SUBBASE DESIGN MANUAL FOR PORTLAND CEMENT CONCRETE PAVEMENTS

by

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prepared for

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Oakland, California

November, 1970

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SECTION 1

INTRODUCTION

§ 1.0 - General

This is the second in a series of manuals prepared for use in designing continuously reinforced concrete pavements (CRCP). The first manual, prepared for the United States Steel Corporation (Ref 1), entitled "Design Manual for Continuously Reinforced Concrete Pavement," by B. F. McCullough, describes a comprehensive set of design methods for continuously reinforced concrete pavements. It briefly discusses subbase materials and their application to the design of CRCP, and recommends the evaluation of the subgrade modulus of reaction by use of a plate load test, preferably utilizing a 30-inch diameter plate. The k value for the subgrade so determined must be used in design whether the fatigue method (Ref 1, Fig 2.1-1) or the static load method (Ref 1, Figs 2.1-2 and 2.1-3) is employed. Little attention, however, is given in the CRCP Manual to the improvement of k by addition of various subbase layers.

The purpose of this subbase manual is to supplement the CRCP Manual and to establish a rational design method for subbase layers to be used with concrete pavements. When used in conjunction with the CRCP Manual, this manual can provide the designer with tools for developing pavements with excellent load carrying capabilities.

§ 1.1 - Concepts of Subbase Design

The performance of a CRCP structure is related to the supporting power and physical characteristics of the natural soils. Poor soil support conditions may be offset by increasing the concrete slab thickness or by the introduction of an intermediate layer of material. The physical characteristics of some soils are such that an intermediate or subbase layer must be used to prevent erosion.

It should be understood that the design of an economical pavement cannot be accomplished piecemeal. If the designer designs the pavement thickness for a specific support value k without investigating the thickness required for other k values and comparing the pavement costs alone against the cost of providing better subgrade support and thinner slabs, he will not determine the most economical pavement design. Unfortunately, many pavements have been so designed and, as a result, not enough attention has been given to the economical design of subbase layers.

Although there are many variables involved in the design of a CRCP slab, many of these variables can be lumped together for effective consideration. Thus, given the subgrade soil conditions, loads to be carried and performance expected, the pavement designer can lump the remaining variables into two categories, (1) those associated with the slab and (2) those associated with the subbase layer. By running a few simple trial solutions, the designer can compare the cost of a slab of specified thickness, strength, and modulus of elasticity and its associated subbase layer, with other combinations of slab and subbase. It is normal procedure in many localities where concrete properties are essentially fixed by the use of locally available materials to compare the reduction in slab thickness possible with an increased k value. The goal is to find the most economical combination of materials to carry the required load.

The methods outlined in this manual are based on the Winkler Foundation Model used by Westergaard (Ref 2) and Hudson and Matlock (Refs 3 and 4) and on layered theory. Additional empirical information available from the AASHO Road Test (Ref 5) as well as field experience with continuously reinforced concrete pavements is utilized to investigate the account for the ability of some subbase materials to maintain their strength throughout the life of the pavement while the load carrying capacity of other materials deteriorates rapidly under repeated loads.

§ 1.2 - Background on Subbase Performance

A number of the presently available design methods allow for measuring a support value at the top of a subbase layer or provide a means for estimating the value. However, these approaches do not adequately describe the ability of the system to maintain its strength and integrity under repeated loading and pumping conditions. There is a great deal of information available, however, to assist in evaluating this factor empirically. Prior to 1958, the use of granular subbases "to prevent pumping" was almost universal in the United States. With this in mind, the AASHO Road Test rigid pavement experiment (Ref 5) was designed to study the thicknesses of granular subbase layers and their effects on pavement performance under repetitive loads. A smaller experiment was included to compare the performance of pavement constructed directly on a fat clay subgrade ($k = 40\text{-}50$ pci) with pavements constructed on 3, 6, or 9 inches of coarse gravel subbase material ($k = 100\text{-}120$ pci). In these studies, all except the thickest slabs pumped to some degree for each loading condition. The so-called "non-pumping" subbase material pumped very badly in combination with thinner slabs under all loads. The sections with a subbase did perform considerably better however than those without a subbase. For example, the life of sections with a subbase was on the average one-third longer than comparable sections without a subbase (Ref 5, pp 160-161). In general, at the Road Test the difference in the amount of pumping, and the pavements' resulting performance was much greater than could be estimated by the difference in the k value alone. The change in k from 45 to 110 would have increased performance for an 8-inch pavement from 2.2×10^6 to 2.9×10^6 equivalent single axle load applications according to the AASHO Interim Design Guide (Ref 17), when in reality an increase in performance from 1.43×10^6 to 2.95×10^6 equivalent single axle loads was gained by adding granular subbase on top of the clay for the 8-inch pavement (Ref 5). According to the AASHO Interim Design Guide a change in k from 45 to 500 would have been required to produce this change in observed performance, with all other factors held constant. The resulting difference in performance, then, can be evaluated as the increased ability of the granular subbase to retain its strength and integrity over the life of the pavement.

There is little known quantitative data relating to the erosion or pumping of various types of stabilized subbase layers under CRCP. A number of observations by the authors over the last 10 years, however, can be used to evaluate pavement performance qualitatively. During this time no known cases of pumping or erosion of tar-stabilized or asphalt-stabilized materials were observed. These materials are waterproof and therefore resistant to the erosive action of water. Materials treated with Portland Cement, when properly designed, constructed, and cured, have not eroded under pumping action. Therefore, tar-treated, asphalt-treated, or cement-treated materials are preferred for use as subbase layers.

Lime-stabilized materials, usually heavy clays, have eroded under certain combinations of heavy load and severe moisture conditions. This can sometimes be prevented by placing an asphalt surface treatment between the lime-stabilized layer and the pavement slab. Such a design, however, is considered inferior to a tar-stabilized, asphalt-stabilized, or portland cement-stabilized layer.

§ 1.3 - Scope of Manual

This report is intended as a manual for current use in the design of subbases, with particular attention to their use in continuously reinforced concrete pavements. As additional information is made available through research and study, the manual should be upgraded or replaced by more complete information, but until such information is available, this manual should prove useful to the designer.

The balance of this manual is divided into five major sections. Section 2 provides an evaluation of subbase materials, with particular emphasis on stabilized materials, and provides a general reference for selecting materials for use in design. Section 3 outlines the design of the subbase layer and provides supporting information for the development of a rational method for evaluating subbase support strength, based on available experience and theory. Section 4 supports the basic report with specific examples and charts for the use by the designer. A step-by-step procedure is provided. Section 5 summarizes the report and the method. The final section is a set of appendicies providing detailed information about various aspects of the problem which may be useful to the designer.

SECTION 2

EVALUATION OF SUBBASE MATERIALS

§ 2.0 - Introduction

The pavement designer must consider selection of materials for use as a subbase. The choice is usually between a high-quality granular material and a stabilized material, either the natural material stabilized with an additive or some sort of a stabilized granular material. This section provides a brief general description of these types of materials and their properties. References are provided to use in obtaining details of the design of the specific soil mixture. Section 3 presents an overall method for evaluating the effect of the selected subbase on the pavement design.

While raw granular materials are often used as subbases, the need for improved pavement performance and the improvement of substandard construction materials have led to the increased use of stabilized materials. Many methods of stabilization have been developed; the more extensively used are those classified as chemical stabilization. Of the various chemical stabilizers in use, asphalt, tar, cement, and lime are the most widely used and will be discussed herein. However, other methods may be used if evaluated properly.

§ 2.1 - Granular Subbase Materials

Granular subbases are usually composed of high-grade aggregates, such as crushed slag, crushed stone, gravel, and sand. The material should be well graded to provide good compaction, but the fines should be held to a minimum to promote good drainage. Maximum aggregate sizes depend on layer thickness, but may range up to 2 inches without causing difficulty. Since a major problem with granular materials is pumping, as exhibited at the AASHO Road Test, it is essential that trench construction be avoided if at all possible. Almost any good quality, well-compacted granular material will improve the performance of concrete pavements. However, if large numbers of heavy loads are expected during the life of the pavement, the value of stabilized materials should be investigated.

Strength. Granular subbases do not exhibit any tensile strength capabilities. Therefore, compressive strength is generally used as a guideline for judging quality. Generally, the compressive strength for most adequately compacted granular materials will range between 20 and 150 psi.

Elastic Properties. The stiffness or resilient modulus of granular materials will vary from 8000 psi to 30,000 psi depending on the character of the material (Ref 11). Poisson's ratio for these materials will generally range from 0.40 to 0.45.

Erodability Factor. The erodability factor for granular materials will depend to a large extent on the amount of water present in the subbase layers. With heavy traffic, these materials will exhibit some degree of pumping leading to a loss of performance as experienced at the Road Test (Ref 5). It is recommended that an Erodability Factor of 3.0 be used for fine grained materials and 2.0 to 2.5 be used for materials having larger percentages of coarse aggregates. (For an explanation of erodability factor refer to Appendix D.)

§ 2.2 - Bituminous Stabilization (Asphalt and Tar)

Bituminous stabilization may be achieved by using either asphalt or tar mixed with a wide range of soils, varying from relatively clean, coarse-grained materials to clays. The actual function of the bituminous material is dependent on the type of soil being stabilized. For coarse-grained soils possessing little cohesion, the primary function of the bituminous stabilizer is to provide cohesion by cementing the soil particles together, and at the same time provide waterproofing. For cohesive soils, on the other hand, the primary function of the bitumen is to serve as a waterproofing agent.

§§ 2.2-1 General. Three types of bituminous stabilized mixtures can be used for subbase materials: soil bitumens, sand bitumens, and sand-gravel bitumens.

A sand-bitumen mixture includes loose sand, which has little cohesion and little strength unless confined. The function of the bituminous material in this case is to provide cohesion and, thus, increased strength. This type of mixture is used extensively since sands suitable for stabilizing are found throughout the world. By employing various types and grades of bituminous binders, a range of strengths can be obtained.

The sand-gravel bituminous mixture is a system in which a well-graded material is waterproofed and provided with cohesion by the uniform distribution of small amounts of bitumen. Available deposits of gravel may be used in such mixtures, with fine and coarse aggregate added as needed. These mixtures are of intermediate to good quality when satisfactorily constructed.

The soil-bitumen mixture primarily involves cohesive soils which have satisfactory bearing capacity at low moisture contents. Since this type of soil tends to lose stability at high moisture contents, the bituminous materials serve as waterproofing agents to maintain the low moisture content. The bituminous material blocks or plugs the capillary pores in the material and forms a partially protective film around the soil aggregation.

§§ 2.2-2 - Properties of Bituminous-Treated Materials. The properties of bituminous-treated soil depend on many factors, the most important of which are type of soil, type of bituminous material, temperature, and quantity of bituminous materials.

Strength. The addition of a bituminous material may increase or decrease the strength of the material. For treated cohesionless soil, the strength is increased. In high-quality mixtures approaching hot mix asphalt concrete, the unconfined compressive strength varies from 100 to 600 psi, depending on quality of materials, temperature, and loading rate. In lower quality mixtures, the unconfined compressive strength can be expected to range from 100 psi to 250 psi.

The strength of a well-graded cohesive soil decreases with the addition of small quantities of asphalt, provided that the mixture is relatively dry. If, however, the specimen is allowed to absorb water after treating, the resulting loss of strength will be less for those specimens treated with bitumen (Ref 18). For poorly graded soils, a decrease in strength becomes evident only at relatively high asphalt contents.

The tensile strength of asphalt-treated granular soils may be as high as 200 psi, although a more common range is 50 to 150 psi (Ref 18). For other types of bituminous-treated materials, the tensile strengths are much lower, in the range of 20 to 30 psi, and probably can be estimated to be approximately 10 percent of the unconfined compressive strength (Ref 18).

