# LATERITE AND LATERITIC SOILS AND OTHER PROBLEM SOILS OF THE TROPICS

AN ENGINEERING EVALUATION AND
HIGHWAY DESIGN STUDY FOR
UNITED STATES AGENCY FOR
INTERNATIONAL DEVELOPMENT
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## VOLUME II

## INSTRUCTION MANUAL

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## CHAPTER I
### *SUMMARY OF BACKGROUND INFORMATION*

## INTRODUCTION

This is an instructional manual for field inspectors and laboratory technicians who work on engineering and construction projects which utilize tropical soils. The engineering descriptions, procedures and specifications described herein are a consolidation of information obtained from Volume I "Laterite and Lateritic Soils and Other Problem Soils of the Tropics". Both Volumes I and II were prepared by the firm of Lyon Associates, Inc., in 1975 and are the final report of a worldwide tropical soil study sponsored by the United States Agency for International Development. The data sources are referenced in Volume I and omitted in Volume II to simplify the presentation.

## SOILS AND ENGINEERING DESIGN

Soil materials are a vital element in all civil engineering designs. Therefore, an understanding of the origin, development and use of soil materials is a basic requirement for the field and laboratory personnel who work with them. The following is a brief review of the soil development process and a discussion about the properties which influence design.

All soils are formed of materials which have undergone a complex chemical and physical weathering process. This weathering process is complex because it takes millions of years and during the process the principal agents which affect soil development, e.g. physiography, geology and climate, also undergo changes. The complexity is further demonstrated by the fact that soils which are formed within a few feet of each other can have very different properties; yet each of these soils can be similar to other soils located thousands of miles away. The similarities and differences between all soils are commonly measured in terms of their chemical, mineralogical and physical properties. Most soils are classified into one or more systems devised to group soils according to such properties. These systems are used to simplify the description and identification of soil types.

Classifications based on physical properties and behaviors are of the greatest interest to engineers, since soils with similar physical characteristics can often be used in engineering designs of similar facilities even though the soils may exhibit different chemical and mineralogical properties. A soil's physical behavior is of prime importance to the engineer because it reflects the soils strength characteristics and such characteristics are the keystone to a safe and efficient design.

Chapter I of Volume I discusses the elements of the weathering process in greater details. In addition Chapter I introduces the elements of the soil forming process and the properties of the soil which are the most significant to the engineer.

## TROPICAL ENVIRONMENTS AND SOILS DEVELOPMENT

There are a number of regional environmental features which significantly affect the soil development process. Among these environmental features physiography, geology and especially climate are the most important in the development of tropical soils.

A basic knowledge of a region's geology provides a background for determining the probable type of parent materials which would be encountered and the probable duration of the weathering process. A knowledge of the physiography, namely the topographic description of the region, when coupled with a knowledge of the climate, namely the amount and seasonal distribution of rainfall lead to a fundamental understanding of the tropical weathering process.

The climate of a region is perhaps the most important element in weathering and the soil formation process. A knowledge of the geology is an important factor during the early stages of soil development and identification of the physiography is important in understanding the current stage of topographic development and drainage conditions.

The location of environmental features in South America, Central America, Africa and Southeast Asia which effect the soil and soil development process are described and illustrated in Chapter 2 of Volume I. The chapter is organized to provide an overview of the environmental setting of the entire Equatorial region and to identify the general geotechnical problem areas.

Tropical soils are principally the product of chemical weathering. A more detailed discussion of the development of tropical soils is provided in the first part of Chapter 3 of Volume I. In summary, there are many factors which add to the complex weathering process in tropical environments. The composition and texture of the parent rocks are important in the initial stages of weathering but become less important with time. Climatic features such as the quantity of rainfall, and particularly the seasonal distribution of rainfall, determine the intensity of the weathering process. Topography affects vertical water movement and consequently the rate of removal of soluble materials. On steep slopes run-off may be as active on eroding the weathered material as infiltration is in forming it. The type and amount of vegetation can be important in the formation of organic acids and in the assimilation of silica. Finally, time is a controlling factor, for example in warm and humid climates typical of the tropics, the time required to chemically alter a rock material is considerably less than in temperate climates.

## SOIL PROFILE

The alteration of rock by the processes of chemical weathering takes place progressively through a series of events and stages which result in a profile of weathering. Those who have worked with tropically weathered residual soils have noted the frequent occurrence of an upper clayey zone a few feet thick, underlain by a silty or sandy zone which, in turn, passes through a very irregular transition into weathered and finally into sound rock. The thickness of each member of the profile varies greatly from site to site because of the complexity of the interrelationships among the controlling soil-forming factors. Moreover, at a given site there may be great differences in the depth and thickness of the weathered rock within lateral distances of only a few feet. The differences arise because of differences in lithology, such as the presence of a dike or a contact, or because of differences in the degree of jointing or extent of shearing. The latter features increase the permeability and the depth to which weathering can proceed.

The soil profile is a vertical cross-section of all the soil horizons from the surface to the fresh unaltered bedrock. Four or five general horizons are recognized: O, A, B, C and D. The O horizon is the surface layer of organic debris. The next layer or A horizon is the leached layer from which material is removed; it is usually less than 30 cm thick. The B horizon is the zone in which the dissolved material is deposited; it is usually one meter to several meters thick. The C horizon is weathered rock or parent material. The D horizon is the unaltered bedrock. These horizons are shown in Figure 1.1. New terminology has been introduced in recent years in order to further refine horizon descriptions. The descriptions and terminology given in Table 1.1, Soil Horizon Nomenclature are recommended for tropical regions.

The soils in these horizons can be identified in terms of their color, texture, structure and organic matter contents.

## Color

Color is one of the most obvious soil properties but it must be used with caution. The color varies with the moisture content and as a consequence the moisture content must always be mentioned when describing color. The color of mottled clays must be described at the natural or in-situ moisture content and without any undue manipulation which would tend to blend the colors.

Color may be a clue in indicating the presence of certain elements or compounds. Dark brown, dark grey and black colors usually indicate organic matter, except in tropical residual black clays. They have very low organic contents, usually 2 percent or less and the color is due to the manner in which the finely divided organic matter is held by the clay minerals. Red, yellow and some brown colors are often the results of chemical weathering. Deep red shades indicate iron oxides whereas lighter shades of yellow and yellowish brown indicate hydrated iron oxides. Yellow colors can also be due to hydrated aluminum oxides, to a mixture of aluminum and iron oxides, to allophane-rich soils and even to some organic compounds. Grey, dark grey and mottled colors generally indicate poor drainage; light greys are due to leaching.

Color has three characteristics. They are hue (the color of the spectrum), value (the amount of light reflected) and chroma (the purity of the color). The Munsell system of color identification is the one most widely used.*

### TABLE 1.1
**Soil Horizon Nomenclature* (p.4)**
**(after Birkeland, 1974)**

MASTER HORIZONS

O horizon Surface accumulation of organic material overlying a mineral soil. Lower limits are 30 percent organic matter if the mineral fraction contains more than 50 percent clay or 20 percent organic matter if the mineral fraction has no clay. The horizon is 01 if most vegetative matter is recognizable, 02 if the original form of plant or animal matter is not recognizable.

A horizon Accumulation of humified organic matter mixed with mineral fraction. Occurs at the surface or below an O horizon. Organic matter contents are less than those required for the O horizon. If data are available, the A horizon can be subdivided into the following:

*Mollic A horizons* are dark colored (chroma of 4.0 or less, value darker than 3.5, when moist), contain at least 1 percent organic matter (0.58 percent organic carbon), and have a base saturation of over 50 percent. Generally associated with grassland vegetation.

*Umbric A horizons* are similar to mollic A horizons except that the base saturation is less than 50 percent. Generally associated with forest vegetation.

*Ochric A horizons* are too light in color and low in organic matter to be mollic or umbric A horizons. Generally associated with young soils and/or semiarid vegetation.

E horizon Underlies an O or A horizon and is characterized by less organic matter and/or less sesquioxides, and/or less clay than the underlying horizon. Horizon is light colored due mainly to the color of primary mineral grains because secondary coatings on grains are absent. Also known as an A2 or albic horizon.

B horizon Underlies an O, A, or E horizon and shows little or no evidence of the original rock structure.

Argillic B horizons have more silicate clay than the A or E horizon and/or the assumed parent material. In other words, silicate clays have been translocated into the B from overlying horizons, or they have formed in place within the B, or both. Clay translocation is recognized in the field by oriented clay films that coat either mineral grains or small channels or ped surfaces. Clay films can be destroyed during subsequent pedogenesis.

*Natric B horizons* meet the requirements of the argillic B horizons, have columnar or prismatic structure, and more than 15 percent saturation with exchangeable sodium in some subhorizons.

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* Available from the Munsell Color Company, Inc.: 2441 Calvert Street, Baltimore, Maryland.

| | Horizon | |
|---|---|---|
| ORGANIC DEBRIS LODGED ON THE SOIL, USU-ALLY ABSENT ON SOILS DEVELOPED BY GRAS-SES | O or $A_0$ | ORGANIC DEBRIS. |
| ZONE OF ELUVIATION | $A_1$ | A DARK-COLORED HORIZON CONTAINING A RELATIVELY HIGH CONTENT OF ORGANIC MATTER BUT MIXED WITH MINERAL MATTER |
| | $A_2$ | A LIGHT-COLORED HORIZON REPRESENTING THE REGION OF MAXIMUM LEACHING (OR REDUCTION) WHERE PODZO-LIZED * OR SOLODIZED ** .ABSENT IN SOME SOILS. |
| THE SOLUM (THIS PORTION INCLUDES THE TRUE SOIL DEVELOPED BY SOIL BUILDING PROCESSES.) | $A_3$ | TRANSITIONAL TO B BUT MORE LIKE A THAN B. SOME-TIMES ABSENT. |
| | $B_1$ | TRANSITIONAL TO B BUT MORE LIKE B THAN A. SOME-TIMES ABSENT |
| ZONE OF ILLUVIATION. (EXCLUSIVE OF CAR-BONATES OR SULPHATES AS IN CHERNOZEM, BROWN, AND SIEROZEM SOILS IN SUCH SOILS THIS HORIZON IS TO BE CONSIDERED AS ESSEN-TIALLY TRANSITIONAL BETWEEN A AND C.) | $B_2$ | A DEEPER-COLORED (USUALLY) HORIZON REPRESENTING THE REGION OF MAXIMUM ILLUVIATION WHERE PODZO-LIZED OR SOLODIZED. IN SOME SOILS, THIS REGION HAS A DEFINITE STRUCTURAL CHARACTER, FREQUEN-TLY PRISMATIC, BUT DOES NOT HAVE MUCH IF ANY ILLUVIAL MATERIALS; IT REPRESENTS A TRANSITION BETWEEN A AND C. FREQUENTLY ABSENT IN THE IN-TRAZONAL SOILS OF THE HUMID REGIONS. |
| | $B_3$ | TRANSITIONAL TO C. |
| THE PARENT MATERIAL | C | PARENT MATERIAL |
| ANY STRATUM UNDERNEATH THE PARENT MATERIAL SUCH AS A HARD ROCK OR A LAYER OF CLAY OR SAND, THAT IS NOT PA-RENT MATERIAL BUT MAY HAVE SIGNIFICANCE TO THE OVERLYING SOIL. | D | UNDERLYING STRATUM OF UNALTERED ROCK OR PA-RENT MATERIAL |

* PROCESS OF WATER LEACHING DOWNWARD THROUGH A AND B HORIZONS

** PROCESS OF ACCUMULATING SURFACE MINERALS THROUGH LEACHING UPWARD, PRODUCED BY EVAPORATION IN AREAS OF LOW RAINFALL CAUSING MOISTURE MOVEMENTS TO BE TOWARD THE SURFACE.

FIGURE 1.1 – A HYPOTHETICAL SOIL PROFILE HAVING ALL THE SOIL HORIZONS (After PCA Soil Primer)

TABLE 1.1-Continued

*Spodic B horizons* generally occur beneath an E horizon and are characterized by a concentration of organic matter and sesquioxides that have been translocated downward from the E horizon.

*Oxic B horizons* are highly weathered subsurface horizons that are characterized by hydrated oxides of iron and aluminum, 1:1 lattice clays, and a low cation-exchange capacity. Few primary silicate minerals remain with the exception of quartz, which is quite resistant to weathering.

*Cambric B horizons* lie in the position of the B horizon and are characterized by at least enough pedogenic alteration to eradicate most rock structure, form some soil structure, and remove or redistribute primary carbonate. Their color has higher chromas or redder hues than does the color of the underlying horizons. Similar horizons can be found in other soil profiles, for example, below argillic B horizons; these horizons do not qualify as cambric B horizons, however, because they are not in the position of the B horizon. Although there is some difficulty in identifying the cambric B horizon, it is a useful term for those B horizons that are characterized primarily by the development of soil structure and/or fairly intense oxidation.

K horizon A subsurface horizon so impregnated with carbonate that its morphology is determined by the carbonate. Authigenic carbonate coats or engulfs all primary grains in a continous medium and makes up 50 percent or more by volume of the horizon. The uppermost part of the horizon commonly is laminated. If cemented, the horizon corresponds with some caliches and calcretes.

C horizon A subsurface horizon. excluding bedrock, like or unlike material from which the soil formed or is presumed to have formed. Lacks properties of A and B horizons, but includes weathering as shown by mineral oxidation, accumulation of silica, carbonates, or more soluble salts, and gleying.

R horizon Consolidated bedrock underlying soil.

