 1979

42 McCullough, B.F. and Luhr, D.R., "LVR Pavement Design and Management System User's Manual", University of Texas, Austin, Research Report No. 60, Final Report Phase III, January 1979.