Elastic Properties. The stiffness for asphalt-treated base materials ranges from 350,000 psi - 1,000,000 psi (Ref 18) with the lower value recommended for warm climates. For areas where high temperatures are not experienced, a larger value may be used. The stiffness values for asphalt emulsion-treated

Modulus of Elasticity:

$$E' = (1{\cdot}50504 - 1{\cdot}58707\,T' + 1{\cdot}43003\,V' - 1{\cdot}34341\,V'T' + 1{\cdot}84210\,A'_s$$
$$- 57753\,G'V'A'_c\,T') \times 10^5$$

where

$$T' = \frac{T-100}{25}$$

$$V' = \frac{V-2}{3}$$

$$A'_s = A^2 - \frac{17}{24}$$

$$G' = \frac{G-4}{2}$$

$$A'_c = \frac{A_c - 7}{1{\cdot}5}$$

where symbols are defined as follows:

T = temperature of mix for design condition, $^{\circ}$F.

V = viscosity of coal tar.

A = aggregate type using coding values as follows:
- -1 for rounded river gravel
- 0 for slag material
- +1 for crushed stone with surface texture.

G = gradation of aggregate using coding values as follows:
- -1 for fine gradings
- 0 for medium gradings
- +1 for coarse gradings.

A_c = coal tar content for values between 4.5 - 9 percent by weight.

Fig 2.2-1. Procedure for estimating stiffness of a coal tar concrete for various mix variation.

bases are generally lower ranging from 40,000 to 300,000 psi (Ref 18).

The stiffness value for tar-stabilized bases may be computed using the equations presented in Fig 2.2-1 that were developed from a regression analysis of laboratory data. Specifications for coal tar is presented in Appendix C.

Erodability Factor. The erodability factor of bituminous treated bases is a function of the amount of bitumen used. It is recommended that a value of zero to 1.0 be used. If sufficient bitumen is used for the treated mixture to retain its structural integrity in the presence of water, a value of zero may be used, otherwise higher values must be used.

§ 2.3 - Cement Stabilization

Cement stabilization consists of treating soil with small amounts of portland cement plus water. The addition of portland cement to a soil usually results in a material with engineering characteristics which are significantly improved as compared to the original properties of the soil. In general, these changes are:

(1) reduced plasticity indices,
(2) increased plastic limits,
(3) relatively unchanged liquid limits,
(4) increased shrinkage limits,
(5) increased strengths,
(6) reduced volume changes,
(7) reduced permeabilities,
(8) increased effective grain size distribution.

§§ 2.3-1 - General. Three types of cement-stabilized soils are commonly used. These three types differ primarily in terms of their function and intended use and are designated as compacted soil-cement, cement-treated or modified soil, and plastic soil-cement.

Compacted soil-cement contains sufficient cement to harden the soil and enough moisture for adequate compaction and hydration of the cement. The properties of soil-cement may differ greatly from those of the untreated soil and, in general, the primary purpose for using compacted soil-cement is to provide permanent increased strength.

Cement-treated soil is an unhardened or semi-hardened mixture of soil and cement in which small quantities of cement are used in order to modify certain undesirable characteristics of the untreated soil, such as plasticity and volume change. Unlike compacted soil-cement, the quantity of cement in cement-treated materials is not sufficient to produce a substantial hardening or strength gain although some strength gain will occur.

Plastic soil-cement is also a hardened mixture of soil and cement which, in contrast to compacted soil-cement, contains sufficient water at the time of placing to produce a mortarlike consistancy. As such, it is a special purpose material not usually used for subbases and will not be considered in any further detail here.

§§ 2.3-2 - Properties of Cement-Treated Soils. The properties of cement-treated soils depend on many factors, the most important of which are the type of soil, cement content, age of the compacted mixture, the moisture content at time of compaction, and the degree of densification. It has generally been found that cement-treated materials should be compacted to a density equal to or greater than 95 percent AASHO (Mod or Std) at a moisture content equal to or slightly greater than optimum.

Strength. Many methods exist for evaluating the strength characteristics of soil-cement. The most common method is the unconfined compression test. Table 2.3-1 contains typical values of unconfined compressive strength for three textural soil groups with minimum cement contents satisfying accepted criteria for soil-cement as determined by wet-dry and freeze-thaw tests. Strength may be lower or significantly higher depending on the cement content and age. Table 2.3-2 contains an upper level of compressive strength for four different types of soil. It can be expected that the strength at lower cement contents will vary linearly with concrete content and that strength will increase at a decreasing rate with an increase in age. This relationship is similar to the strength gains curve for concrete.

The flexural test is used as a measure of tensile or flexural strength with results expressed in terms of the modulus of rupture. As seen in Table 2.3-2 the modulus of rupture for granular soil-cement may be as high as 260 psi after 28 days of moist curing and as high as 100 psi for a fine-grained soil. For

TABLE 2.3-1. TANGES OF UNCONFINED COMPRESSIVE STRENGTHS OF SOIL-CEMENT

Soil Type	Wet Compressive Strength (psi)	
	7-Day	28-Day
Sandy and gravelly soils: AASHO groups A-1, A-2, A-3. Unified groups GW, GC, GP, GF, SW, SC, SP, SF.	300-600	400-1,000
Silty Soils: AASHO groups A-4 and A-5. Unified groups ML and CL.	250-500	300-900
Clayey Soils: AASHO groups A-6 and A-7. Unified groups MH and CH.	200-400	250-600

TABLE 2.3-2. ILLUSTRATIVE VALUES OF THE ELASTIC AND STRENGTH PROPERTIES OF SOIL-CEMENT MIXTURES

Soil	Cement Content, Percent	Values at 28 days, Moist Cure, psi			
		Compressive Strength	Modulus of Rupture	Modulus of Elasticity E_d	E_{se}
Sand (Soil 1)	3.8	450	110	2.05×10^4	...
	6.0	800	180	2.75	...
	8.5	1225	260	3.30	...
Sandy loam (Soil 2)	3.8	300	80	1.40	0.90×10^4
	6.1	650	145	2.00	1.25
	8.6	1025	215	2.60	1.65
Clayey sand (Soil 3)	5.7	475	105	1.30	...
	8.3	625	150	1.50	...
	11.0	800	195	1.75	...
Silt loam (Soil 4)	8.0	525	125	0.90	0.55
	11.1	725	155	1.05	0.65
	14.2	900	190	1.25	0.75

estimation purposes, the modulus of rupture can be assumed to be approximately 20 percent of the compressive strength.

Elastic Properties. Measurements of the four types of soils shown in Table 2.3-1 indicate that the static modulus of elasticity in compression at 28 days may be as high as 2,000,000 psi for cement-treated sandy soils and 1,000,000 psi for silty soils (Ref 19). Other studies have indicated that the stiffness values for cement-stabilized bases range from 500,000 to 1,000,000 psi (Ref 8). Since cement-stabilized materials do not have a healing capability after cracking occurs, the lower range of values is recommended.

Poisson's ratio values determined dynamically range from about 0.20 to 0.27 for granular soils, 0.30 to 0.36 for clayey soil, and 0.24 to 0.31 for silty soil. Poisson's ratios determined from triaxial test strains between 10 to 90 percent of ultimate strength average 0.14 for cement-treated sandy soil mixtures and 0.12 for cement-treated silty soil mixtures.

Erodability Factor. As was the case for bituminous-treated materials, if sufficient cement is used to retain structural integrity in the presence of water, an erodability value of zero may be used. For cement contents less than three percent by weight a value of 0.5 is recommended.

Plastic Properties. The first noticeable property change that occurs when cement is mixed with moist cohesive soils is a marked reduction in plasticity. Normally, cement changes the plasticity index by increasing the plastic limit (Ref 8). Cement may also change the liquid limit, but normally this is to a lesser degree. Liquid limits usually are reduced if the original limit was greater than 40 and increased if the original limit was less than 40.

§ 2.4 - Lime Stabilization

Lime has been used successfully for stabilizing fine-grained and granular soil materials. The addition of lime to soil usually changes the properties in the following ways:

(1) reduced plasticity indices,
(2) increased plastic limits,
(3) relatively unchanged liquid limits,
(4) increased effective grain sizes,
(5) increased strengths,

(6) reduced volume changes,
(7) reduced permeabilities,
(8) decreased maximum dry densities,
(9) increased optimum moisture contents.

§§ 2.4-1 - General. Lime stabilization can be divided into two categories based upon the purpose. Past use of lime-treated soil has been to improve plasticity and workability, characteristics which occur very soon after the lime and soil are mixed. Typical applications include subgrade, subbase, and base course stabilization through modification of their plasticity and workability characteristics, and use as a dry agent.

The second category includes the improvement of such characteristics as strength and volume changes. These changes require a longer period of time than the changes in the first category.

The above changes are attributable to one or more of a number of physical-chemical reactions which can occur between the lime and soil. Some of the reactions occur very quickly, such as those affecting the plasticity characteristics of soil, while the others tend to be long-term in nature.

§§ 2.4-2 - Properties of Lime-Soil Mixtures. The properties of lime-treated soils depend on many factors, the most important of which are type of soil, percent lime, and time of curing.

Strength Properties. The strengths of lime-treated materials vary widely, depending primarily on the above mentioned factors. Probably the most important factor influencing strength is soil type, since soils vary in their ability to react with lime and in some cases, may actually be nonreactive.

The major effect of lime on the shear strength properties of a reactive fine-grained soil is a substantially increased cohesion with some minor increase in apparent angle of internal friction. After only one day of curing at 120° F cohesion values have been shown to increase by 500 percent.

Unconfined compressive strength may be expected to increase by as much as 1,000 percent with actual values ranging up to 100 psi or higher. Tensile strengths normally range from one-tenth to one-eighth of the unconfined compressive strength.

Elastic Properties. Lime stabilized materials normally exhibit a brittle type stress-strain relationship with a limited amount of inelastic yielding. The modulus of elasticity for these materials may be as much as 25 times that of the untreated soil and with values in the range of 30,000 to 160,000 psi.

Erodability Factor. If the concrete pavement is placed directly on top of the lime stabilized layer, an erodability value of 1.0 to 2.0 is recommended. Lime stabilized materials will sometimes lose their structural integrity in the presence of water and pumping action, hence these higher values are recommended.

Plasticity and Grain-Size. When lime is added to soil, the first effects involve its apparent grain-size distribution and its plasticity characteristics.

Lime causes the soil particles to flocculate or agglomerate, producing an apparent increase in grain-size and a more pliable, workable soil. Some important factors affecting this agglomeration of soil particles are soil type and the amount and type of lime used. Plastic soils agglomerate more readily than do silts, sands, and coarse-grained soils. In addition, the amount of agglomeration increases as the amount of lime increases and it appears that quicklime is more effective than hydrated lime.

For nearly all soils, the plasticity index is reduced with the addition of even small quantities of lime. This reduction is primarily due to an increase in the plastic limit while the liquid limit remains relatively unchanged. Plastic soils exhibit the largest reduction in plasticity, amounting to as much as 50 to 80 percent. In many cases, the soil may actually become nonplastic with as little as 3 percent lime. If a soil remains plastic, further reductions in the plasticity index may be achieved by increasing the lime content although the first increments of lime are the most effective and only minor changes occur after some optimum value is exceeded.

The shrinkage limit is also significantly increased and volume changes inhibited by the addition of small amounts of lime. In fact, treatment with as little as one percent lime may be effective with certain soils. Quantities in excess of about 5 percent probably will produce little additional benefit.

The combined effect of reduced plasticity characteristics and increased effective grain size distribution results in a more workable material.

§ 2.5 - Selection of Stabilizing Agent

There are no specific guidelines that can be presented as to the optimum stabilizing agent for a given subbase material. The compatability of a given subbase material and a stabilizing agent can be reliably established only by testing. Figure 2.5-1 can be used as a general guideline for establishing several possible stabilizing agents for consideration. Starting with the plasticity index of the minus 40 particles in the raw subbase material and proceeding vertically to the appropriate gradation classification, a possible stabilizing agent may be established.

§ 2.6 - Desirable Properties of Stabilized Materials

In the design of a treated or untreated base or subbase material certain properties of the material are desirable. These properties are

(1) workability
(2) stability
(3) durability
(4) flexibility
(5) fatigue resistance
(6) permeability
(7) tensile strength.