SELECTED SUBORDINATE DEPARTURES

The following symbols are used with the master horizon designation to denote special features. They follow the master horizon designation, as well as any numbers (e.g. B2t).

b Buried soil horizon. May be deeply buried and not affected by subsequent pedogenesis or shallow and part of a younger soil profile.

ca Accumulation of carbonates of alkaline earths, usually calcium, in amounts greater than the parent material is presumed to have had. Occurs in A, B, C, or R horizons.

cs Accumulation of gypsum in amounts greater than the parent material is presumed to have had. Occurs in A, B, C, or R horizons.

g Horizon is characterized by strong gleying or reduction of iron, so that colors approach neutral, with or without mottles. Occurs in A, B, or C horizons.

h Illuvial concentration of humus, appearing as coatings on grains or as silt-size pellets.

ir Illuvial concentration or iron, appearing as coatings on grains or as silt-size pellets. A spodic B horizon characterized by both illuvial humus and iron is designated Birh.

m Strong irreversible cementation, for example by accumulation of iron, calcium carbonate, or silica.

sa Accumulation of salts more soluble than gypsum, in amounts greater than the parent material is presumed to have had. Occurs in A, B, C or R horizons.

si Cementation by silica, as nodules or as a continuous medium. If cementation is continuous the notation sim is used. Such horizons are also known as duripans or silcrete.

t Accumulation of translocated clay, such as in an argillic B horizon.

x Denotes subsurface horizon characterized by a bulk density greater than that of the overlying soil, hard to very hard consistence, and seemingly cemented when dry.

ox,n,

In many unconsolidated Quaternary deposits, the C horizon consists of an oxidized C overlying a seemingly unweathered C. The oxidized C does not meet the requirements of the cambric B horizon. In stratigraphic work it is important to differentiate between these two kinds of C horizons. It is suggested that Cox be uxed for oxidized C horizons, and Cn for unweathered C horizons.

cn An accumulation of concretions, usually of iron or manganese and iron.

v Denotes spherically shaped voids or vesicles in ochric A horizons in desert regions.

* In the new classification there are restrictions on some horizons located beneath the A horizon that require a minimum thickness for the horizon in question or a minimum content of various salts. Because many of these restrictions seem to serve little purpose and are inappropriate for many geomorphological studies, they are not used here.

## Texture

Texture, which is one of the most important properties of a soil profile, commonly refers to the particle size distribution. It also refers to the particle shape and arrangement. The first is determined by mechanical analysis whereas the others require special examination, which, for the finer sizes, requires a magnifying lens or microscope.

## Structure

Descriptions of the principal kinds of soil structure are shown below in Table 1.2. Individual aggregates, which are called peds, are classified according to shape. These terms are widely used and should be useful for field descriptions.

### TABLE 1.2
**Types of Soil Structure (after Hunt, 1972)**

1. Loose single grains without cohesiveness.
2. Crumb: small, soft, porous aggregates.
3. Platy: particles or partings arranged in planes, generally at or near the ground surface and parallel to it: plates generally less than 10 mm (0.4 in) thick.
4. Prismatic or columnar: particles or partings arranged in columns: partings planar or curved; columns generally 2 to 5 mm (0.08 to 0.2 in) in diameter and normal to the surface. In many kinds of ground the prisms or columns end upward at the base of a layer having platey structure.
5. Blocky: aggregated, closely packed clumps, commonly 2 to 3 mm (0.08 to 0.12 in) in diameter and roughly equidimensional but having very irregular surface; corners round or angular.
6. Granular: similar to 4 but aggregates small, mostly less than 5 mm (0.2 in).
7. Nodular: like 4 and 5 but nodules not closely packed and commonly widely spaced; nodules generally differ in composition from the matrix.
8. Tubular: irregular tubelike filling; cast of foreign material filling former cavities such as those developed from decaying roots or burrows of rodents or insects.
9. Massive, or structureless.

### Organic Matter

The organic content affects the physical properties of the soil profile. The water-holding capacity is increased with organic matter. The soil consistency is often affected. Weathering is enhanced by the organic acids that are produced.

Organic matter in the soil profile is composed of a variety of material which ranges from decomposing plant and animal matter to humus. The carbon content is generally used as a measure of the organic content of the soil; the organic content is considered to be 1.724 times the organic carbon content. The carbon to nitrogen ratio, C:N, indicates the amount of decomposition that has occurred in the original organic material.

## SOIL CLASSIFICATIONS

The terms laterite and lateritic soils must be included in any discussion of tropical soils, particularly red tropical soils, in spite of the fact that these terms do not have generally accepted definitions. The word laterite has been defined and redefined many times since it was first used in the early 1,800's.

There have been several pedological classifications of soil adopted in recent years to clarify the description of soils. The most prominent are: The French pedological classification, the U.S. New Soil Taxonomy and the FAO-UNESCO Classification. These classifications are discussed and compared in the latter half of Chapter 3 of Volume I; a summary follows.

The French pedological classification has been used for the only existing regional pedological map of Africa. Three units of red tropical soils, i.e., ferruginous soils, ferrallitic soils and ferrisols are defined in this classification.

Ferruginous soils are found in the lower rainfall areas, generally under 1,830 mm per year in areas with pronounced dry seasons. Ferrallitic soils occur in more humid areas with over 1,500 mm of rain per year and supporting dense vegetation. Ferrisols occur in intermediate to high rainfall areas between 1,250 and 2,750 mm per year, and where erosion is sufficiently high to prevent mature profile development.

The New Soil Taxonomy classification replaces the word laterite with "plinthite" which is defined as a material that indurates irreversibly on exposure to wetting and drying. The term oxisol replaces most applications of the term lateritic soils.

The FAO-UNESCO Classification was organized as a compromise system in order to prepare a Soil Map of the World. To date, only Volume IV, South America has been published but other volumes should be available in the near future. The system has 25 soil units but only nine units represent the red tropical soils, black clays and volcanic soils of the tropics. Most red soils of South America are ferralsols Arenosols, acrisols and luvisol are also common. Cambisols and nitosols are geographically restricted.

An approximate correlation of the French Classification, U.S. New Soil Taxonomy and FAO-UNESCO Systems is given in Table 1.3.

There is no single recommended terminology for engineers. Pedological maps using the terms described above are of value to soils engineers and geologists as well as agricultural scientists. Such maps are available in many areas. The Great Soil Groups classification (an older system) has been used for many of these, at least in South America and Southeast Asia. The French system has been used in Africa. The New Soil Taxonomy has also been adopted in some localities. The summary definitions found in Chapter 3 of Volume I should be of some assistance in utilizing these maps.

The Soil Map of the World has introduced the compromise terminology. It is expected that these terms and maps will be widely used. These terms should be adopted wherever possible. If a soil is named a ferralsol, no other distinguishing term is needed. The term plinthite is also recommended for soils which harden on exposure to repeated wetting and drying. Lateritic gravel should be called concretionary gravel. Hardened crusts should be termed ironstone.

The old term "lateritic soil" may be necessary when pedological maps and details are unavailable. Under these circumstances, both "laterite" and "lateritic soils" may be the most suitable terms. The geological definition of these terms should be adopted for such usage. The American Geological Institute (1972) defines laterite as:

> "A highly weathered, red subsoil or material rich in secondary oxides of iron, aluminum, or both, nearly void of bases and primary silicates, and maybe containing large amounts of quartz and kaolinite. It

**TABLE 1.3**
**Approximate Correlation of the FAO Classification System With Those of the U.S. Soil Taxonomy and the French Classification System (modified from National Academy of Sciences, 1972)**

| FAO | U.S. Soil Taxonomy | French Classification |
|---|---|---|
| FLUVISOLS | Fluvents | Juvenile soils on recent alluvium and colluvium |
| REGOSOLS | Psamments<br>Orthents | Juvenile soils on recent eolian deposits and weakly developed soils |
| ARENOSOLS Ferralic A. | Oxic Quartzipsamments | Ferrallitic soils on loose sandy sediments |
| GLEYSOLS | | Mineral hydromorphic soils |
| Eutric G. | Tropaquepts | |
| Dystric G. | | |
| Humic G. | Humaquepts | |
| Plinthic G. | Plinthaquepts | Hydromorphic soils with an accumulation of iron or a plinthite horizon. |
| ANDOSOLS | Andepts | Eutropic brown soils of tropical regions on volcanic ash |
| PLANOSOLS | | |
| CAMBISOLS | | |
| Dystric C. | Dystropepts | Ferrallitic soils, rejuvenated; |
| Eutric C. | Eutropepts | Ferruginous or ferrallitic soils, rejuvenated; |
| Humic C. | Humitropepts | Ferrallitic soils, humic, rejuvenated |
| LUVISOLS | Tropudalfs<br>Paleudalfs<br>Paleustalfs | Ferruginous tropical soils |
| ACRISOLS | | Yellowish-brown Ferrallitic soils |
| Rhodic A. | Rhodudults<br>Rhodustults | |
| FERRALSOLS | Oxisols | Ferrallitic soils |
| LITHOSOLS | Lithic subgroups | Lithosols and lithic Soils |
| NITOSOLS (some cambisols) | Udalfs (?) | Ferrisols |
| VERTISOL | Vertisols | Vertisols |

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 a treatment of wetting and drying. . ."

The Geological definition of lateritic soil is (AGI 1972):

"A soil containing laterite; also any reddish tropical soil developed from much weathering."

The last term is highly generalized but it is not always possible to be more precise in the field.

## PHYSICAL AND ENGINEERING PROPERTIES

The physical and engineering properties of three tropical soil groups are discussed in Volume I. These are: (1) volcanic soils which are discussed in the beginning of Chapter 4; (2) red tropical soils which are discussed in the latter half of Chapter 4; and (3) black clay soils which are discussed in Chapter 10. The red tropical soils are the most abundant in the tropics. Volcanic and black soils are not as common but they have certain properties which require special consideration in design. Black clay soils are discussed further in Chapter 5 of this volume.

The physical and engineering properties of tropical red soils and volcanic soils are discussed below. In addition some relationships are discussed which can be used to tentatively identify the soils. The discussion is somewhat complex because the information is based on soil analyses undertaken at different times in three tropical regions, i.e., South and Central America, Africa and Southeast Asia. The discussion is further complicated because three different soil classification systems are used in these tropical regions, i.e., FAO, French and Great Soils Groups classifications.

To simplify the discussion, the red tropical and volcanic soils will be discussed separately.

## RED TROPICAL SOILS

The results of the analysis of the soils from three tropical regions was used to show the similarities and differences among the various soil classification. These are summarized briefly.

- Luvisols and ferruginous soils occur in the more arid extremes of lateritic soils, in areas with pronounced dry seasons. They have formed over all rock types. They display lower Atterberg limits, higher densities and CBR values than other.
- Ferralsols, acrisols and ferrallitic soils occur in the more humid extremes for lateritic soils and in areas with dense vegetation. These soils have also formed over all rock types. Ferralsols and Ferrallitic soils may contain plinthite which hardens on exposure; however dehydration does not normally occur because of dense vegetative cover.
- Nitosols, ferrisols and perhaps some cambisols have formed over all types of rocks in intermediate to high rainfall areas where erosion has kept pace with profile development. The high degree of hydration of clay materials is responsible for the similarity of properties among these soils and ferrallitic soils.

Some relationships and correlations are found to exist among the engineering properties of various soil groups. These relationships may be useful to the engineer in the selection of materials for highway construction since preliminary conclusions can be drawn based on routine classification tests.

The first relationship is that between liquid limit and plasticity index of the tropical red soils. The relation is customarily shown on the Casagrande plasticity chart. The combined data for ferralsols, acrisols, arenosols, luvisols and nitosols are shown in Figure 1.2 to illustrate the correlation among these groups.

The equations of the relationships for the individual soil groups are as follows:

$PI = 0.45\ LL - 3.50$ for the ferralsols
$PI = 0.47\ LL - 3.80$ for the acrisols
$PI = 0.82\ LL - 13.95$ for the arenosols
$PI = 0.75\ LL - 12.70$ for the luvisols
$PI = 0.23\ LL + 7.96$ for the nitosols

Most of the soils plot above the A-line or immediately below the A-line. Those that plot far below the A-line are mostly ferralsols and nitosols which have formed over basalt. The implication of this relationship is discussed in the first part of Chapter 4, Volume I.

The combined data for ferruginous, ferrallitic and ferrisols is shown on the Casagrande plasticity chart, Figure 1.3.

The equations of the relationships for the individual soil groups are:

$PI = 0.71\ LL - 8.50$ for the ferruginous soils
$PI = 0.57\ LL - 3.62$ for the ferrallitic soils
$PI = 0.50\ LL - 1.50$ for the ferrisols

In South America it was found that the relationship between optimum moisture content and plastic limit can be expressed by the following general equation:

$$OMC = 0.61\ PL - 0.84$$

Whereas, the relationship between OMC and the clay content can be expressed as:

$$OMC = 0.34\ (2\mu) + 6.16$$

In addition Figure 1.4 shows the ranges in CBR values, for a 95 percent level of confidence, when these soils are grouped according to the AASHO classification system. The use of this figure allows a quick means of providing a rough estimate of the bearing value of a soil when only the AASHO classification is known.

In Africa it was found that the relationship between optimum moisture and dry density at Modified AASHO compaction can be expressed by the following general equation:

$$MDD = 160 - 2.78\ OMC$$

A relationship which could be used as a guide in the selection of base course materials is shown in Figure 1.5. It has been generally accepted that high quality base course materials should have a CBR in excess of 75 under the moisture conditions which will prevail during the design period of the pavement. The data shown in Figure 1.5 have been used to classify the soils into three suitability groups: (1) soils which can be used as base course materials under moisture conditions which exceed the optimum moisture content; (2) soils which can be used as base course materials under moisture conditions which will not exceed the optimum moisture content; and (3) soils which do not have a suitable CBR value under either condition. The relationship is shown as a function of maximum dry density at Modified AASHO compaction and the granulometric modulus, which is the accummulated percentage passing the 1", 3/4", 1/2", 3/8", No. 4, No. 10, No. 40 and No. 200 sieves of the laboratory CBR sample prior to compaction. The soils which conform to the first suitability group are enclosed by the points A, B, B', A. The second group is enclosed by B, C, C', B', B, and the unsuitable soils by A', C, C', A'.

In South East Asia it was observed that the great soil group classifications provide an excellent key to the geographic areas in which laterite may form. Numerous field correlations were made between the agricultural soil maps and laterites observed in-situ throughout Southeast Asia. Based on these observations, some general statements may be made concerning the engineering characteristics of the lateritic soils formed in the various great soil groups, and the utility of using agricultural soil maps for preliminary material source surveys. These are discussed in detail at the conclusion of Chapter 4, Volume I.

## VOLCANIC SOILS

It has been known for a number of years that certain soils found over volcanic materials displayed unusual properties. Many of these soils contain abnormally high moisture contents and yet display surprisingly high strengths. Some even appear granular, but after manipulation become highly plastic saturated clays. Most of these soils change properties with air or oven drying and actually become granular or "granulate", such changes are generally believed to be irreversible.

The phenomenon of change in properties with drying is not restricted to volcanic soils. It occurs as well with other soils but the changes in other soils are generally not very large. Those soils which are highly moisture-sensitive are invariably found over volcanic or basic intrusive rocks and are called Andosols.