§§ 2.6-1 - Workability. In order to achieve desirable properties of the treated material, it is necessary that the stabilizing agent can be satisfactorily mixed with the soil and that the resulting mixture can be placed. In addition, this must be possible economically. This is one of the primary factors restricting the use of stabilizing agents with certain soils. This property may be established during the mixing operations for the laboratory testing, although caution must be used in extending these results to full scale mixing operations.

§§ 2.6-2 - Stability. Stability may be defined as strength or as resistance to deformation under load where the deformation is considered to be permanent distortion. This property is important for the material to resist shearing stresses. It is especially applicable to the bituminous-stabilized layers. Generally,

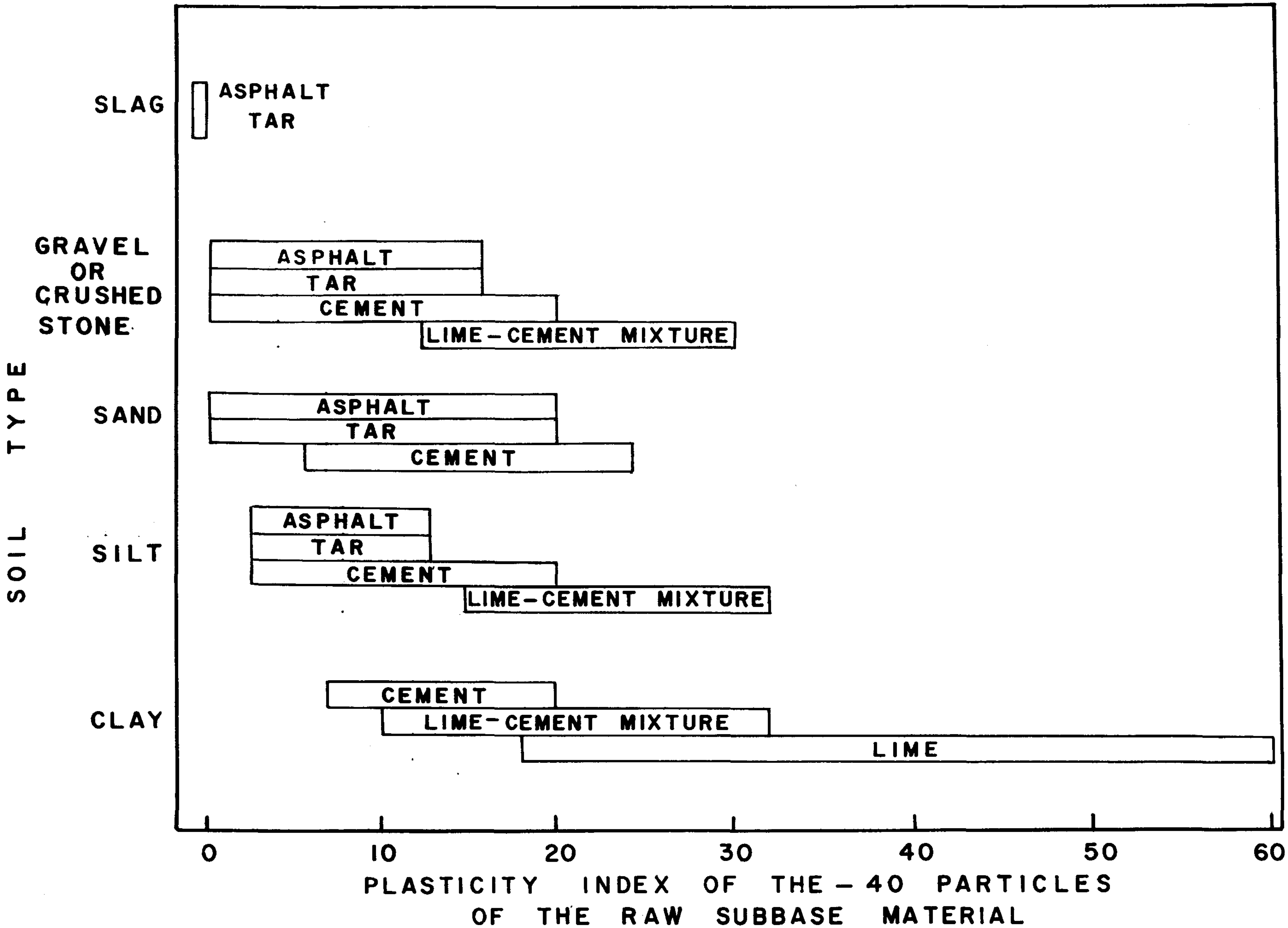

Fig 2.5-1. General guideline for selecting possible stabilizing agents for subbase materials.

stability values as measured by the Hveem (Ref 20), Marshall (Ref 20), Hubbard Field (Ref 20), etc. are included in the specifications to protect against distortion. The use of the same stability values called for with a surface layer, e.g., Hveem value 35 for asphalt concrete, may be overly conservative since the temperatures and load pressures are lower. Hence, the use of these specifications without modifications may require an excessive expenditure of funds.

§§ 2.6-3 - Durability. Durability refers to the ability of a material to resist change through weathering, which includes temperature, moisture, freezing, and thawing. Good durability is especially important in cold climates. Several ASTM tests are available for establishing the durability of aggregates and mixtures. These tests should be used in some form in qualifying aggregate sources, but again caution is urged since the subbase layer is not subjected to such severe performance conditions as the surface layers. The freeze-thaw test (ASTM) is an example of a test that may be used to establish durability criteria.

§§ 2.6-4 - Flexibility. Flexibility is defined as the ability of a mixture to conform to or withstand long-term variations in subbase and subgrade elevations. This characteristic is probably more important for lime-treated and cement-treated materials than for asphalt-treated materials. The first two materials must be able to withstand soft spots by bridging and/or conforming to the disturbed supporting materials without failing prematurely.

§§ 2.6-5 - Fatigue Resistance. Fatigue resistance is the ability of a material to resist many repetitions of load without failing. All materials suffer some fatigue when subjected to many applications of stresses or strains much smaller than their ultimate failure values. Although the number of load applications which a given material can withstand decreases as the magnitude of the applied stresses and strains increase, the actual value or the fatigue resistance of a material is dependent on many mix design factors.

§§ 2.6-6 - Permeability. Permeability is defined as the rate at which air and water can pass into or through a material. Generally, low permeabilities are desirable in order to increase the durability of the stabilized subbase and to prevent surface water from percolating through the pavement to the underlying components.

§§ 2.6-7 - Tensile Strength. Tensile strength is important when the application of heavy loads to the pavement is being considered and when the material underlying the stabilized subbase is comparatively weak. It is also an important factor in the durability of the stabilized material when subjected to moisture, temperature variations, freezing and thawing, and volume changes.

SECTION 3

DESIGN OF SUBBASE LAYER

§ 3.0 - Introduction

As pointed out in Section 1, the economical design of a continuously reinforced concrete pavement cannot be accomplished in a piecemeal fashion. Thus, the selection of a subbase layer depends on many economic factors, such as material availability and cost, construction costs and design factors, such as existing subgrade soil conditions, loads carried, performance expected, and concrete slab variables. In order to simplify the problem for the average designer, however, the effects of most of these variables can be lumped together and are often fixed, (e.g., subgrade soil conditions and loads to be carried at a particular site). With the major effect of these variables fixed, the interaction of slab and subbase can be considered. It will normally involve a comparison of subgrade support value k with slab thickness, with concrete strength and modulus of elasticity fixed. If, however, concrete with a variable modulus is available, then an economic study including the modulus of elasticity of the concrete versus the subgrade support value can be made for various slab thicknesses. The goal is the most economical combination of materials which will provide the required load carrying capacity.

The remainder of this section discusses the design of the subbase layer. The procedural aspects of the design system are outlined in general terms in 3.1, and 3.2 and 3.3 pertain to the measurement of pertinent properties for the natural soil and subbase materials. The composite k value of the subbase system is developed in 3.4, and 3.5 covers the economic comparison and selection of the most economical design.

§ 3.1 - Design System

The step-by-step procedure for the design system is shown, as a flow-chart, in Fig 3.1-1. Where applicable the appropriate design chart is listed. The solid lines indicate that the charts are contained in this manual, i.e., Steps 1-6 and 9-10, and dashed lines indicate steps, i.e., Steps 7-8 that must be

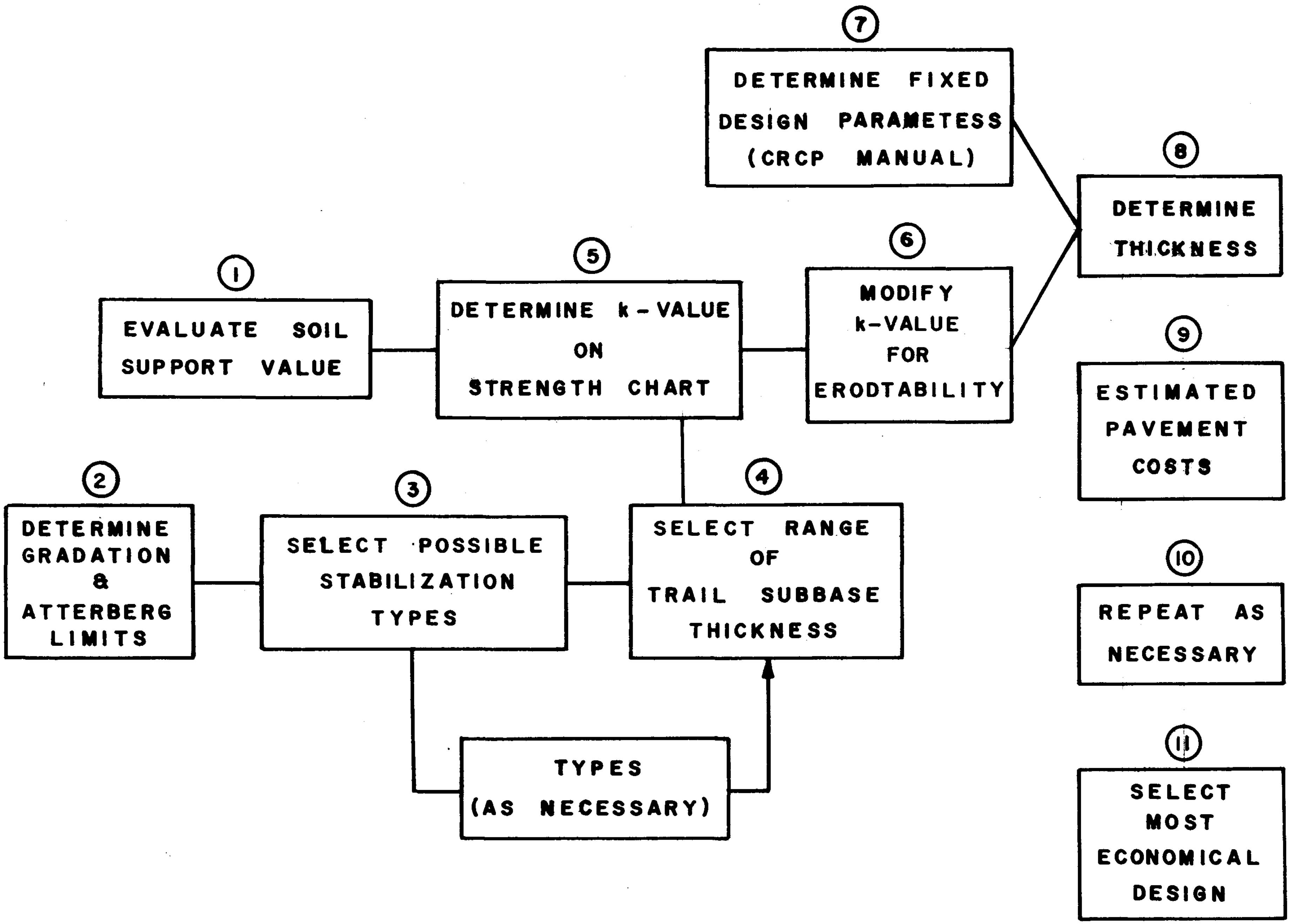

Fig 3.1-1. Approach for pavement design problem.

accomplished through the use of a supplemental manual, such as the CRCP Design Manual (Ref 1). Although the example problems in this manual specifically use the CRCP Manual, any design manual requiring a k value for pavement thickness design may be used.

The design approach is as follows:

(1) Evaluate subgrade support modulus of the natural material.

(2) Ascertain the gradation and Atterberg limits (plasticity index) of the materials being considered for the subbase layer.

(3) Select possible stabilization types for each subbase material, based on the information in Section 2 and a cost per square yard per inch of thickness.

(4) Select a range of trial subbase thicknesses based on minimum and maximum thicknesses derived from construction limitation, agency administrative requirements, etc.

(5) Using the data from Step 1, determine a composite k value for the layers from Fig 3.4-1.

(6) Modify the k value for use in design, based on the erodability characteristics of the subbase material.

(7) Determine fixed design parameters including loads to be carried and performance expected, concrete modulus of elasticity, and concrete strength (modulus of rupture), as outlined in Ref 1.

(8) Using the information from Steps 6 and 7, determine a thickness for the concrete pavement.

(9) Estimate pavement costs for this design configuration.

(10) Repeat Steps 1 thru 8 for other design configurations.

(11) Compare the resulting costs and select the most economical design or make other trial designs which promise better economics.

§ 3.2 - Evaluating Existing Soil Support

The modulus of subgrade support or reaction k must be evaluated for the existing material. As described in the CRCP Design Manual (Ref 1) this value represents the soil as an elastic spring with units of pounds per square inch per inch of deflection (psi/in) or pounds per cubic inch (pci). The value can best be determined by the use of a plate load test with a 30-inch-diameter plate, as discussed in Appendix A. There are many procedures for evaluating this modulus of reaction, as discussed in Refs 6 and 7.

If plate load test equipment is not available or if funds are not available to have such tests performed commercially, it may be necessary to estimate these values from other soil tests. Reference 8 gives a variety of information on subgrade modulus with relation to the Unified Soil Classification System, the R-value Test, the CBR Test, the FAA Soil Classification System, and the AASHO Soil Classification. Other information can be obtained from deflection measurements (Refs 9 and 10).

As an alternative to determining the k value through correlations with agency soil tests, a resilient modulus test may be performed on the natural soil and the resulting value entered directly in the design charts. The procedure for performing the M_r test is briefly described in Appendix B and more detailed information may be obtained from Ref 101. Figure 3.2-1 is a correlation chart showing the relationship between the k value and M_r test result.

§ 3.3 - Select Subbase Stabilization

The problem of selecting an optimum subbase is not as complicated as it first may seem. Based on knowledge of materials and costs in a particular locality, the designer should select for trial one or more preliminary subbase designs, based on the information in Section 2. The information may be used to establish preliminary values or ranges of stiffness (modulus of elasticity) and the erodability factor. The choices of subbase types will depend on the availability of local materials as well as the cost of stabilizing agents and materials processing, such as selective grading for natural subbases or mixing for stabilized materials. After some preliminary investigation, a set of unit costs for various processed subbase materials in-place can be developed for design use.

When frost action may affect pavement performance fine-graded materials such as very fine sands, silts and clays should be avoided or held to a minimum in natural materials. Materials which are stabilized are not as susceptable to this frost action as are natural materials. Additional information for design in frost susceptable areas can be obtained from Refs 12 thru 16.

§ 3.4 - Evaluating the Composite Pavement Support Modulus

There are two important factors to be considered in evaluating the strength of the proposed subbase-on-subgrade combination. These are: (a) improved

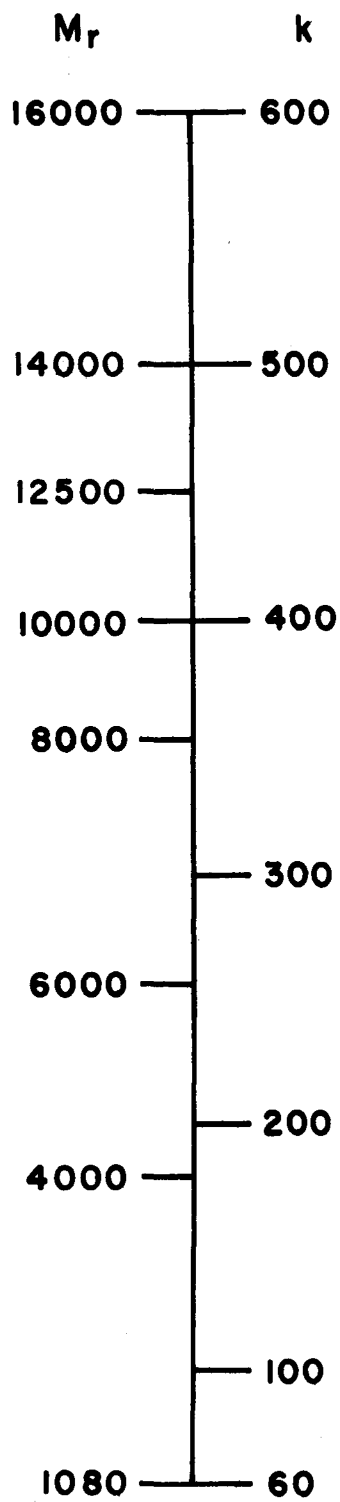

Fig 3.2-1. Correlation between resilient modulus and subgrade support value.

support strength of the layered system and (b) the capability of a layered system to maintain its strength and integrity under the pounding of heavy highway traffic in the presence of moisture.

The effect of the composite k value due to the layered effect of the pavement structure may be accounted for as described in § 3.41. In using this approach, the designer assumes the material does not lose its integrity due to water erosion. Since most untreated materials lose part of their integrity during their service life due to pumping, consolidation, erosion, etc. this effect should be considered in design. Therefore, § 3.42 is included to allow the designer to qualitatively correct the k value based on loss of subbase support.

§ 3.41 - Effect of Layered System

The design chart for evaluating the effect of the layers in a pavement structure is shown in Fig 3.4-1. The material parameters required in this analysis are the stiffness of the subbase material as derived from § 3.3 and the support modulus as determined from § 3.2. The designer begins with the assumed subbase thickness and projects horizontally to the subbase stiffness, then vertically to the appropriate value of subgrade support. The corrected k value at the top of the subbase is then read on the upper vertical scale.

If more than one material is being used for the subbase layer, the designer may take this into account by applying this procedure for each layer. The first time through gives the corrected support value at the top of the first layer. With this value and the thickness and stiffness of the next layer, a new k value at the top of the next layer is determined. This process is repeated until the k value immediately below the concrete pavement is obtained.

§ 3.42 - Correction for Erodability

The influence of material erodability on the long-range characteristics of subbase support may be evaluated by using Fig 3.4-2. The layered k value from Fig 3.4-1 is projected from horizontal axis to the erodability factor. The composite k value will always be equal to or less than the layered k value, with any reduction depending on the material quality.

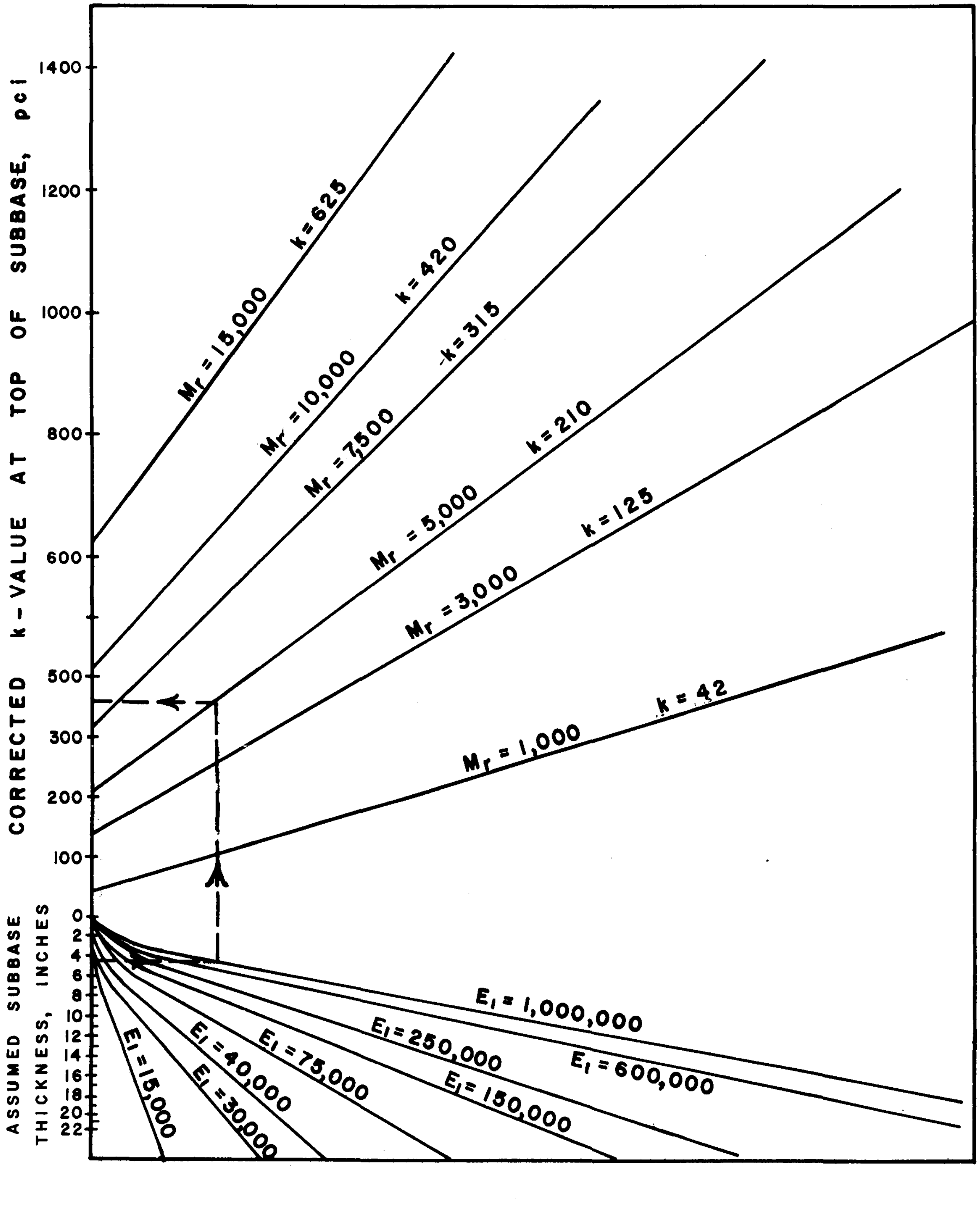

Fig 3.4-1. Chart for evaluating effect of subbase layer.