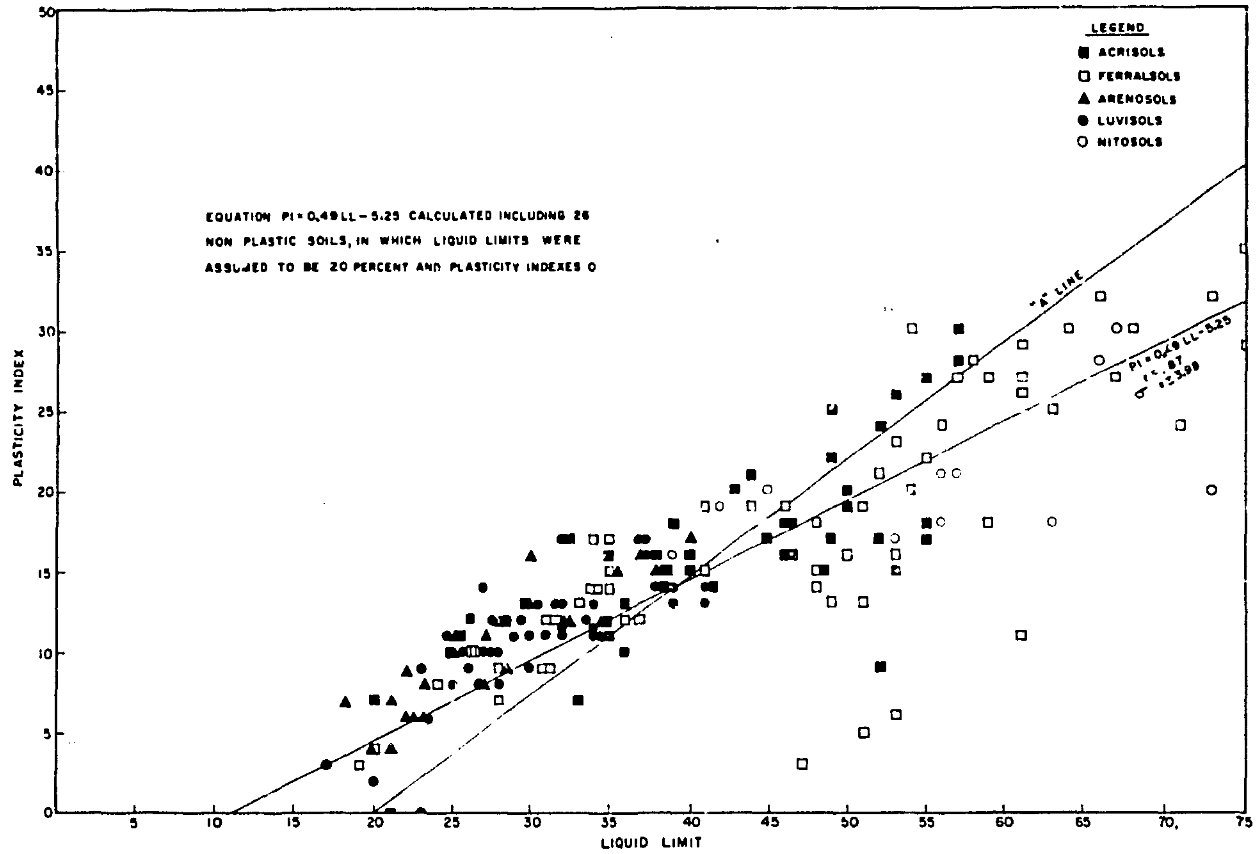



FIGURE 1.2–LOCATIONS OF THE TROPICAL RED SOILS OF SOUTH AMERICA ON THE CASAGRANDE CHART

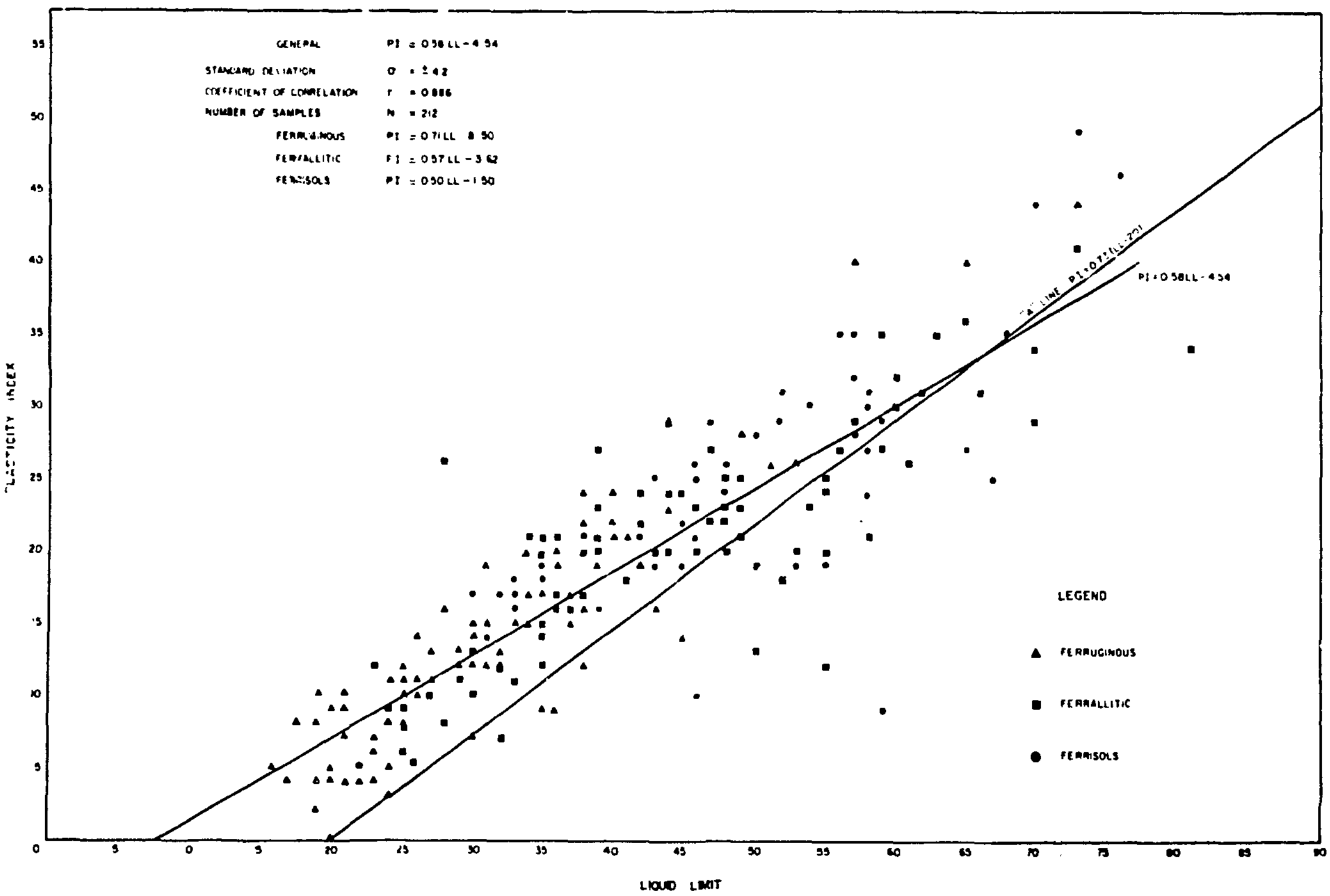



FIGURE 1.3–LOCATIONS OF THE TROPICAL RED SOILS OF AFRICA ON THE CASAGRANDE CHART

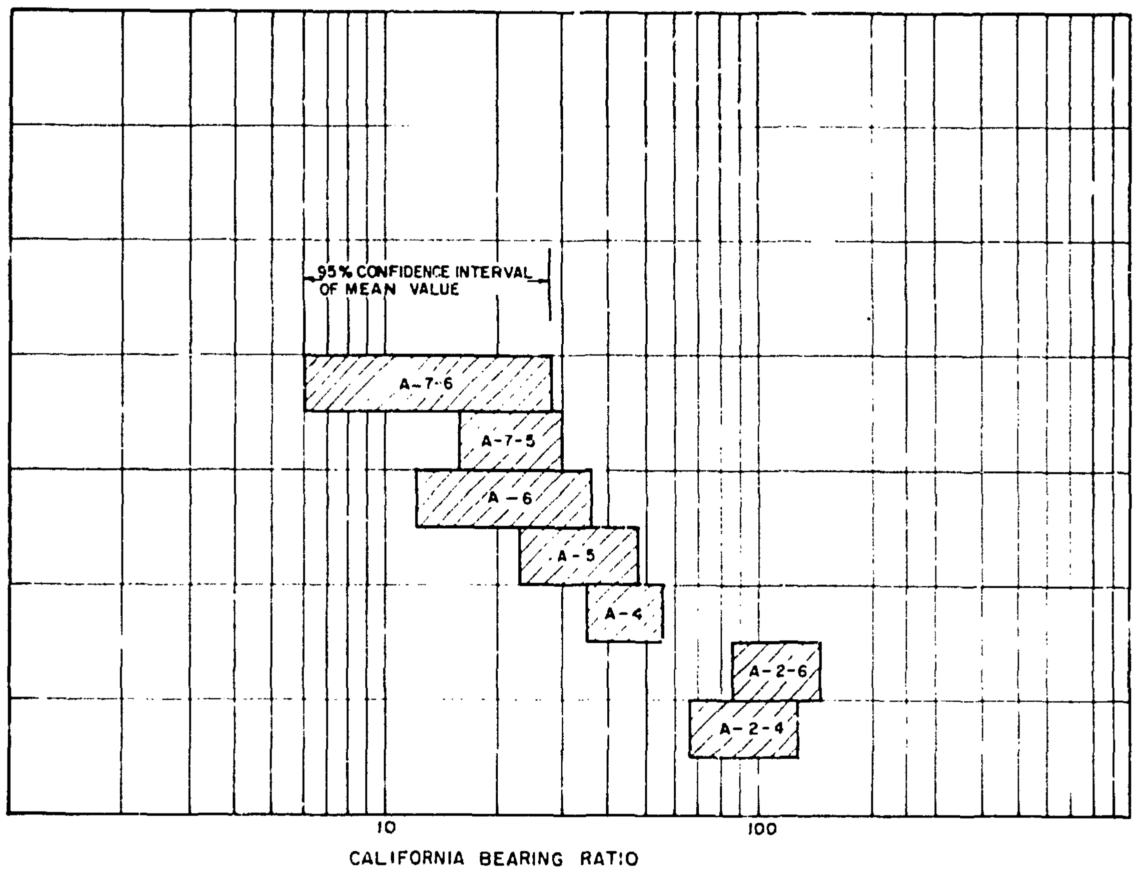



FIGURE 1.4 – GENERAL CORRELATION BETWEEN CBR AND AASHO CLASSIFICATION, FOR RED TROPICAL SOILS OF SOUTH AMERICA

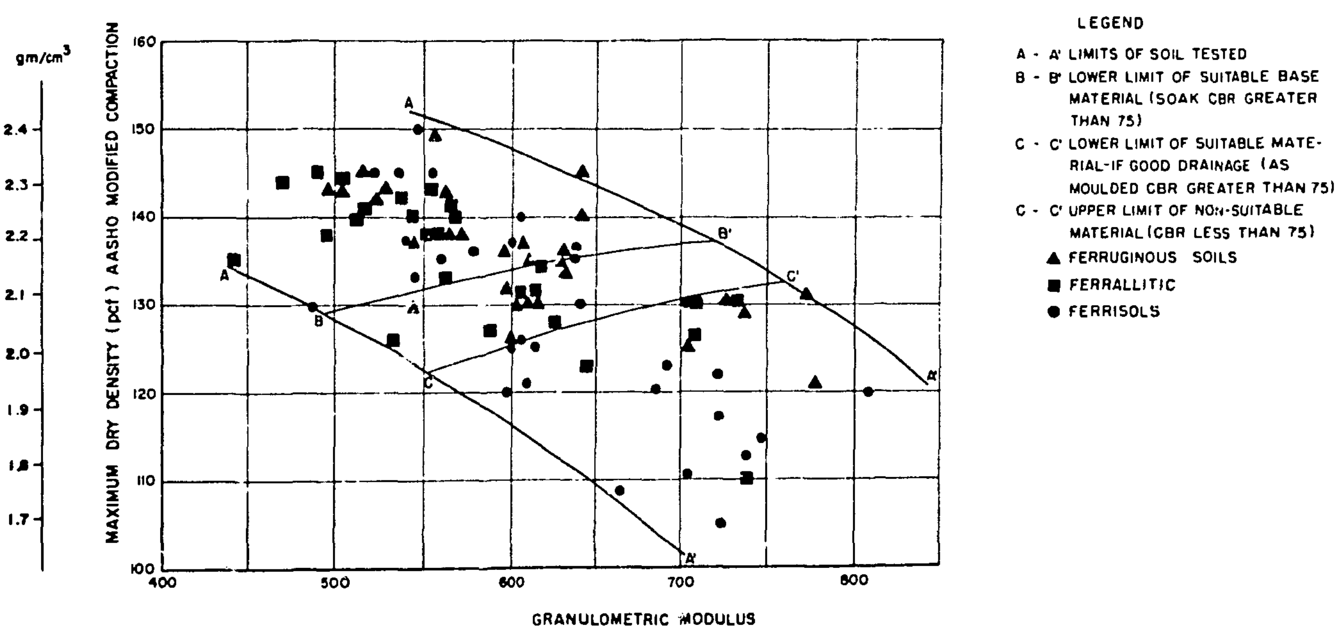



FIGURE 1.5 – SUITABILITY OF MATERIAL AS BASE COURSE ON THE BASIS OF MAXIMUM DRY DENSITY, AT AASHO MODIFIED, AND GRANULOMETRIC MODULUS OF CBR SPECIMEN

The Casagrande chart, Figure 1.6, illustrates the great changes which occur in the andosols with air-drying. The known latosols generally display less change and these tend to be parallel to the A-line. The only sample that lies above the A-line is a latosol.