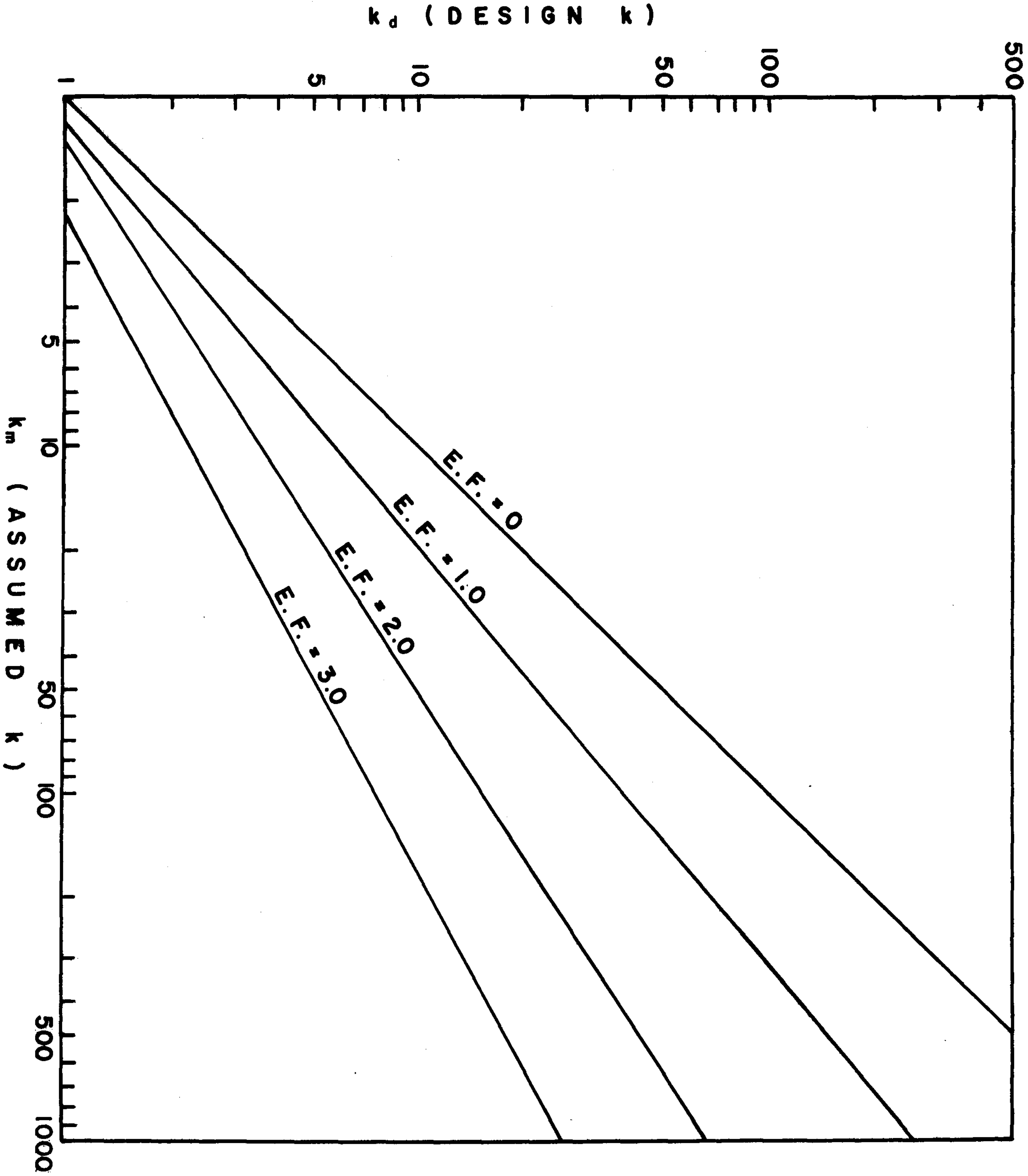

Fig 3.4-2. Correction of k-value for effect of erosive nature of subbase.

§ 3.5 - Pavement Structure Cost

A procedure is outlined here for comparing the relative cost of various acceptable designs so that the engineer may select the most economical one. The total pavement structure cost may be expressed as

$$PSC = (ICC) + (MC) \tag{3.5-1}$$

Where

PSC = total pavement structure cost during the design life in dollars per square yard.

ICC = initial construction cost of the pavement structure in dollars per square yard.

MC = maintenance cost of the pavement structure during the design life in dollars per square yard.

The initial construction cost may be expressed as follows:

$$ICC = (CPC) + (SC) + (SPC) \tag{3.5-2}$$

Where

CPC = cost of concrete slab in-place in dollars per square yard.

SC = cost of subbase in-place in dollars per square yard.

SPC = cost of subgrade preparation, such as compaction and treatment in dollars per square yard.

In most cases, the same type of pavement is to be used; hence, it may be assumed that the maintenance cost of Eq 3.5-1 will be a constant value, but it should be recognized that an inadequate subbase may have a pronounced influence on maintenance cost. Therefore, the primary variable in a subbase selection problem is the initial construction cost. In Eq 3.5-2, the subgrade preparation cost is generally constant for a specific project, but the subbase cost and concrete pavement cost are interrelated. Hence, the engineer may select the minimum cost design by selecting the subbase-concrete-pavement combination that gives the lowest cost.

Figure 3.5-1 is a sample format that the engineer may use in making a cost comparison for selecting the most economical design. The form allows a tabulated solution of Eq 3.5-2 with the subgrade preparation cost deleted. The total number of designs considered will be the product of the number of materials available, stabilization types considered, and subbase thicknesses. Where more complete cost information is available relating to maintenance cost and subgrade preparation cost to subbase type, the engineer should make the more detailed analysis required by Eqs 3.5-1 and 3.5-2.

After filling in the form (Fig 3.5-1), the engineer can make a preliminary design selection based on the design combination showing the minimum cost.

The designer can make a reasonable estimate of the subbase cost based on material and labor costs or on previous bid estimates with similar materials. If the material is priced on a cubic yard or a ton basis, then the next step is to change it to a cost per square yard per inch of subbase. The cost in the chart is assumed to be the value for the in-place material at the compacted density. If the material cost is on a dollar per ton basis, the in-place density of the material must be used with the chart.

Subbase Type	Design Information				Cost Information				
	Subbase Thickness (Inches)	K top	K corr.	Pavement Thickness (Inches)	Subbase Cost		Pavement Cost		Pavement Structure Cost
					$/sy/in	$/sy	$/sy/in	$/sy	$/sy
1	2	3	4	5	6	7	8	9	10
	T_{A1}			D_{A1}					C_{A1}
A	T_{A2}			D_{A2}					C_{A2}
	=			=					=
	T_{A-N}			D_{AN}					C_{AN}
	T_{B-1}			D_{B1}					C_{B1}
B	T_{B-2}			D_{B2}					C_{B2}
	=			=					=
	T_{B-N}			D_{BN}					C_{BN}
	T_{C-1}			D_{C1}					C_{C1}
C	T_{C-2}			D_{C2}					C_{C2}
	=			=					=
	T_{C-N}			D_{CN}					C_{CN}

Fig 3.5-1. Example of Cost Comparison of Acceptable Designs.

SECTION 4

EXAMPLE PROBLEM FOR SUBBASE DESIGN

In this section, the concepts and procedures discussed in the previous sections are used to illustrate the design steps required in selecting an optimum subbase type and thickness. The example problem follows the procedural steps discussed in the Fig 3.1-1. Since the selection of the optimum subbase type and thickness is related to the design of the pavement thickness, the design example must be correlated with a pavement thickness design method. For this example, the CRCP fatigue method for determining pavement thickness is used (Ref 1). For agencies utilizing other design methods, the pavement thickness as determined should be used.

Step 1: Using plate load tests as described in Appendix A, a k value of 125 pci is obtained for the natural subgrade.

Step 2: A survey of the available material sources indicates a slag aggregate material is available nearby and also a natural gravel material. The plasticity indexes for the slag and gravel are found to be 0 and 10, respectively. The gradation for both materials is such that the use of stabilizing agents may be considered.

Step 3: Using Fig 2.5-1 as a guide, possible stabilizing agents are tar and asphalt for the slag, and tar, asphalt, or cement for the natural river gravel. A preliminary check of cost for the proposed materials is shown in Table 4-1. These data indicate the asphalt to be slightly higher than the tar, i.e., \$0.17 per gallon for asphalt and \$0.13 per gallon for tar; therefore, assuming the materials would be used in approximately the same quantities, the asphalt may be eliminated from further consideration as a stabilizing agent. Based on preliminary tests, the percentages by weight of stabilizing agent required for each material type are 8 percent cement and 5 percent tar for slag and 9 percent cement and 6 percent tar for gravel.

Using these weights and the cost information from Table 4-1, a cost per square yard per inch of thickness is developed for each of the possible subbase types. Note that there are six possible subbase combinations that warrant further consideration. The elastic properties for each of these combinations are shown in Table 4-2.

Step 4: In order to investigate cost over a wide range of thicknesses, trial values of 4, 8, 12, and 16 inches for each subbase combination will be considered. For the zero subbase thickness, the k value would be that of the subgrade. These values are entered in Column 2 of the design table (Table 4-1).

Step 5: Using Fig 3.4-1, an estimate of the k value at the top of the subbase may be made for the various subbase types and thicknesses. The results of this analysis are shown in Column 3. Note that the improved k value of the subgrade (125 pci), ranges from a low of 140 for four inches of natural gravel or slag subbase to a value of 850 pci for 16 inches of cement-stabilized subbase.

Step 6: This step consists of evaluating the capability of the subbase to retain its full support value. Using the information discussed in Section 2, it was estimated that the natural soil should be rated with an erodability factor of 3.0 and the gravel subbase with an erodability of 1.0. Due to the lack of fine grained material in the slag its erodability factor was considered as 0.5. The brush test of specimens indicates sufficient stabilizing agent has been provided in all cases so that no erosion will be experienced by the stabilized subbase layers; hence an erosion factor of zero may be used. Using Fig 3.4-2 the k values in Column 3 are modified on the basis of their estimated capability to retain pavement support. The corrected values are shown in Column 4 of the figure. These values in Column 4 are then used with the CRCP Design Manual (Ref 1) to estimate the required pavement thickness based on the various design parameters. At this point in the procedure, either the CRCP Design Manual or some other design method should be utilized.

Step 7: In order to determine the pavement thickness by the fatigue method from the CRCP Manual, the modulus of elasticity and flexural strength of the concrete are required along with a cumulative total equivalent 18-kip axle loads

TABLE 4-1. COST DATA FOR MATERIALS BEING CONSIDERED

Material	Cost, dollars
Slag	3.25 per cu. yard
River gravel	3.00 per cu. yard
Tar	0.13 per gallon
Asphalt	0.17 per gallon
Cement	1.20 per sack
Concrete slab in-place	25.00 per cu. yard

TABLE 4-2. COST DATA AND ELASTIC PROPERTIES FOR FEASIBLE SUBBASE LAYERS BEING CONSIDERED

Material	Cost, \$/cy	Cost, \$/sy/in	Modulus of Elasticity
Slab	3.25	0.091	16,000
River gravel	3.00	0.083	13,000
Tar-stabilized slag	4.84	0.134	260,000
Tar-stabilized gravel	4.92	0.137	240,000
Cement-stabilized slag	5.87	0.163	1,000,000
Cement-stabilized gravel	5.97	0.166	950,000

Fig 4-1. Cost Comparison of Feasible Designs Example Problem

Subbase Type	Design Information: Subbase Thickness (Inches)	K top	K corr.	Pavement Thickness (Inches)	Cost Information: Subbase Cost $/sy/in	Subbase Cost $/sy	Pavement Cost $/sy/in	Pavement Cost $/sy	Pavement Structure Cost $/sy
1	2	3	4	5	6	7	8	9	10
	0	125	16	9.5	0.163	0	0.695	6.60	6.60
Cement-stabilized slag	4	220	220	8.0		0.66		5.56	6.22
	8	430	430	7.5		1.32		5.21	6.53
	12	650	650	7.2		1.98		5.00	6.98
	16	850	850	7.0		2.64		4.86	7.50
Tar-stabilized slag	4	170	170	8.1	0.134	0.53		5.62	6.15
	8	300	300	7.8		1.06		5.41	6.47
	12	410	410	7.5		1.59		5.21	6.80
	16	530	530	7.3		2.12		5.07	7.19
Gravel	4	140	50	8.7	0.083	0.33		6.04	6.37
	8	155	60	8.6		0.66		5.98	6.64
	12	170	70	8.5		1.00		5.90	6.90
	16	190	75	8.4		1.33		5.84	7.17
Slag	4	140	80	8.4	0.091	0.36		5.84	6.20
	8	155	90	8.3		0.72		5.76	6.48
	12	170	95	8.25		1.08		5.73	6.81
	16	190	100	8.2		1.45		5.70	7.15

Design Parameters:

Modulus of Elasticity of Concrete $E = 4 \times 10^6$ psi

Total Traffic = 7×10^6 18^k Axles

Concrete Strength = 690 psi

Subgrade Modulus K = 125 pci

expected for the design period (Ref 1). The values for the project in question were found to be as follows:

(1) modulus of elasticity of concrete = 4×10^6 psi,

(2) flexural strength of concrete = 690 psi,

(3) total equivalent 18-kip single axle loads = 7 million applications in 20 years, and

(4) concrete support values k as presented in Column 4 of Fig 4.1-1.

Step 8: Utilizing the parameters designated in Step 7 and Fig 2.1-1 in the CRCP Design Manual, the estimated pavement thickness is obtained, and entered in Column 5 of Fig 4.1-1. Note that the pavement thickness ranges from 7 inches with 16 inches of cement-stabilized subbase to 9.5 inches if no subbase is used. The concrete and subbase costs from Tables 4-1 and 4-2 are then entered in Columns 6 and 8. Using the cost per square yard per inch of thickness and the subbase and pavement thicknesses from Columns 2 and 5, the costs in dollars per square yard for each subbase and pavement are computed and entered in Columns 7 and 9, respectively. Adding Columns 7 and 9 gives the total pavement structure cost and is entered in Column 10.

Step 9: Steps 1 through 8 are repeated for each subbase type. Note that the corrected k value at the top of the subbase for a given stabilizing agent is equal for each material type, since for preliminary testing, the same stiffness value is used for a given stabilizing agent, regardless of aggregate type. If the designer has previous experience with a given aggregate, more exacting values may be used at this point. Therefore, the stabilized river gravel may be eliminated at this point, since its unit cost is higher than the slag for both the tar and cement additives. Hence, the number of feasible subbase layer types has been reduced to four, i.e., unstabilized slag, unstabilized river gravel, tar-stabilized slag, and cement-stabilized slag.

Step 10: Any one of the subbase and pavement thickness combinations in Fig 4-1 is a satisfactory design from a performance standpoint. Therefore, the designer can select an optimum design with minimum cost. Since there are several methods of interpreting minimum cost the selection of the final design is presented in more detail in Section 5.

SECTION 5

SUMMARY AND INTERPRETATION OF DESIGNS

§ 5.0 - General

Section 1 emphasized that the subbase was a part of the pavement structure system, and thus subbase design should be considered relative to the entire system. The example problem in Section 4 illustrates how a number of adequate designs can be obtained which give equal performance. Section 5.1 discusses two possible methods of deriving the most economical design. Section 5.2 illustrates how this manual may be used if a fixed subbase thickness is specified. Section 5.3 briefly discusses an example problem for a poor subgrade.

§ 5.1 - Selection of Most Economical Design

The designer should interpret the cost data in Fig 4-1 based on the procedures and equipment anticipated for construction. If slip form paving is expected, the designer may select thicknesses based on optimum cost. In contrast, if conventional form construction is expected, restrictions are imposed by the height of the forms available, reducing the number of alternates from which he can choose.

§§ 5.11 - Optimum Design. It was pointed out in Ref 1 that the use of a slip form paver permits the placement of any depth pavement. Hence, the designer is no longer faced with pavement thickness increments of one inch. Therefore, the most economical combination of thicknesses may be selected directly. For a clearer picture of the meaning of the data in Fig 4-1, graphs showing total pavement structure cost and CRCP thickness in terms of subbase thickness may be prepared, as illustrated in Figs 5.1-1 and 5.1-2. The optimum or minimum cost of pavement structure may be found in the figures. The subbase thickness associated with the minimum cost may be obtained from low point of the family of curves. Figure 5.1-2 may be used to determine the optimum CRCP thickness for the subbase selected. For this example, the minimum cost is $6.15 per

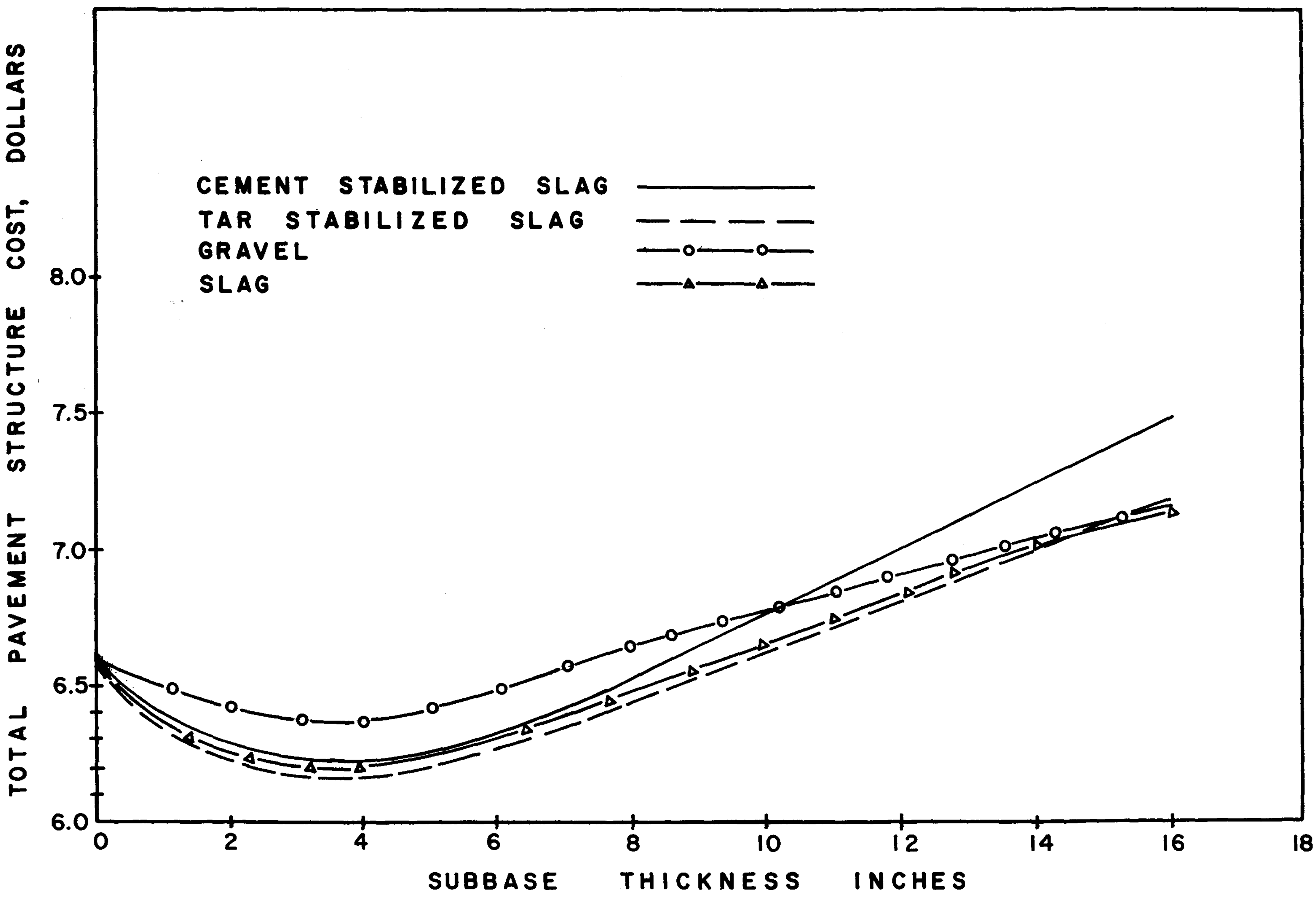

Fig 5.1-1. Subbase thicknesses for minimum cost.

square yard, using a tar-stabilized slag subbase 4 inches thick with a CRCP thickness of 8.1 inches.

§§ 5.12 - Sub Optimum Design. If conventional forms are anticipated on the project, the designer is usually restricted to pavement thickness increments of one-half inch. In this case, Fig 5.1-2 is used to determine subbase thicknesses that are compatible with one-half inch increments. For this example, note that 8.0 and 8.5 inches of CRCP are feasible thicknesses. From Fig 5.1-2, the subbase thicknesses corresponding to the two CRCP thicknesses for each subbase type may be obtained to complete Columns 1 and 2 of Table 5.1-1. Figure 5.1-1 is used to obtain a total pavement structure cost for each subbase thickness shown in Column 2 and entered in Column 3 of Table 5.1-1. The designer now has sufficient information to select the minimum cost design. For this example, the minimum cost design for a one-half inch increment of CRCP thickness is either 5.2 inches tar-stabilized slag with 8.5 inches of CRCP or 3.2 inches of plain slag with 8.5 inches of CRCP.

In this instance, the cost of the two acceptable designs is very close to the optimum. Therefore, the engineer may select stabilized layer to facilitate use by construction equipment. In some instances considerable difference may be noted.

§ 5.2 - Procedures for Fixed Subbase Thickness

The designer may use this manual for fixed thickness subbases if he so desires. For these conditions, the same procedure would be used as described schematically in Fig 3.1-1, except that in this case only the fixed thickness would be considered, rather than a range.

TABLE 5.1-1. SELECTION OF SUB OPTIMAL COST DESIGN (ONE-INCH INCREMENTS OF CRCP THICKNESS)

Material (1)	Pavement Thickness (inches) (2)	Subbase Thickness (inches) (3)	Pavement Structure Cost \$/sy (4)
Slag	8.0	16(+)	7.20(+)
	8.5	3.2	6.20
Gravel	8.0	20(+)	7.20(+)
	8.5	12	6.90
Tar-stabilized slag	8.0	5.2	6.20
	8.5	2.1	6.22
Cement-stabilized slag	8.0	4.0	6.22
	8.5	2.0	6.30

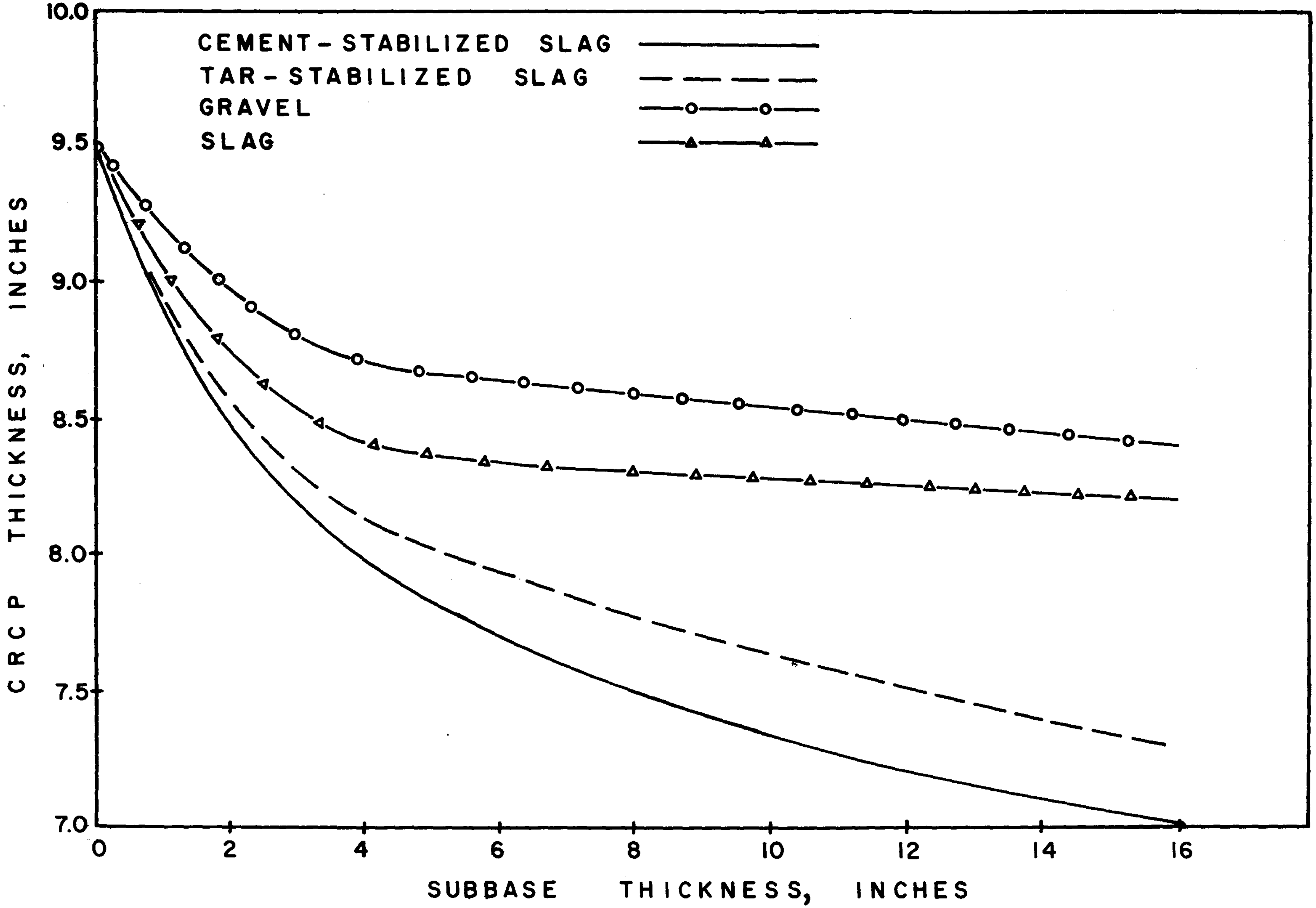

Fig 5.1-2. Optimum CRCP thicknesses for subbases thickness selected.