Recommended highway construction procedures for these soils suggest that shallow excavations should be handled by bulldozer only when it is possible to do so in one pass. All other excavations should be handled by drag-line or shovel. Exposed subgrade should be covered with a minimum of 45 cm of select material. This should serve as a travel-way for construction vehicles and also as the sub-base. Embankments should be constructed with a 45 cm rocky layer placed after every 1.5 m of soil to serve as a travel-way. The compaction should only be that which occurs from construction traffic. End-dumping should be permissible for low, short embankments only.

Changes in properties with air drying can be significant in construction. Laboratory testing, if performed on air-dried samples only, may produce test results on altered soils. If the material will not be similarly air-dried during construction, erroneous values for the Atterberg limits, gradation, and density may be applied.

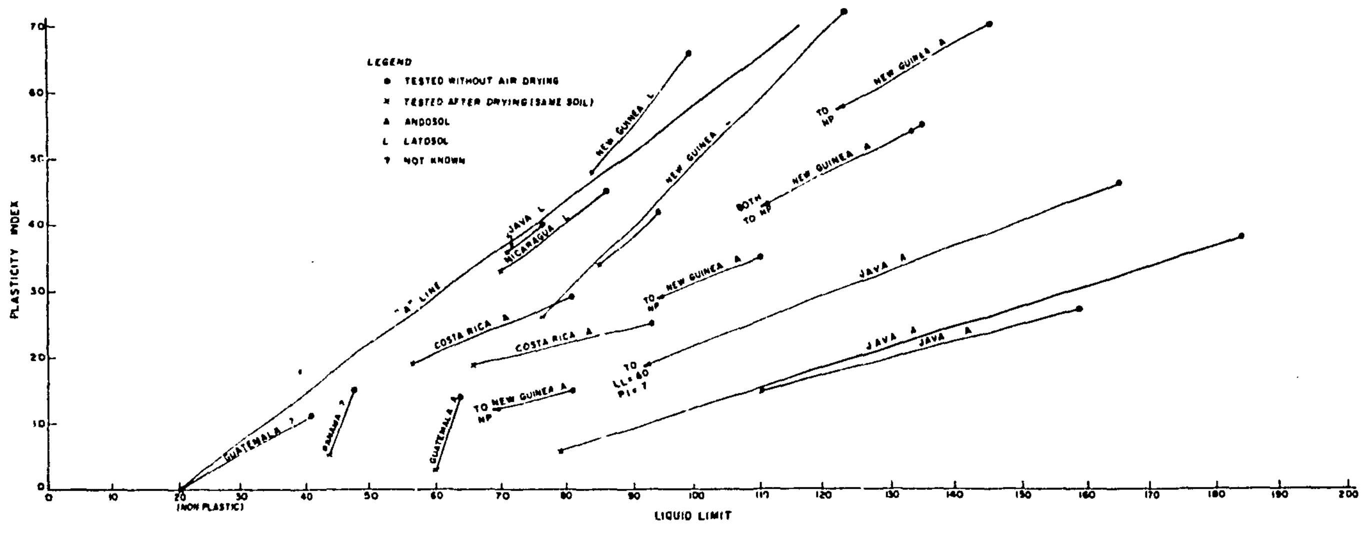



FIGURE 1.6 – LOCATIONS OF SOME OF THE VOLCANIC SOILS ON THE CASAGRANDE CHART

## CHAPTER 2
*TEST PROCEDURES FOR EVALUATION OF TROPICAL SOIL PROPERTIES*

### INTRODUCTION

Well established testing procedures used in engineering evaluations of temperate soils are not always suitable for evaluating tropical soils. Sometimes a modification of the standard tests are necessary in order to obtain a proper evaluation. For example, experience with tropical soils has shown that the procedure of manipulating and pre-heating preparations of temperate soils will change the properties of tropical soils unless the procedure is altered.

The changes in engineering properties that occur with pre-heating prior to testing are usually irreversible. The gradation, Atterberg limits and the moisture-density relationship are all affected. An example of such changes are shown in Figure 2.1. The sample was obtained from a construction site at the Juan Santa Maria International Airport, San Jose, Costa Rica. The testing was conducted by the Departamento Laboratorio de Materiales, de Obras Públicas y Transportes, San Jose. A special study was undertaken since the contractor was unable to obtain the specified compaction which had been determined in the laboratory. The data shown on the left side of Figure 2.1 are the original test results while the data on the right are the test results of the special study. In the latter test the sample was not dried prior to testing. The compaction curve on the right represents the moisture-density relationship of the material as it exists in the field. The subgrade had a natural moisture content of 70 percent, therefore the difficulty in obtaining the specified laboratory density in the field is obvious. However, it should be noted that the problem was restricted only to the compaction since the four-day soaked CBR for both samples was the same. It is not known if the as-molded CBR would have been the same for both samples but it is known that the in-situ CBR's at the site were in the order of 16.

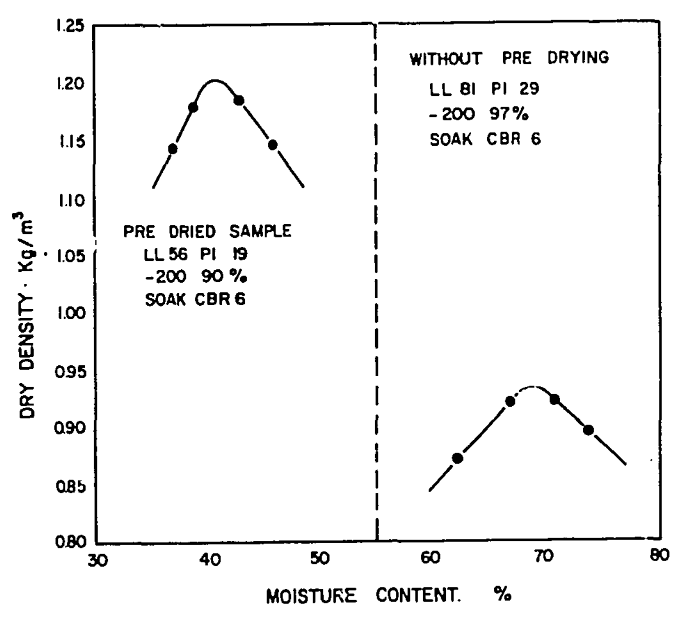



FIGURE 2.1 – COMPARISON OF PHYSICAL PROPERTIES TESTED WITH AND WITHOUT PRE-DRYING (COURTESY OF DEPARTAMENTO LABORATORIO DE MATERIALES, MINISTERIO DE OBRAS PUBLICAS Y TRANSPORTES, SAN JOSE, COSTA RICA)

It is important that such moisture-sensitive soils be identified in preliminary investigations in order to avoid delays during the construction phase. An "aggregation index" is recommended to determine the propensity of a soil to change after dehydration. This index is defined as the sand equivalent value of the soil in its natural state divided by the sand equivalent of the oven-dried sample. An index of two indicates a moderately sensitive soil and an index of 12 indicates a highly sensitive soil.

The effect of drying on the Atterberg limits for several soils tested during the African study is illustrated in Table 2.1. Tests were conducted initially without drying (at the field moisture content), with air drying and with oven drying at various temperatures and drying time. The data appears to indicate that temperature causes the greatest change and the time of drying is secondary.

Variations also occur in the Atterberg limits depending upon the amount of manipulation of the sample prior to testing. Excessive manipulation prior to testing leads to breakdown of the soil structure and disaggregation. Both consequences produce fines which results in higher liquid limit values.

The hydrometer analysis is a particularly difficult test in that it is often difficult to reproduce results. The strong tendency of tropical soils to aggregate or flocculate presents a problem in dispersing the soils prior to testing.

### TEST PROCEDURES

The following are the recommended testing procedures to be used in engineering evaluation of tropical soils. Recommendations and modifications are suggested in view of the nature of these soils mentioned above.

| Test | AASHO | British Std. |
|---|---|---|
| 1) *Dry Preparation of Soil Samples* | T 87-70 | 1377 Part 2 Sec. 4 |

*Comments:* It is recommended that the soil be air dried regardless of relative humidity and that oven drying be avoided.

| Test | AASHO | British Std. |
|---|---|---|
| 2) *Wet Preparation of Soil Samples* | T 146-49 | 1377 |

*Comments:* It is recommended that the soil be air dried and oven drying be avoided.

| Test | AASHO |
|---|---|
| 3) *Preparation of Soil Samples at Natural Moisture Content* | NA |

*Comments:* This procedure has not been standardized by either AASHO or ASTM. It is a special preparation procedure and was developed to facilitate the testing of

**TABLE 2.1**
**Change in Atterberg Limits After Drying at Various Temperatures and Time Periods**

| Soil Samples | | As Received | Air Dried | 6 hours at 50°C | 24 hours at 50°C | 6 hours at 105°C | 24 hours at 105°C |
|---|---|---|---|---|---|---|---|
| A | LL | 63.4 | 62.2 | 60.1 | 60.7 | 57.3 | 55.1 |
| | PL | 39.1 | 31.2 | 27.2 | 28.7 | 27.4 | 28.4 |
| B | LL | 54.3 | 47.6 | 47.1 | 44.8 | 41.8 | 41.9 |
| | PL | 22.4 | 22.9 | 22.2 | 24.1 | 23.1 | 22.4 |
| C | LL | 51.8 | 44.6 | 44.7 | 45.5 | 42.8 | 42.9 |
| | PL | 29.8 | 26.8 | 23.9 | 22.9 | 19.8 | 20.3 |
| D | LL | 45.2 | 40.0 | 40.5 | 41.9 | 41.0 | 37.6 |
| | PL | 21.7 | 21.0 | 21.6 | 21.8 | 24.8 | 20.7 |
| E | LL | 36.5 | 34.5 | 36.1 | 36.8 | 36.0 | 35.2 |
| | PL | 38.2 | 30.2 | 31.4 | 29.1 | 26.3 | 28.2 |
| F | LL | 29.0 | 26.5 | 26.2 | 26.5 | 25.8 | 24.7 |
| | PL | 21.4 | 16.6 | 15.6 | 14.8 | 14.1 | 13.8 |
| G | LL | 65.0 | 62.9 | 58.3 | 58.5 | 49.4 | 46.0 |
| | PL | 27.2 | 27.8 | 29.4 | 28.5 | 24.4 | 25.9 |
| H | LL | 61.6 | 53.4 | 54.4 | 53.9 | 44.4 | 42.7 |
| | PL | 28.2 | 25.5 | 26.2 | 24.8 | 23.6 | 23.8 |
| J | LL | 45.7 | 44.2 | 44.4 | 44.4 | 43.4 | 44.4 |
| | PL | 23.2 | 23.5 | 25.9 | 25.9 | 26.2 | 26.1 |

tropical soils at their natural moisture content. This was necessary because some tropical soils, particularly andosols, exhibit changes in engineering properties with drying. Two factors which cause the change in properties with drying are: (1) the tendency to form aggregation on drying and (2) the loss of water in hydrated minerals. The first generally results in an increase in strength while this is not necessarily true with the second effect.

The following is the procedure for preparing a sample for the Atterberg limit tests:

Wet Method:

1) Break up the required amount of material with rubber-covered pestle or rolling pin.
2) Transfer sample to saucepan and cover with water. Let soak until all material is disintegrated. This may require 2 to 12 hours.
3) Place a No. 40 sieve in a saucepan and transfer entire soaked sample into the sieve. Wash any material still adhering to the soaking pan into the sieve by squirting water from a battery filler.
4) Pour clean water into pan containing sieve until level of water is about 1/2 inch above mesh in sieve.
5) Agitate the sieve with one hand without lifting the sieve. Concurrently, stir material with the other hand until all fine material appears to have passed through sieve.
6) Hold sieve slightly above water surface in pan and squirt water from battery filler onto sieve until retained particles and the sieve are clean. Discard material retained in sieve.
7) Place pan where it will not be disturbed and block it up on one side so water on the other side barely reaches rim of pan. Allow soil to settle for several hours.
8) Pour off liquid slowly by gradually increasing tilt of pan until cloudy layer overlaying the sediment reaches rim of pan.
9) Air-dry material to a smooth paste consistency and put in small mixing dish.

For most lateritic soils, material in suspension will settle out. If there is no indication of this after several hours, the following method may be used.

Place filter paper in a funnel and place wet soil inside the funnel in a jar or other container and allow to stand until all the excess water is filtered off.

The procedure of conducting the Atterberg test is the same with the exception that the low count (high moisture) is established first. The material is allowed to dry and the second point is established. This procedure is followed until a flow-curve is developed that will define the moisture content at 25 blows. The Plastic Limit is then determined after the Liquid Limit Test.

For many of these tropical soils that change properties the amount of +40 material is such a minor constituent that it is often unnecessary to sieve the material prior to conducting the liquid limit test.

The following is an outline of the procedure to follow when preparing a sample at its natural moisture content for establishing the moisture-density relationship.

1) The sample at natural moisture content is passed through a 3/4 inch sieve.

2) The sample is split into five more or less equal parts each of which are sufficient for compaction in a six inch mold.

3) Two percent moisture is added to one sample and allowed to cure in a plastic bag or sealed container for a minimum of 12 hours prior to compaction; 18 to 24 hours would coincide better with normal working hours.

4) A second sample is then compacted at its natural moisture content with the appropriate compactive effort and prepared for CBR testing if required.

5) The remaining three samples are permitted to air dry for different periods of time prior to compacting the sample.

| Test | AASHO | British Std. |
|---|---|---|
| 4) *Particle Size Analysis* | T 88-70 | 1377 Test 6 |

*Comments:* It is not recommended to dry the soil prior to testing. It is recommended that the mechanical analysis be performed at the natural moisture content. A moisture correction must be applied prior to computations. The appropriate apparatus for performing the sieve analysis is shown in Figure 2.2.

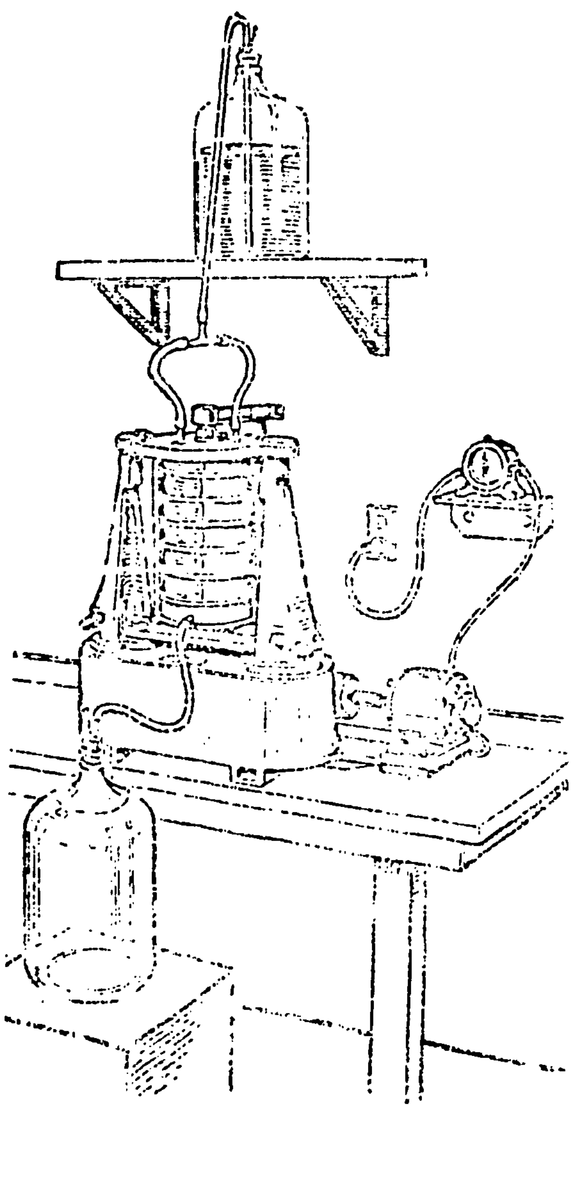


FIGURE 2.2 – WET TEST SETUP WITH MECHANICAL SIEVE SHAKER (ASTM 1969)

Sodium hexametaphosphate should be used as the dispersing agent in the hydrometer analysis. The dispersing time should be 15 minutes. Experience has shown that sedimentation tests are difficult to perform on tropical soils It should be remembered that Stokes' law of sedimentation does not give the actual diameter of the particles but only the diameter of an equivalent sphere. The diameter of a clay plate can be five times greater than the one determined from Stokes' law. Any sedimentation test, no matter how accurately performed, gives only a general indication of the size and quantity of soil particles.

| Test | AASHO | British Std. |
|---|---|---|
| 5) *Liquid Limit* | T 89-68 | 1377 Test 2A |

*Comments:* The "Single-point Method" evaluated during the African study can be used. Figure 2.3 shows the comparison of test results of the South American soils obtained in the standard laboratory procedure plotted against the results obtained with the African equation

$$LL = W \frac{(N)^{0.15}}{25}$$

where:

W = water content at N blows percent

N = number of blows.

A maximum of five minutes mixing time is recommended because tropical soils are susceptible to breakdown with manipulation.

| Test | AASHO | British Std. |
|---|---|---|
| 6) *Plasic Limit and Plasticity Index* | T 90-70 | 1377 Test 6 |

| Test | AASHO | |
|---|---|---|
| 7) *Moisture Density Relations* | T 99-70 | 2. |
| | T 180-70 | 2. |

*Comments:* After mixing the samples with the various percentages of water the sample should be sealed in an air-tight container and allowed to cure for a period of 12 hours to insure a homogeneous mixture prior to compaction. When significant amounts of gravel size materials are present which are hard and impermeable large moisture samples are necessary. The following quantities are recommended for moisture determinations:

10 grams for minus No. 40 material
200 grams for minus No. 4 material
1,000 grams for minus 3/8 inch material
2,000 grams for minus 3/4 inch material

| Test | AASHO | British Std. |
|---|---|---|
| 8) *Specific Gravity* | T 100-70 | 1377 Test 5 |

*Comments:* When the moisture density relation is established without drying the specific gravity should also be determined without allowing the sample to dry prior to testing. A moisture correction is used prior to computations.

| Test | AASHO |
|---|---|
| 9) *California Bearing Ratio* | T 192-63 |

*Comments:* The recommendations given for AASHO T 99 and T 180-70 should be followed in compaction. If initial testing indicates that the soil is moisture sensitive, the compaction should be accomplished without predrying.

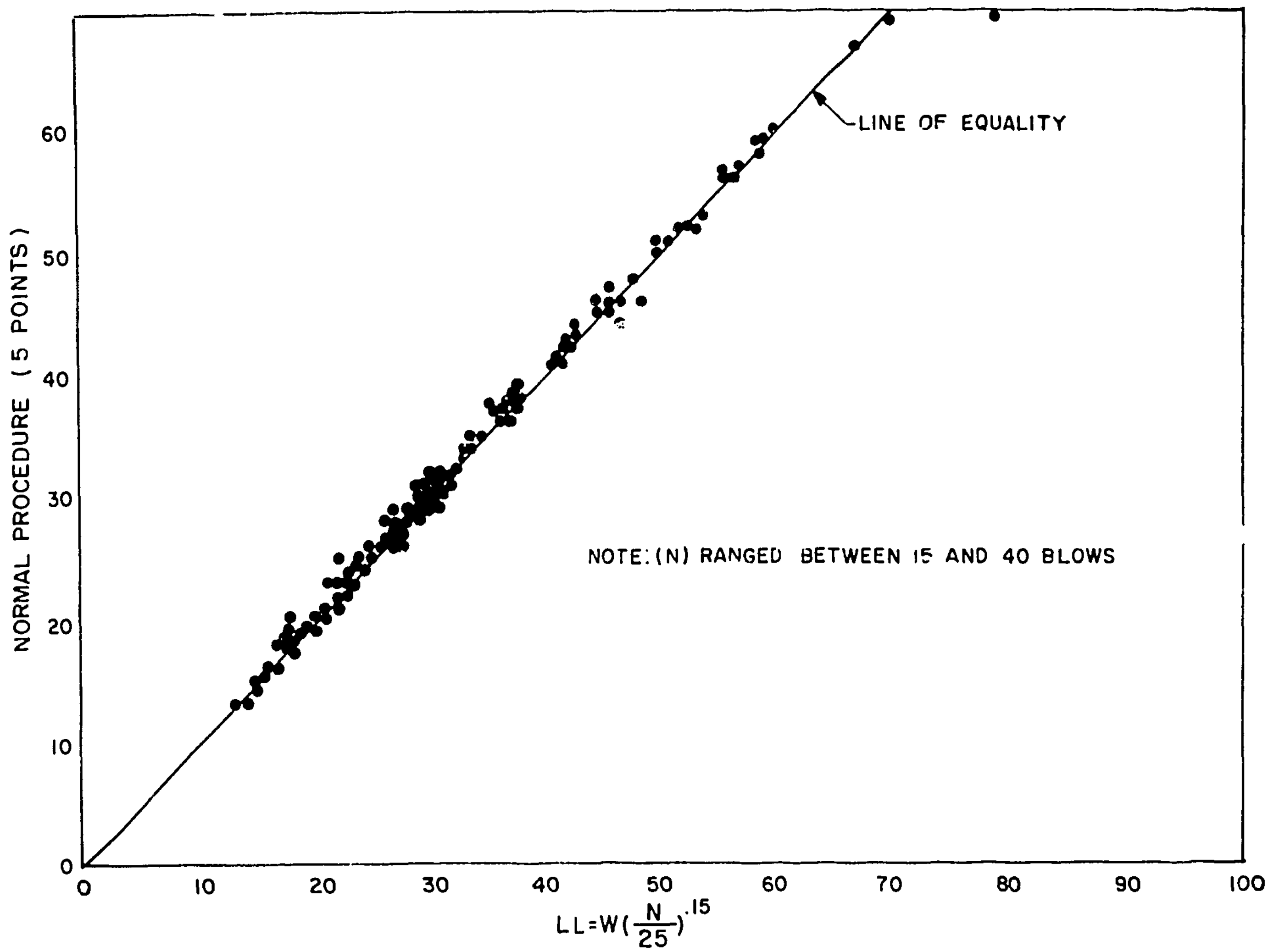



FIGURE 2.3 – COMPARISON BETWEEN 5 POINTS LIQUID LIMIT AND LIQUID LIMIT DETERMINED BY SINGLE POINT METHOD

| Test | AASHO |
|---|---|
| *10) Sand Equivalent Value* | T 176-70 |

*Comments:* The susceptibility to change of engineering properties due to dehydration can be estimated by the aggregation index, the ratio of the oven dried sand equivalent value to the sand equivalent value at the field moisture content. The sand equivalent test is run in accordance with AASHO T 176-70 except for the field moisture sample which is not dried prior to testing. An aggregation index value greater than 2 indicates the soil may be susceptible to change in engineering properties upon drying.

The following three test procedures may not be readily available. These are reproduced in their entirety from the sources provided.

* Reproduction from test Method No. Calif. 229-C, July, 1963, Materials and Research Department – State of California, Department of Public Works, Division of Highways.

*11) Method of Test for Durability of Aggregates* (Mechanical Agitation in Water)*

**Water)**

### Scope

This method describes the procedure for determining the durability factor of aggregates. The durability factor is a value indicating the relative resistance of an aggregate to producing detrimental claylike fines when subjected to the prescribed mechanical methods of degradation.

### Procedure

#### A. APPARATUS

1. Mechanical washing vessel ("pot"): A flat bottom, straight sided cylindrical vessel conforming to the specifications and dimensions shown in Fig. 2.4a. This vessel is

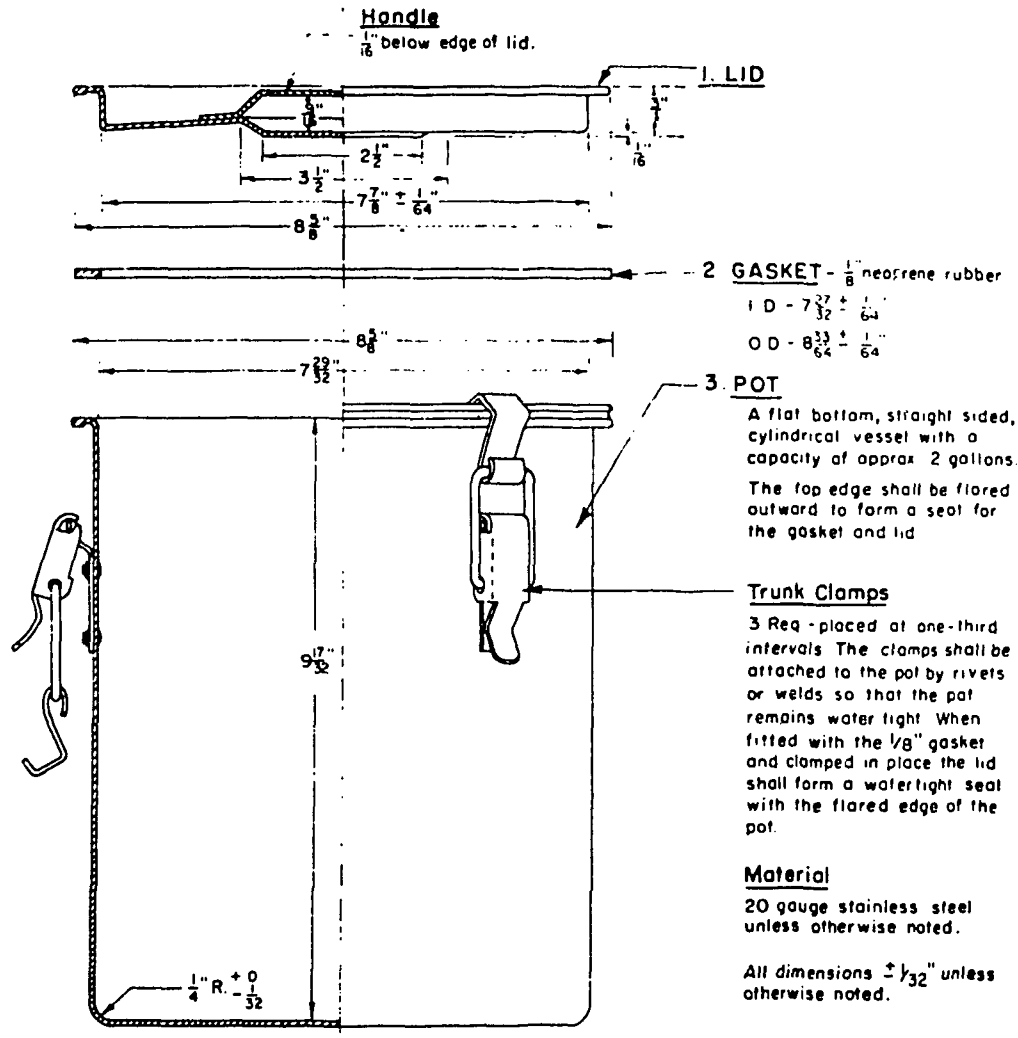



FIGURE 2.4a – MECHANICAL WASHING VESSEL

available to California State Agencies from the Service and Supply Department of the Division of Highways (Stock No. 69,577 NR).

2. Vessel: A round pan suitable to collect the wash water from the washed sample.

3. Agitator: A Tyler portable sieve shaker, modified as shown in Fig. 2.4b and set to operate at 285 ± 10 complete cycles per minute. The two agitation periods specified under E, Preparation of Sample, and F, Test Procedure are for this modified shaker. Other types of sieve shakers may be used, provided the length of time and/or other factors are adjusted so that results can be obtained which duplicate the results obtained with the modified Tyler portable sieve shaker. See Figure 2.4c for a photograph of the mechanical washing vessel secured in position in the standard mechanical agitator.

4. Graduated cylinders of 10 ml and 1,000 ml capacities.

5. A graduated plastic cylinder, rubber stopper, irrigator tube, weighted foot assembly and siphon assembly all conforming to their respective specifications and dimensions shown in Fig. 2.4d.

A sand equivalent test kit, which contains the necessary equipment, except for a 1-gal. bottle, is available to California State agencies from the Service and Supply Department of the Division of Highways (Stock No. 69,690 NR). Fit the siphon assembly to a 1-gal. bottle of working calcium chloride solution placed on a shelf 3 ft. ± 1 in. above the work surface (See Fig. 2.4e). In lieu of the specified 1-gal. bottle, a glass or plastic vat having a larger capacity may be used providing the liquid level of the working solution is maintained between 36 and 46 inches above the work surface.

6. Measuring tin: A 3-oz. tinned box approximately $2\frac{1}{4}$ in. in diameter with Gill style cover, and having a capacity of 85 ± 5 ml.