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APPENDIX A

PROCEDURE FOR DETERMINING k VALUE

APPENDIX A. PROCEDURE FOR DETERMINING THE MODULUS OF SUPPORT k

(AASHO Road Test Plate Load Tests)

Equipment

The basic equipment consists of (1) a reaction trailer, (2) a hydraulic ram and a jack, (3) various heights of steel spacers for use where required by various depths of test, (4) a 12-inch-diameter cylindrical steel loading frame cut out on two sides to allow the use of a center deflection dial, (5) a spherical bearing block, (6) a series of one-inch-thick steel plates that are 12, 18, 24, and 30 inches in diameter, and (7) a 16-foot aluminum reference beam. A schematic diagram of the apparatus is given in Fig A-1.

A trailer of the flat-bed type, having no springs and four sets of dual wheels on the rear can be used as the reaction trailer. A cantilever beam protruding from the rear of the trailer is used as the reaction beam. The distance from the load to the rear wheels should be 8 feet. A maximum reaction of about 12,000 pounds could be obtained with a 17,000 pound loaded rear axle.

A standard hydraulic ram is used to apply the load. A calibration curve, which should be checked periodically, is used to convert gage pressures to load in pounds.

The load is applied to the plates through the 12-inch-diameter steel loading frame and the sperical bearing block. The deflection is measured with a dial gage as shown in Fig A.1.

The weight of the loading frame and the plates is allowed to act as a seating load for which no correction should be made.

Test Procedure

Tests are made in areas about 3 to 4 feet wide. The procedure provides for the application and release of 5, 10, and 15 psi loads on a 30-inch plate and for measurement of the downward and upward movement of the plate. The loads are applied slowly with no provision for the deformation to come to equilibrium.

Basic steps in the procedure are

(1) Cover the test area with fine silica sand and level by rotating the plate.

(2) Set the equipment in place (Fig A.1).

(3) Apply a seating pressure of 2 psi and release. Then set the dial gages to zero.

(4) Apply the first increment of pressure and hold for 15 seconds, then read the dial gage.

(5) Release the load and read the dial gage at the end of a 15-second period.

(6) Reapply the load and release in the same manner three times, taking readings each time.

(7) Repeat Steps 4 through 6 for the second and third increments of load, 10 psi and 17 psi, respectively.

(8) Compute the gross and elastic deflections from the dial gage readings.

Computation of Modulus of Support

The gross k value k_g equals the unit load divided by the maximum gross deflection obtained after three applications of a given unit load. The reported k is then an average of the computations for each of the unit loads.

The elastic k value k_e equals the unit load divided by the elastic deformation at each application of each incremental load. The reported k_e is an average of all nine of these computations (3 loads $\times$ 3 applications each). The elastic deformation is equal to the difference between the maximum gross deflection and the final reading on the dial.

The relationship between the two k values as developed through correlation from numerous tests on the AASHO Road Test is $k_e = 1.77\ k_g$.

Values of k are reported as pounds per cubic inch.

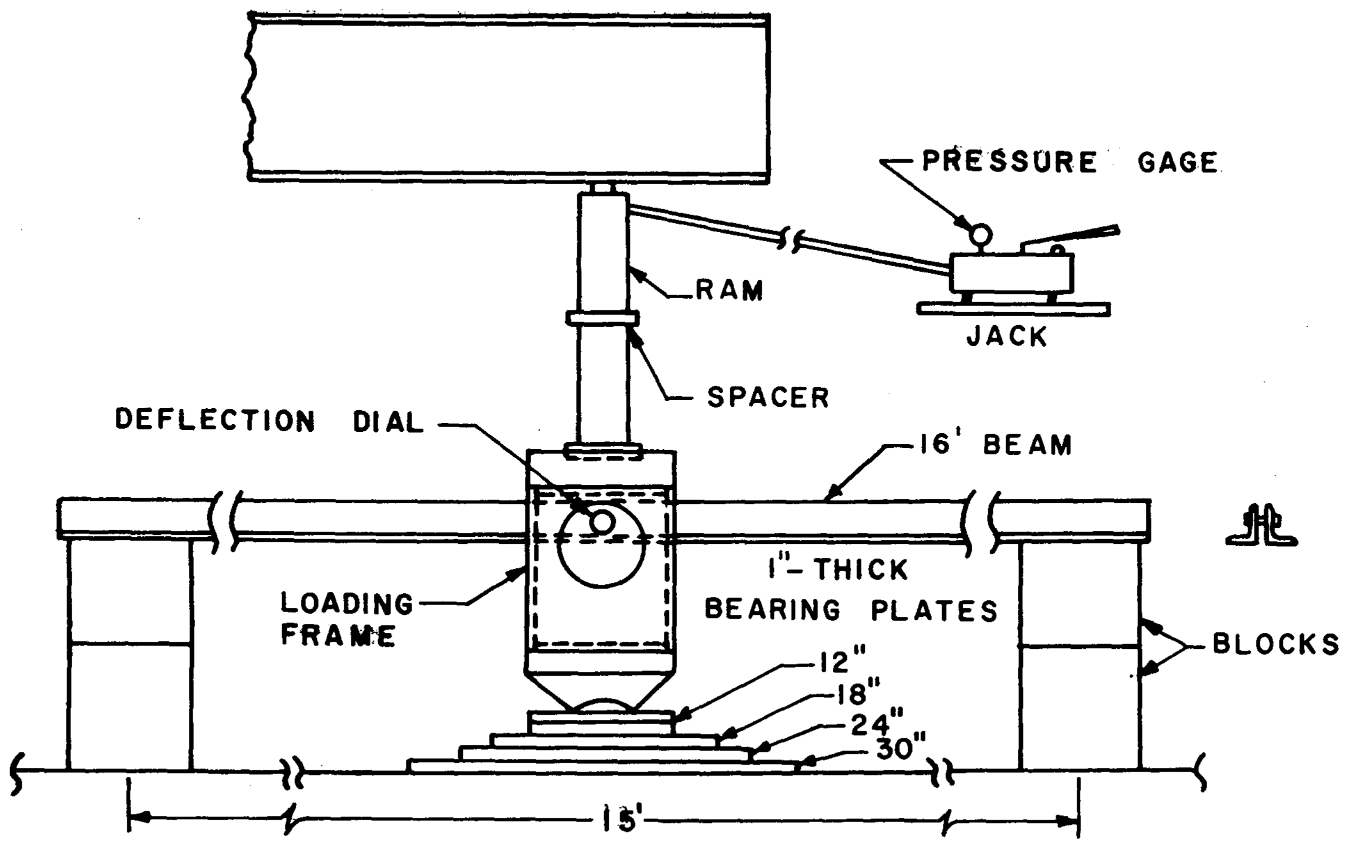

Fig A.1. Apparatus for plate load test.

APPENDIX B

PROCEDURE FOR RESILIENT MODULUS TEST

APPENDIX B. PROCEDURE FOR RESILIENT MODULUS TEST

Scope

This method describes a procedure for testing, under dynamic loading with controlled stress conditions, untreated aggregate specimens or aggregate specimens bound with flexible binders. Stress control is defined as the process of applying a predetermined axial load to a specimen and measuring the axial deformation or strain which the specimen undergoes. Data obtained with this procedure can be used in determining damping characteristics and moduli of resilience of the test specimen. The equipment for dynamic triaxial loading under controlled stress consists of three basic components:

(1) triaxial cell with loading piston and transducers for measuring load and strain or deflection,

(2) controlled cyclic air supply, and

(3) power amplifier with oscillograph.

Apparatus

(1) loading piston.

(2) triaxial cell of suitable size for testing 2-1/2-in. × 5-in. × 12-in. specimens.

(3) cyclic air supply.

(4) LVDT's suitably mounted for measuring the deformation due to the applied load.

(5) timer to regulate speed of testing machine at frequencies up to 3 cycles per second.

(6) load cell, for controlling stress.

(7) CP amplifier.

(8) plug-in module.

(9) visicorder.

(10) rubber membranes of suitable size for confining 2-1/2-in. × 5-in. and 6-in. × 12-in. test specimens.

(11) o-rings, of suitable size to fasten membrane to base and top caps.

Procedure

(1) Measure and record height and weight of specimen.

(2) Place suitable membrane around specimen. For testing under unconfined conditions, i.e., (confining pressure = 0, omit 3(b), (c), (f) and (g)).

(3) Secure membrane to top cap and base cap with o-rings.

(4) Place specimen with membrane in triaxial cell.

(5) Extend rod from main load piston to top cap of specimen.

(6) Apply predesignated confining pressure.

(7) Make appropriate rod correction.

(8) Set air pressure at inlet to give predesignated load stress to the specimen.

(9) Record applied load and deflection on an oscillograph trace at the following designated intervals:

Interval	Number of Cycles
1	0 to 10
2	50 to 60
3	500 to 510
4	1,000 to 1,010
5	2,000 to 2,010
6	5,000 to 5,010
7	10,000 to 10,010
8	20,000 to 20,010

If specimen fails during test, report the number of cycles to failure.

Calculations

From the data reported above, develop the following plots:

(1) hysteresis loops at representative cycles for each interval,

(2) permanent set as a function of number of load repetitions.

For each test interval, make the following calculations:

(1) Damping coefficient (η) - the energy absorbed during a dynamic cycle D divided by the total energy applied during the cycle W .

(2) Modulus of resilience - $(M_R)'$ dynamically applied deviator stress $\Delta\sigma$ divided by the resulting dynamic elastic (recoverable) strain ε : $M_R = (\Delta\sigma)/\varepsilon = (\sigma_1 - \sigma_3)/\varepsilon$.

APPENDIX C

SPECIFICATIONS FOR COAL TAR

SPECIFICATIONS FOR COAL TAR CUT-BACK, RT-6-C (DH-2)

These specifications cover coal tar cut-back for use in surface treatment, soil bituminous stabilization, bituminous surface courses AT-1, CP-2, FB-1 and FB-2.

The material is to be heated, if required, for proper application between 130 F and 175 F depending on the viscosity of the material.

This material shall contain not less than 50 nor more than 95 percent by volume of refined coal tar base, fluxed with a tar material (liquid at 60 F) which shall make a homogeneous mixture. The base shall contain only products obtained from high temperature carbonization of coal. The flux shall be a water gas tar or either distillates of water gas tar or coal tar or a combination of water gas tar and the above distillates. The flux, base, and mixture shall conform to the following requirements respectively:

	Flux		Base		Mixture	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
Water, percent by weight	-	-	-	-	-	1.5
Specific gravity at 25/25 C	0.98	1.12	1.15	1.26	1.14	1.24
Float test at 50 C, sec	-	-	30	220	-	-
Specific viscosity, Engler						
50 cc at 40 C	-	3.6	-	-	-	-
50 cc at 50 C	-	-	-	-	26.0	40.0
Bitumen soluble in carbon disulphide, percent by weight	95	-	80	95	86	97
Distillation, dry basis, percent by weight:						
0-170 C	-	7	-	2	-	2
0-235 C	-	-	-	-	2	10
0-270 C	-	-	-	-	6	21
0-300 C	45	87	-	25	14	28
Specific gravity at 38/38 C of total distillate (water free) to 300 C	-	-	1.00	-	0.98	-
Softening point, of distillation residue C (ring and ball method)	-	-	-	60	35	50
Sulfonation index (on 300 to 355 C distillate)	-	-	-	-	-	1.5

SPECIFICATIONS FOR COAL TAR CEMENT RT-12-C (BM-2)

These specifications cover coal tar cement for use in bituminous base course, bituminous penetration course DP-1, bituminous surface course and premix patch A and B.