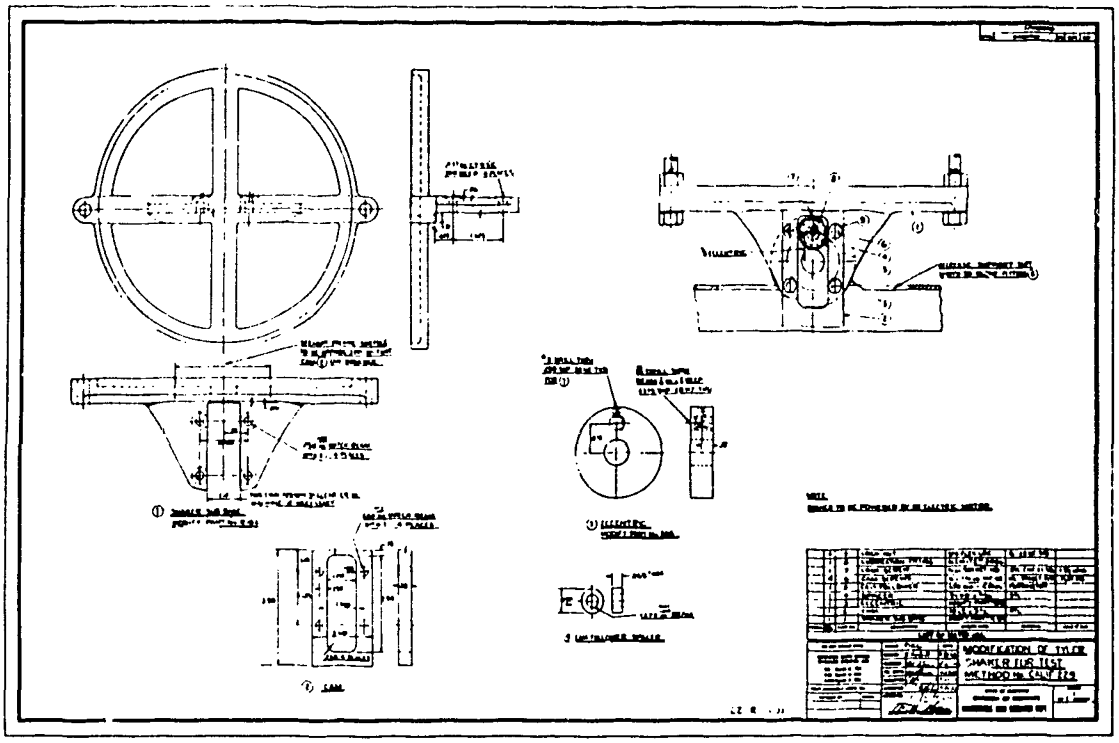


FIGURE 2.4b – MODIFICATION OF TYLER SHAKER FOR TEST

7. Funnel: A wide-mouth funnel approximately 4 in. in diameter at the mouth.

8. Clock or watch: A clock or watch reading in minutes and seconds.

9. Mechanical Sand Equivalent Shaker[1]. A shaker conforming to the specifications and dimensions shown in the State of California, Division of Highways, Materials and Research Department Plans Designation D-256 (See Fig. 2.4f). Prior to use, fasten the mechanical sand equivalent shaker securely to a firm and level mount and disconnect the timer so that the sample can be agitated continuously for the prescribed 10-minute shaking time.

10. Sieves: The sieves shall be of the woven wire type with square openings and shall conform to the "Standard Specifications for Sieves for Testing Purposes," AASHO Designation M-92.

11. A balance or scale with a minimum capacity of 5,000 grams and sensitive to 1 gram.

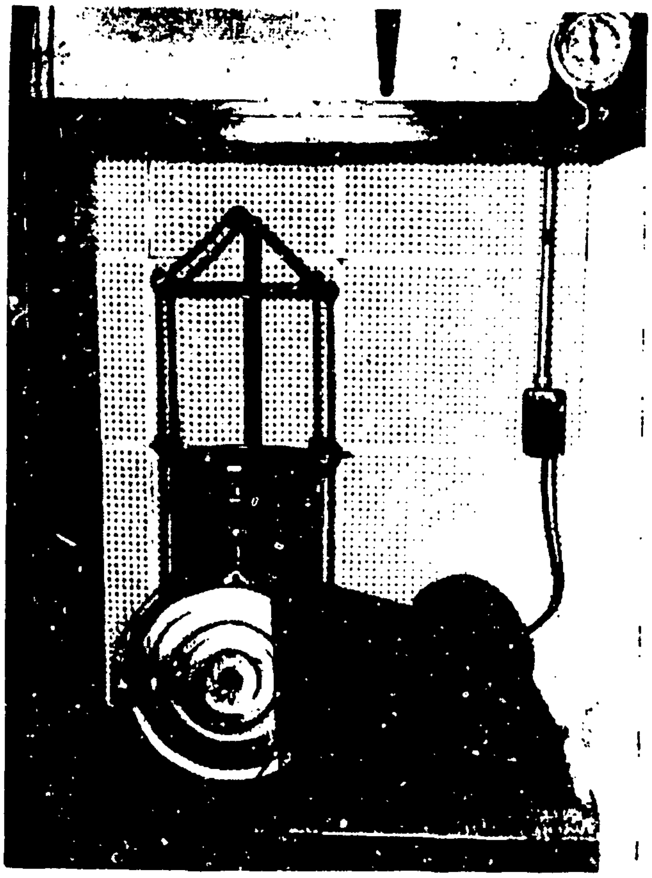


FIGURE 2.4c – MECHANICAL WASHING VESSEL IN TYLER SHAKER

## B. MATERIALS

1. Stock calcium chloride solution (same as stock solution used in Sand Equivalent Test) consisting of:

454 g (1 lb.) tech. anhydrous calcium chloride
2,050 g (1,640 ml) U.S.P. glycerine
47 g (45 ml) formaldehyde (40 percent by volume solution).

1 This mechanical shaker is a modification of shaker designs originally developed by Henry Davis of the California Division of Highways, and by the Laboratorie Centrol des Ponts et Chaussees, Paris, France, under the direction of Mr. R. Peltier. The mechanical shaker is available to California Division of Highways agencies from the Service and Supply Department, Stock No. 69,849 NR.

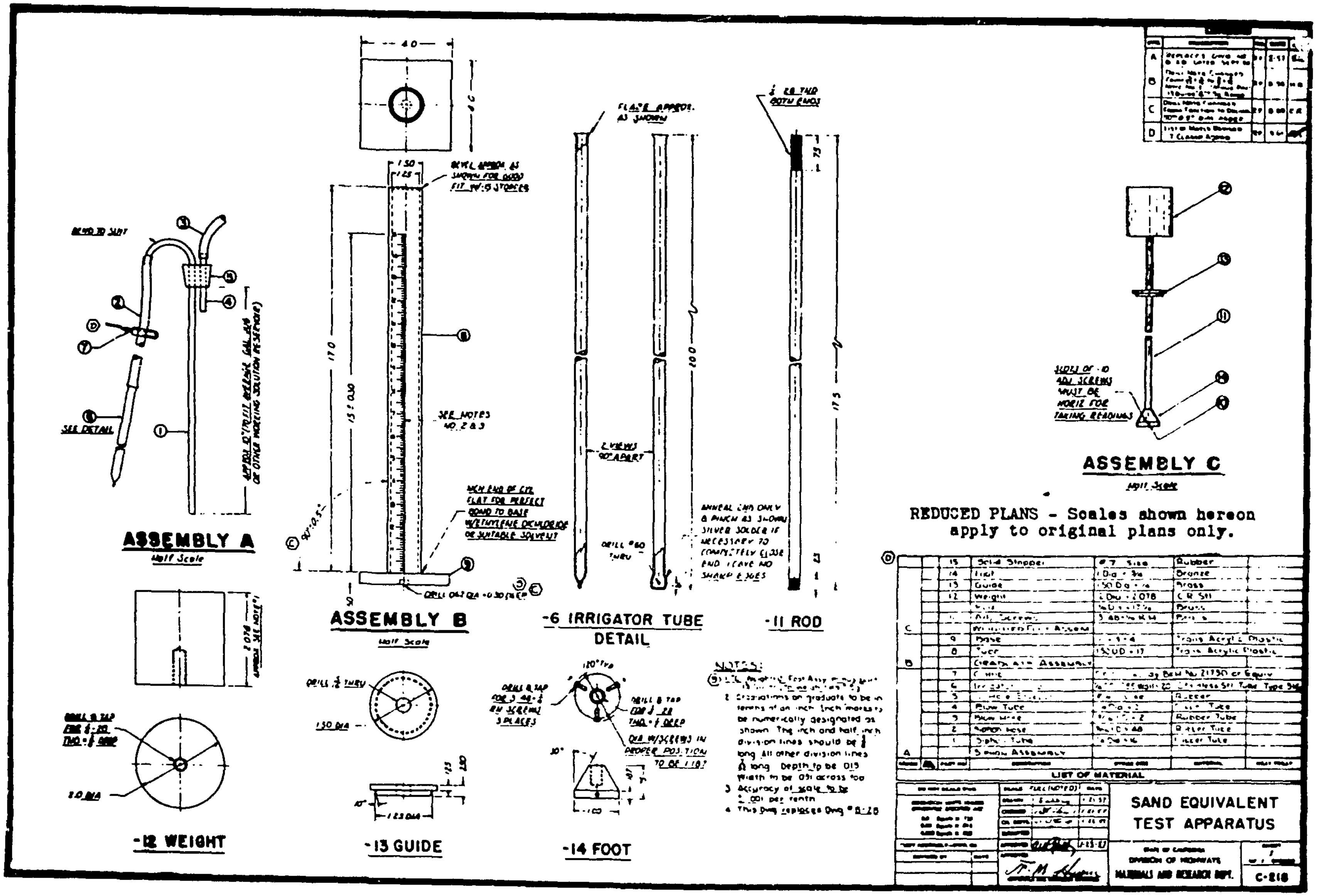



FIGURE 2.4d — SAND EQUIVALENT TEST APPARATUS

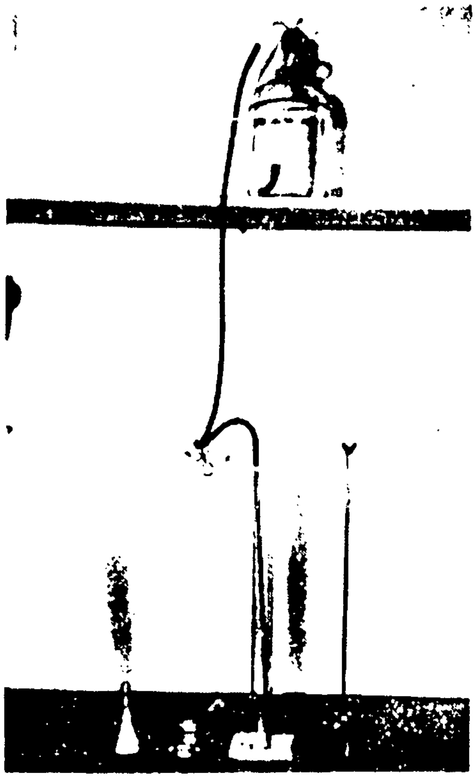


FIGURE 2.4e – SAND EQUIVALENT TEST APPARATUS EXCLUDING SHAKER

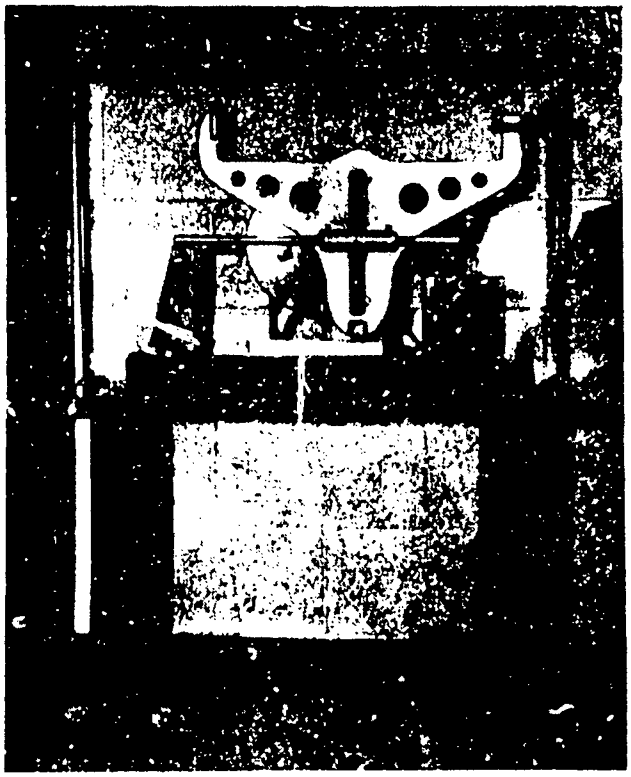


FIGURE 2.4f – MECHANICAL SAND EQUIVALENT SHAKER

Dissolve the calcium chloride in 1/2 gal of distilled or demineralized water. Cool the solution, then filter it through Whatman No. 12 or equivalent filter paper. Add the glycerine and formaldehyde to the filtered solution, mix well, and dilute to 1 gal. with distilled or demineralized water. District laboratories should secure stock calcium chloride solution from the Service and Supply Department of the Division of Highways (Stock No. 69,691).

2. Working calcium chloride solution: Prepare the working calcium chloride solution by diluting one measuring tin full (85 ± 5 ml) of the stock calcium chloride solution to 1 gal. with water. Use distilled or demineralized water for the normal preparation of the working solution. However, if it is determined that the local tap water is of such purity that it does not affect the test results, it is permissible to use it in lieu of distilled or demineralized water except in the event of dispute.

3. Water: Use distilled or demineralized water for the normal performance of this test. This is necessary because the test results are affected by certain minerals dissolved in water. However, if it is determined that the local tap water is of such purity that it does not affect the test results it is permissible to use it in lieu of distilled or demineralized water except in the event of dispute.

### C. TEST RECORD FORM

Record test results on Form T-200 or T-361.

### D. CONTROL

This test may be normally performed without strict temperature control; however, in the event of dispute retest the material with the temperature of the distilled or demineralized water and the working calcium chloride solution at 72 ± 5 F.

### E. PREPARATION OF SAMPLE

1. Prepare the sample as described in Test Method No. Calif 201. Care should be exercised in cleaning the coarse aggregate and breaking up of clods so that the method used does not appreciably reduce the natural individual particle sizes.

2. Separate the sample on the 3/4-inch, 1/2-inch, 3/8-inch and No. 4 sieves. Set aside that portion of the material retained on the 3/4-inch sieve. Weigh and record the weights of material retained on the 1/2-inch, 3/8-inch and No. 4 sieves.

3. Preparation of Coarse Aggregate Test Sample.

a. Determine the grading to be used in preparing each preliminary test sample as follows:

(1) If each of the aggregate sizes listed below represents 10 percent or more of the 3/4"x No. 4 portion, as determined from the weights recorded in paragraph 2 above, use the oven dry weights of material specified below for preparing each preliminary test sample.

| Aggregate size | Oven dry weight-grams |
|---|---|
| 3/4" x 1/2" | 1,050 ± 10 |
| 1/2" x 3/8" | 550 ± 10 |
| 3/8" x No. 4 | 900 ± 5 |
| Test Sample Weight | 2,500 ± 25 |

(2) If any of the aggregate sizes listed in above represents less than 10 percent of the 3/4" x No. 4 portion, use the same percentage of material from the deficient aggregate size or sizes as was determined from the weights recorded in paragraph 2 above and proportionally increase the weight of the remaining size or sizes to obtain the 2,500-gram preliminary test sample weight.

Example 1 – Less than 10% of 3/4" x 1/2" aggregate size material

| Aggregate size | Percent each size | Calculations | Oven dry weight-grams |
|---|---|---|---|
| 3/4" x 1/2" | 6 | .06 x 2500 | 150 ± 10 |
| 1/2" x 3/8" | 26 | $\frac{550(2500-150)}{550+900}$ | 891 ± 10 |
| 3/8" x No. 4 | 68 | $\frac{900(2500-150)}{550+900}$ | 1459± 5 |
| Test Sample Weight | | | 2500± 25 |

b. Prepare two 2,500-gram preliminary test samples using the prescribed grading. Dry the test samples to constant weight at a temperature of 221 to 230 F.

c. After allowing the oven dried material to cool, place one of the preliminary test samples in the mechanical washing vessel, add 1,000 ± 5 ml of distilled or demineralized water, clamp the vessel lid in place and secure the vessel in the sieve shaker.

Example 2 – Less than 10% of 3/4" x 1/2" and 1/2" x 3/8" aggregate size materials

| Aggregate size | Percent each size | Calculations | Oven dry weight-grams |
|---|---|---|---|
| 3/4" x 1/2"– | 4 | .04 x 2500 | 100±10 |
| 1/2" x 3/8"– | 7 | .07 x 2500 | 175±10 |
| 3/8" x No. 4 | 89 | 2500–(100+175) | 2225± 5 |
| Test Sample Weight | | | 2500±25 |

d. Begin agitation after a time of *1 minute ± 10 seconds* has elapsed from the introduction of the wash water. Agitate the vessel in the sieve shaker for *two minutes ± 5 seconds.*

e. After the two-minute agitation time is completed, remove the vessel from the shaker, unclamp the lid and pour the contents into a No. 4 sieve. Rinse any remaining fines from the vessel onto the sieve and direct water (from a flexible hose attached to a faucet) onto the aggregate until the water passing through the sieve comes out clear.

f. Wash the second preliminary test sample in the same manner as prescribed above then combine all of the washed material obtained from both preliminary test samples and dry to constant weight at a temperature of 221 to 230 F.

g. After allowing the oven dried material to cool, separate the washed coarse aggregate on the 1/2-inch, 3/8-inch and No. 4 sieves. Discard the material passing the No. 4 sieve.

h. Prepare the washed test sample as follows:

(1) If the preliminary test samples were prepared using the weights specified in a.(1) above, prepare the washed test sample using the weights specified from representative portions of each size of washed material. Occasionally it may be necessary to wash a third preliminary test sample to obtain the required weight of material of a specific size.

(2) If the weights of material prescribed were adjusted as prescribed in the preparation of the preliminary test sample, use *all* of the material representing the deficient size or sizes obtained from washing the two preliminary test samples and proportionally increase the weight of the remaining size or sizes to obtain the 2,500-gram washed test sample.

4. Preparation of Fine Aggregate Test Sample

a. Split or quarter a representative portion from the material passing the No. 4 sieve of sufficient weight to obtain an oven dry weight of 500±25 grams.

b. Dry this preliminary test sample to constant weight at a temperature of 221 to 230 F. Cool to room temperature.

c. Place this preliminary test sample in the mechanical washing vessel, add 1,000±5 ml of distilled or demineralized water, clamp the vessel lid in place. Secure the vessel in the sieve shaker in sufficient time to begin agitation after ten minutes±30 seconds has elapsed from the introduction of the wash water. Agitate the vessel for a period of two minutes ±5 seconds.

d. After the two minute agitation period is completed, remove the vessel from the shaker, unclamp the lid and carefully pour the contents into a No. 200 sieve. Rinse any remaining fines from the vessel into the sieve. Direct water (from flexible hose attached to a faucet) onto the aggregate until the water passing through the sieve comes out clear.

e. It may be necessary to flood clayey or silty samples prior to pouring them over the sieve to prevent clogging the No. 200 sieve. Flood by adding water to the vessel following the agitation period. This dilutes the wash water and reduces its tendency to clog the sieve. Repeated flooding may be necessary in extreme cases before all of the contents of the vessel can be poured over the sieve.

f. Following the rinsing, transfer the material from the sieve to a drying pan, and dry to constant weight at a temperature of 221 to 230 F. *It is necessary to wash the* material from the No. 200 sieve in order to transfer the retained material to a drying pan. Leave the pan in a slanting position until the free water that drains to the lower side becomes clear, then pour off this clear water. Use large shallow pans and spread the sample as thinly as possible to speed drying.

g. After allowing the oven dried material to cool, mechanically sieve the washed test sample for 20 minutes using the following nested sieves: No. 8, No.16, No. 30, No. 50, No. 100 and No. 200. Place a pan below the No. 200 sieve to catch that portion of the material passing the No. 200 sieve. Refer to Test Method No. Calif. 202 for general instructions on sieving procedure.

h. After sieving the washed test sample recombine all of the material retained on each sieve with the material passing the No. 200 sieve that was caught in the pan.

i. Split or quarter sufficient amount of the washed and sieved material to fill the 3-ounce measuring tin to the brim or slightly rounded above the brim. While filling tin measure, tap the bottom edge of the tin on a work table or other hard surface to cause consolidation of the material and allowing the maximum amount to be placed in the measuring tin. Use extreme care in this procedure to obtain a truly representative sample. If the quartering method is used, follow the procedure as specified for "Hand quartering of samples weighing less than 25 lb." in Test Method No. Calif. 201.

## F. TEST PROCEDURE

1. Test Procedure for Coarse Aggregate

a. Place the plastic cylinder on a work table which will not be subjected to vibrations during the performance of the sedimentation phase of the test. Pour 7 ml of the *stock calcium chloride solution into the cylinder.* Place a No. 8 and No. 200 sieve on the pan or vessel provided to collect the wash water with the No. 8 sieve on top. The No. 8 sieve serves only to protect the No. 200 sieve.

b. Place the prepared aggregate sample in the mechanical washing vessel Then add 1,000±5 ml distilled or emineralized water, clamp the lid in place and secure the vessel in the sieve shaker. Begin agitation after a time of 1 min has elapsed from the introduction of the wash water. Agitate the vessel for 10 minutes ± 15 seconds.

c. Immediately following the 10 min agitation period, take the vessel from the sieve shaker and remove the lid. Then agitate the contents of the vessel by moving the upright vessel vigorously in a horizontal circular motion five or six times in order to bring the fines into suspension.

Immediately pour all of the contents of the vessel into the nested No. 8 and No. 200 sieves placed in the pan provided to collect the wash water.

d. Add enough distilled or demineralized water to bring the volume of dirty wash water to 1,000 ± 5 ml. Then transfer the wash water to a vessel suitable for stirring and pouring.

e. Place funnel in the graduated plastic cylinder. Stir the wash water with the hand to bring the fines into suspension. While the water is still turbulent pour enough of the wash water into the cylinder to bring the level of the liquid to the 15 in. mark.

f. Remove the funnel, place the stopper in the end of the cylinder, and prepare to mix the contents immediately.

g. Mix the contents of the cylinder by alternately turning the cylinder upside down and right side up, allowing the bubble to completely traverse the length of the cylinder *20 times* in approximately *35 seconds.*

h. At the completion of the mixing process place the cylinder on the work table and remove the stopper. Allow the cylinder to stand undisturbed for *20 minutes* ± 15 seconds. Then immediately read and record the height of the sediment column to the nearest 0.1 inch.

i. There are two unusual conditions that may be encountered in this phase of the test procedure. One is that a clearly defined line of demarcation may not form between the sediment and the liquid above it in the specified 20-minute period. If this happens, and the test is being made with distilled or demineralized water, allow the cylinder to stand undisturbed until the clear demarcation line does form, then immediately read and record the height of the column of sediment and the total sedimentation time. If this should occur in a test being made with tap water, discontinue the test and retest using an untested portion of the sample with distilled or demineralized water. The second unusual condition is that the liquid immediately above the line of demarcation may still be darkly clouded at the end of 20 minutes, and the demarcation line, although distinct, may appear to be in the sediment column itself. As for the first case, rerun the test using a new sample with distilled or demineralized water if tap water was used; otherwise read and record this line of demarcation at the end of the specified 20-minute sedimentation period as usual.

2. Test Procedure for Fine Aggregate

a. Siphon 4 ± 0.1 in. of working calcium chloride solution into the plastic cylinder.

b. Pour the prepared test sample into the plastic cylinder using the funnel to avoid spillage (See Fig. 2.4g). Tap the bottom of the cylinder sharply on the heel of the hand several times to release air bubbles and to promote thorough wetting of the sample.

c. Allow the wetted sample to stand undisturbed for 10 ± 1 minutes.

d. At the end of the 10-minute soaking period, stopper the cylinder, then loosen the material from the bottom by partially inverting the cylinder and shaking it simultaneously.

e. Place the stoppered cylinder in the mechanical sand equivalent shaker and allow the machine to continuously shake the cylinder and contents for 10 minutes ± 15 seconds.

f. Following the shaking operation, set the cylinder upright on the work table and remove the stopper.

g. Insert the irrigator tube in the cylinder and rinse material from the cylinder walls as the irrigator is lowered. Force the irrigator through the material to the bottom of the cylinder by applying a gentle stabbing and twisting action while the working solution flows from the irrigator tip. This flushes the fine material into the suspension above the coarser sand particles. (See Fig. 2.4h).

h. Continue to apply a stabbing and twisting action while flushing the fines upward until the cylinder is filled to the 15-inch mark. Then raise the irrigator slowly without shutting off the flow so that the liquid level is maintained at about 15 in. while the irrigator is being withdrawn. Regulate the flow just before the irrigator is entirely withdrawn and adjust the final level to 15 in.

i. Allow the cylinder and contents to stand undisturbed for 20 minutes ± 15 seconds. Start the timing immediately after withdrawing the irrigator tube.

j. At the end of the 20-minute sedimentation period, read and record the level of the top of the clay suspension. This is referred to as the "clay reading". If no clear line of demarcation has formed at the end of the specified 20-minute sedimentation period, allow the sample to stand undisturbed until a clay reading can be obtained, then immediately read and record the level of the top of the clay suspension and the total sedimentation time. If the total sedimentation time exceeds 30 minutes, rerun the test using three individual samples of the same material. Read and record the clay column height of that sample requiring the shortest sedimentation period only.

k. After the clay reading has been taken, place the weighted foot assembly over the cylinder with the guide in position on the mouth of the cylinder and gently lower the weighted foot until it comes to rest on the sand.

l. While the weighted foot is being lowered, keep one of the centering screws in contact with the cylinder wall near the graduations so that it can be seen at all times.

m. When the weighted foot has come to rest on the sand, read and record the level of the centering screw. This reading is referred to as the "sand reading". (See Fig. 2.4i).

n. If clay or sand readings fall between. 0.1-inch graduations, record the level of the higher graduation as the reading. For example, a clay level at 7.95 would be recorded as 8.0. A sand level at 3.22 would be recorded at 3.3.

## G. CALCULATIONS AND REPORTING

1. Durability Factor of Coarse Aggregate

a. Compute the durability factor of the coarse aggregate to the nearest whole number by the following formula:

$$D_c = 30.3 + 20.8 \cot (0.29 + 0.15 H)$$

Where:

$D_c$ = Durability Factor

H = Height of Sediment in inches

Solutions of the above equation are given in Table No. 2.2.

2. Durability Factor of Fine Aggregate

a. Calculate the durability factor of the fine aggregate to the nearest 0.1 using the following formula:

$$D_f = \frac{\text{Sand reading}}{\text{Clay reading}} \text{ X } 100$$

b. If the calculated durability factor is not a whole number, report it as the next higher whole number. For example, if the durability factor were calculated from the example in paragraph 2n of Article F the calculated durability factor would be:

$$D_f = \frac{3.3}{8.0} \text{ x } 100 = 41.2$$

c. Since this calculated durability factor is not a whole number it would be reported as the next higher whole number which is 42.

d. If it is desired to average a series of values, average the whole number values determined as described above. If the average of these values is not a whole number, raise it to the next higher whole number as shown in the following example:

(1) Calculated $D_f$ values: 41.2, 43.8, 40.9.

(2) After raising each to the next higher whole number they become: 42, 44, 41.

(3) The average of these values is then determined.

$$\frac{42 + 44 + 41}{3} = 42.3$$

e. Since the average value is not a whole number it is raised to the next higher whole number and the reported average durability factor is reported as "43".