This material is to be heated, if required, for proper application between 200 F and 250 F depending on the viscosity of the material.

The tar shall be the product consisting entirely of materials derived from the high temperature carbonization of coal refined to the specified consistency.

When used in bituminous concrete, the maximum delivery temperature of the material shall not exceed 275 F. When the temperature of the material falls below the temperature which yields a viscosity between 100 and 250 centistokes, it shall be heated to yield the proper viscosity.

The coal tar cement shall be homogeneous, shall not foam when heated to 250 F and shall meet the following requirements:

	Minimum	Maximum
Water, percent by weight	-	0
Specific gravity 25/25 C	1.20	1.26
Float test at 50 C, sec	150	220
Bitumen soluble in carbon disulphide, percent by weight	80	95
Distillation, percent by weight:		
0-170 C	-	1.0
0-270 C	-	10.0
0-300 C	-	20.0
Specific gravity at 38/38 C of total distillate at 300 C	1.02	-
Softening point of distillation residue C (ring and ball method)	35	60
Sulfonation index (on 300 C to 355 C distillate)	-	1.0

SPECIFICATIONS FOR WATER GAS TAR CEMENT RT-10-W (HH-3)

These specifications cover water gas tar cement for surface treatment, and bituminous surface courses AT-1, CP-2, FB-1 and FB-2 for warm weather use, unless otherwise specified.

This material shall be a product consisting of water gas tar refined to the specified consistency.

The material is to be heated, if required, for proper application to the road between 200 F and 240 F depending on the consistency of the material.

The water gas tar shall be homogeneous, and shall not foam when heated to 240 F and shall meet the following requirements:

	Minimum	Maximum
Water, percent by weight	-	0
Specific gravity at 25/25 C	1.16	-
Float test at 50 C, sec	75	100
Bitumen, soluble in carbon disulphide, percent by weight	85	-
Distillation, dry basis, percent by weight:		
0-170 C	-	1
0-270 C	-	10
0-300 C	-	23
Specific gravity at 38/38 C, of total distillate to 300 C	0.95	-
Softening point, of distillation residue, C, (ring and ball method)	40	70
Suflonation index (on 300 C to 355 C distillate)	-	1.0

This material should not be used for surface treatment prior to April 15, nor after September 1.

APPENDIX D

DERIVATION OF A SUBBASE ERODABILITY TERM

APPENDIX D. DERIVATION OF A SUBBASE ERODABILITY TERM

Introduction

To properly design rigid pavements, a representative value of the modulus of the subgrade reaction k is required. The k-value should represent the support conditions during the life of the pavement, not just the initial conditions. With the same load, higher stresses will develop for lower values of k , and thus a greater thickness of the slab is required.

For various reasons the subgrade material under the slab may be eroded, reducing the extent of soil support and the k-value, hence inducing more stress in the slab. The reduction in the k-value will depend upon the extent of the expected erosion beneath the slab.

At the present time, there is no way to account for this reduction in the k-value of the subgrade. Therefore, during the development of this manual, a procedure was formulated to obtain a k-value accounting for full support and the expected subbase erosion.

The analysis of this study is to a great extent based upon the computer program of stress calculation in the slab as developed by Dr. W. R. Hudson and Hudson Matlock. This program can calculate the stresses in the slab for various conditions of slab support. Therefore, by assuming various degrees of erosion and types of subgrade, stress patterns in the slab can be calculated. Based on these stress patterns, a correlation between the actual k-value and design k-value was developed.

Erodability Factor and Degree of Support

For development of a correlation between the erosion below the subgrade and the k-value, some parameter is required which defines the erosion quantitatively. This parameter has been defined as the "Erodability Factor." The values of this factor have been tentatively assumed, as follows, under a slab of 20 feet by 40 feet size.

Condition	Size of Hole in feet	Area of Hole in square feet	Assumed Erodability Factor
1	0 x 0	0	0
2	3 x 4.25	12.75	1
3	7 x 5.25	36.75	2
4	9 x 7.25	65.25	3

The values of these void areas are based on engineering judgment assuming the erosion of 65.25 square feet about 8 percent under a slab of 20 feet by 40 feet to be the most severe condition. The pattern of erosion assumed for each case is shown in Fig 1.

For various assumed values of k, the stresses were computed for all four cases of erodability. The maximum stresses in the slab were found to increase as the value of EF increases, as would be expected. The values of maximum stresses obtained in the slab for various assumed K and EF values are shown in Fig 2.

Relation Between Assumed K_m and Design K_D

Based on the fact that for EF = G, the value of K_D and K_m must be the same for certain stress values in the slab, the values of K for given values of stress were read from Fig 2 for various EF values. These values were designated as K_D and plotted against the corresponding value of K_m to develop the design chart in the text (Fig 3.4-2). Using the data and the plot, the following expression was developed:

$$G\,(\log K_m - \log F) = \log K_D$$

where

$F = 1 + 0.4(EF) - 0.3(EF)^2 + 0.1(EF)^3$

$G = 1 - 0.155(EF) - 0.34(EF)^2 + 0.0113(EF)^3$

K_D = Modulus of support for use in design equations.

K_m = Modulus of support for conditions of zero erosion.

EF = Erodability factor of subbase or subgrade.

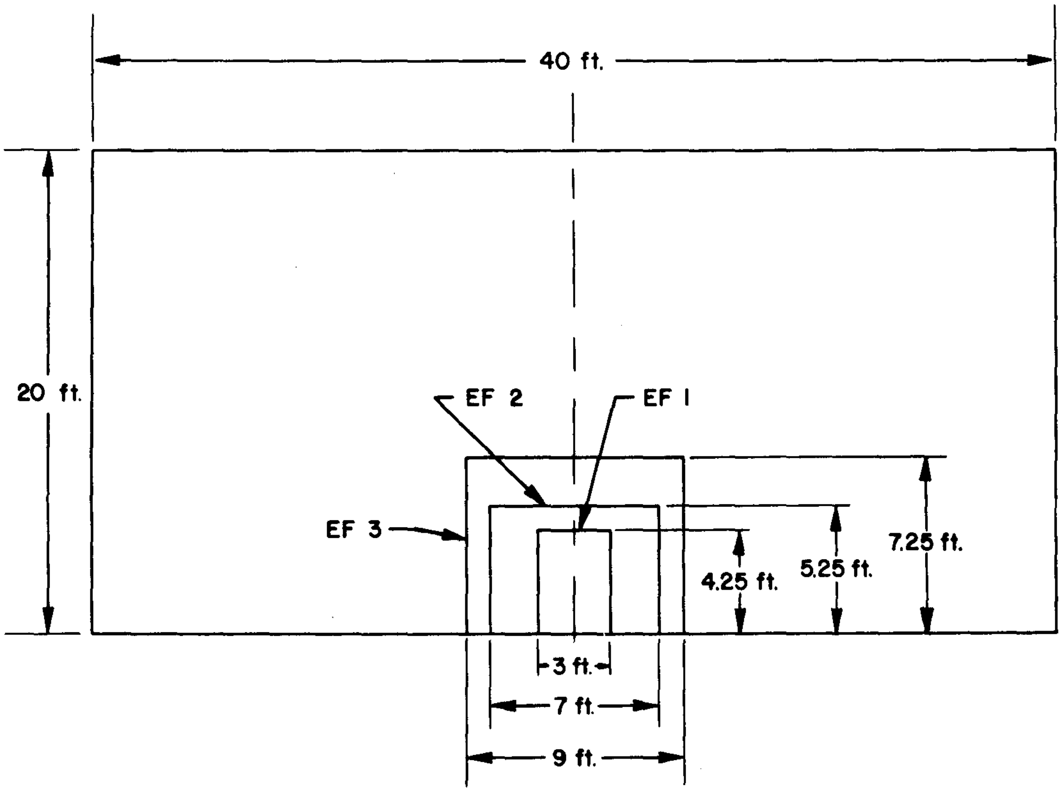

Fig 1. Slab showing holes for various EF values.

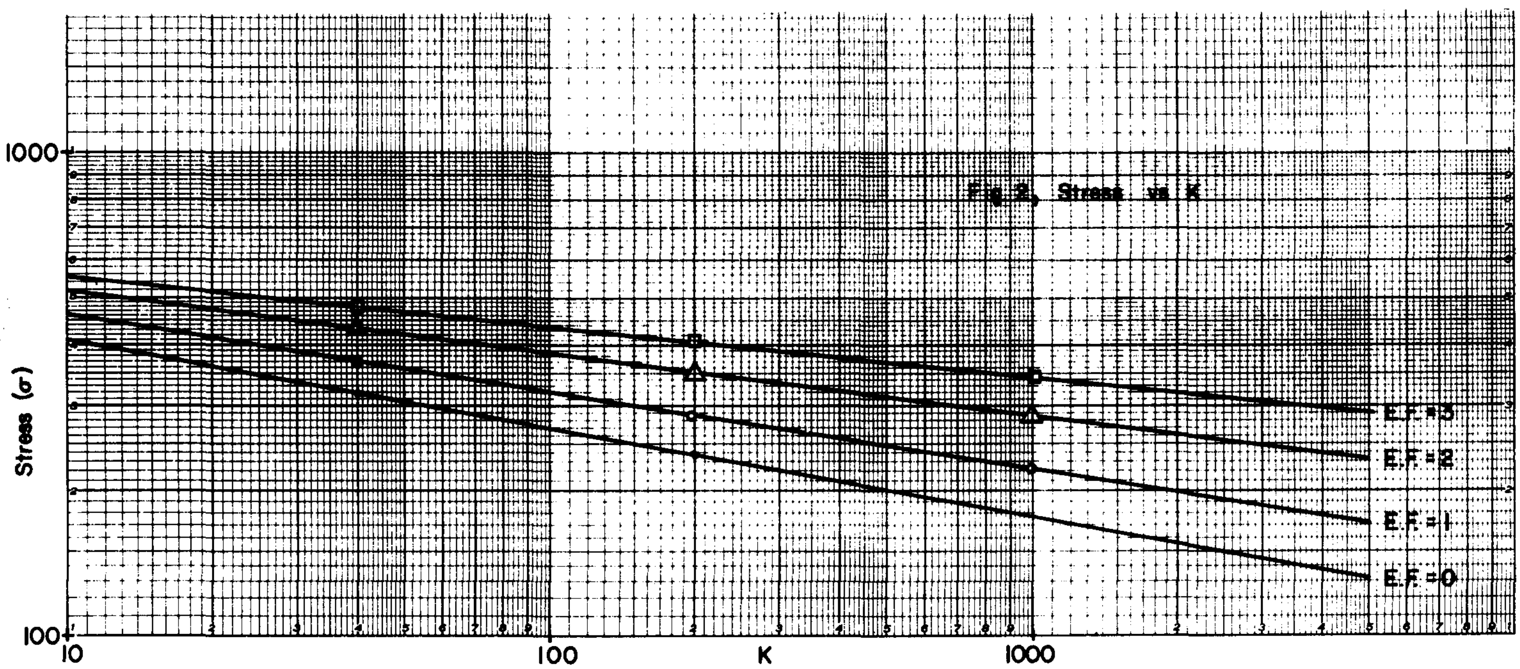

Fig. 2 Stress vs. modulus of support.

The value erodability factor for a given material must be estimated on the basis of the anticipated loss of support during the life of the pavement. At the present time, there are no tests that may be used directly to determine the erodability factor, hence engineering judgment must be used.