FIGURE 2.4g – TEST PROCEDURE FOR FINE AGGREGATE

FIGURE 2.4h – TEST PROCEDURE FOR FINE AGGREGATE

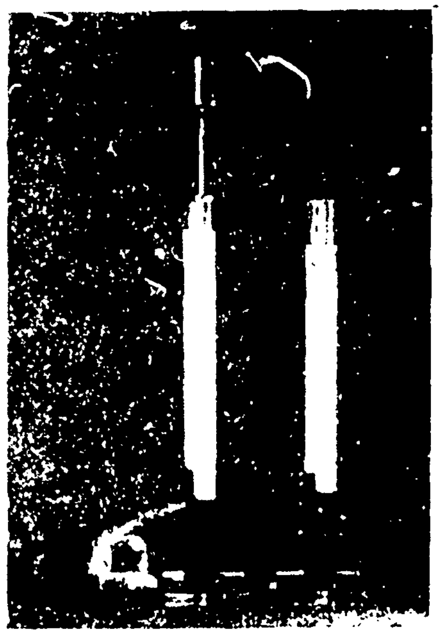


FIGURE 2.4i – TEST PROCEDURE FOR FINE AGGREGATE

## TABLE 2.2

**Durability Factor of Coarse Aggregate**
$D_c = 30.3 + 20.8 \cot (0.29 + 0.15H)$

| Sediment height (inches) | $D_c$ | Sediment height (inches) | $D_c$ | Sediment height (inches) | $D_c$ | Sediment height (inches) | $D_c$ | Sediment height (inches) | $D_c$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 100 | 3.0 | 53 | 6.0 | 39 | 9.0 | 29 | 12.0 | 18 |
| 0.1 | 96 | 3.1 | 52 | 6.1 | 38 | 9.1 | 29 | 12.1 | 18 |
| 0.2 | 93 | 3.2 | 52 | 6.2 | 38 | 9.2 | 28 | 12.2 | 18 |
| 0.3 | 90 | 3.3 | 51 | 6.3 | 38 | 9.3 | 28 | 12.3 | 17 |
| 0.4 | 87 | 3.4 | 51 | 6.4 | 37 | 9.4 | 28 | 12.4 | 17 |
| 0.5 | 85 | 3.5 | 50 | 6.5 | 37 | 9.5 | 27 | 12.5 | 16 |
| 0.6 | 82 | 3.6 | 49 | 6.6 | 37 | 9.6 | 27 | 12.6 | 16 |
| 0.7 | 80 | 3.7 | 49 | 6.7 | 36 | 9.7 | 27 | 12.7 | 15 |
| 0.8 | 78 | 3.8 | 48 | 6.8 | 36 | 9.8 | 26 | 12.8 | 15 |
| 0.9 | 76 | 3.9 | 48 | 6.9 | 36 | 9.9 | 26 | 12.9 | 14 |
| 1.0 | 74 | 4.0 | 47 | 7.0 | 35 | 10.0 | 26 | 13.0 | 14 |
| 1.1 | 73 | 4.1 | 47 | 7.1 | 35 | 10.1 | 25 | 13.1 | 13 |
| 1.2 | 71 | 4.2 | 46 | 7.2 | 35 | 10.2 | 25 | 13.2 | 13 |
| 1.3 | 70 | 4.3 | 46 | 7.3 | 34 | 10.3 | 25 | 13.3 | 12 |
| 1.4 | 68 | 4.4 | 45 | 7.4 | 34 | 10.4 | 24 | 13.4 | 12 |
| 1.5 | 67 | 4.5 | 45 | 7.5 | 34 | 10.5 | 24 | 13.5 | 11 |
| 1.6 | 66 | 4.6 | 44 | 7.6 | 33 | 10.6 | 24 | 13.6 | 11 |
| 1.7 | 65 | 4.7 | 44 | 7.7 | 33 | 10.7 | 23 | 13.7 | 10 |
| 1.8 | 63 | 4.8 | 43 | 7.8 | 33 | 10.8 | 23 | 13.8 | 9 |
| 1.9 | 62 | 4.9 | 43 | 7.9 | 32 | 10.9 | 23 | 13.9 | 9 |
| 2.0 | 61 | 5.0 | 43 | 8.0 | 32 | 11.0 | 22 | 14.0 | 8 |
| 2.1 | 60 | 5.1 | 42 | 8.1 | 32 | 11.1 | 22 | 14.1 | 7 |
| 2.2 | 59 | 5.2 | 42 | 8.2 | 31 | 11.2 | 22 | 14.2 | 7 |
| 2.3 | 59 | 5.3 | 41 | 8.3 | 31 | 11.3 | 21 | 14.3 | 6 |
| 2.4 | 58 | 5.4 | 41 | 8.4 | 31 | 11.4 | 21 | 14.4 | 5 |
| 2.5 | 57 | 5.5 | 40 | 8.5 | 30 | 11.5 | 20 | 14.5 | 4 |
| 2.6 | 56 | 5.6 | 40 | 8.6 | 30 | 11.6 | 20 | 14.6 | 4 |
| 2.7 | 55 | 5.7 | 40 | 8.7 | 30 | 11.7 | 20 | 14.7 | 3 |
| 2.8 | 54 | 5.8 | 39 | 8.8 | 29 | 11.8 | 19 | 14.8 | 2 |
| 2.9 | 54 | 5.9 | 39 | 8.9 | 29 | 11.9 | 19 | 14.9 | 1 |
| | | | | | | | | 15.0 | 0 |

### H. PRECAUTIONS

1. Perform the test in a location free of vibrations, because vibrations may cause the suspended material to settle at a greater rate than normal.
2. Do not expose the plastic cylinders to direct sunlight any more than is necessary.
3. Frequently check the play between the cam and eccentric on the modified Tyler portable shaker by grasping one of the hanger rods and attempt to move the sieve base. If any play is noticed, replace the cam and/or bearing.
4. Lubricate the sieve shaker at least each three months.

### REFERENCES


A California Test Method
Test Method No. Calif. 201
Test Method No. Calif. 202

End of Text on Calif. 229-C

12) *USING THE FHA SOIL PVC METER*

(Reproduced from "Guide to Use of the FHA Soil PVC Meter, by G.F. Henry and M.C. Dragoo, Federal Housing Administration, FHA No. 595, Jan. 1965, Washington).

## A. General

The FHA Soil PVC Meter (Figure 2.5a) is used to perform a swell index test. This test is essentially a measurement of the pressure exerted by a sample of compacted soil when it swells against a restraining force after being wetted. The FHA Soil PVC Meter, in addition to yielding PVC values, can be used to estimate the plasticity index and shrinkage behavior of soils. These values are determined by comparing the results of the swell index test with appropriate values contained in Figures 2.5b, 2.5c, 2.5d and 2.5e in this guide and reading the corresponding extrapolations.

The following categories of PVC have been established:

| PVC Rating | Category |
|---|---|
| Less than 2 | Noncritical |
| 2 to 4 | Marginal |
| 4 to 6 | Critical |
| Greater than 6 | Very critical |

These ratings were established on the basis of the swelling and shrinking behavior of the soil.

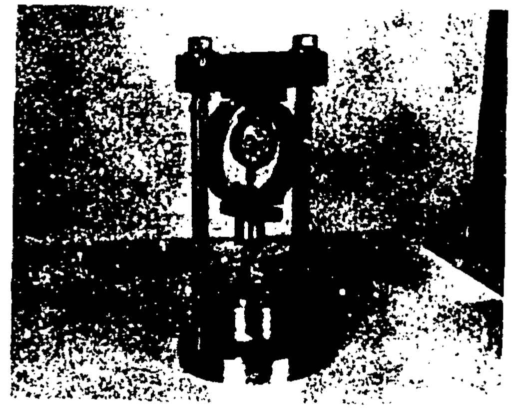


FIGURE 2.5a – PICTURES OF EQUIPMENT

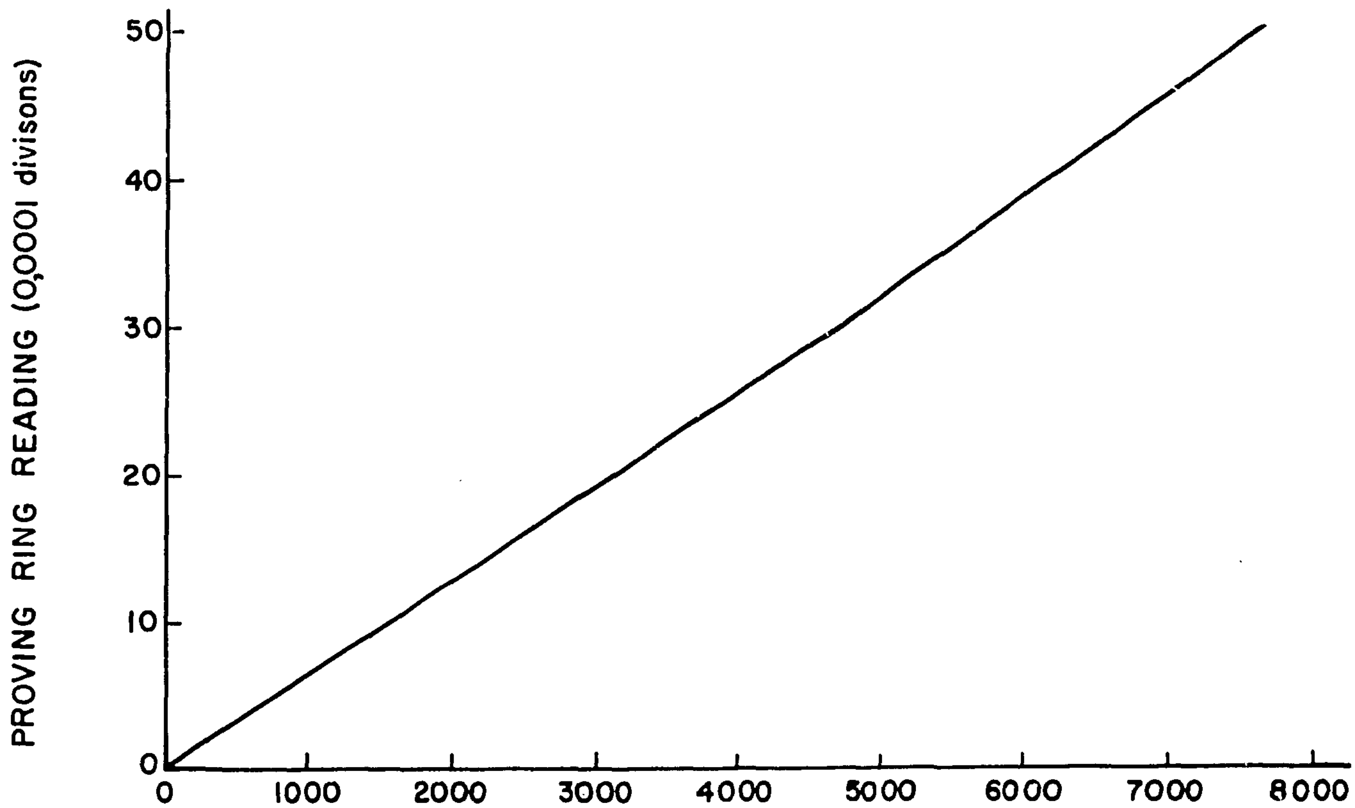



FIGURE 2.5b – PROVING RING CALIBRATION FROM "FHA SOIL PVC METER PUBLICATION" FEDERAL HOUSING ADMINISTRATION PUBLICATION N° 701 (Lambe 1960)

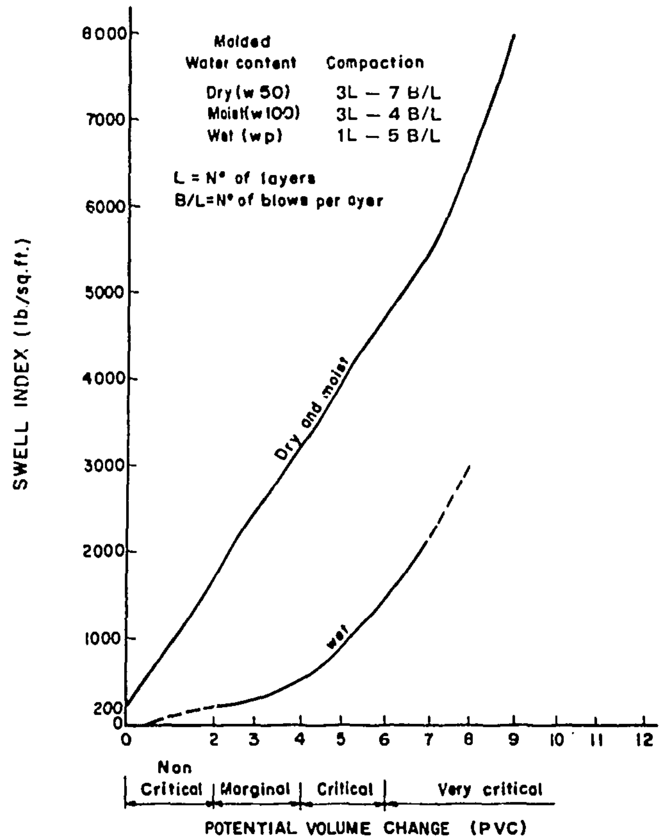



FIGURE 2.5c – SWELL INDEX VS. POTENTIAL VOLUME CHANGE FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

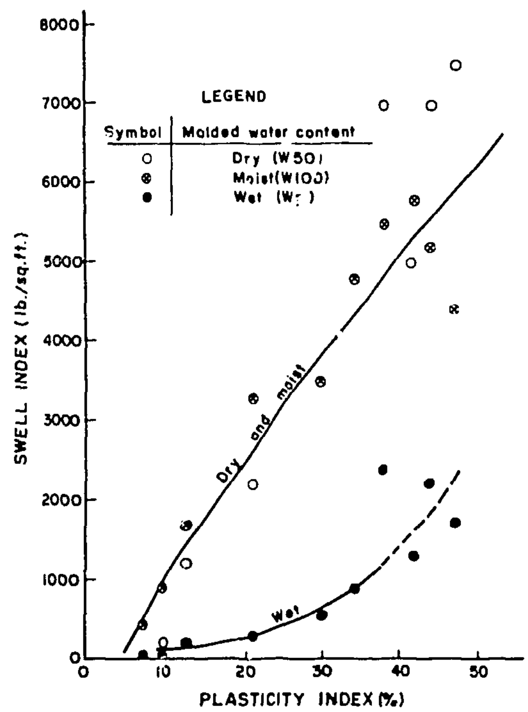



FIGURE 2.5d – SWELL INDEX VS. PLASTICITY FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

## B. Equipment

1. PVC Meter
2. Spacers, plate, and clamp for alternate compaction method.
3. No. 10 Sieve
4. Teaspoon
5. Compaction Hammer and Sleeve
6. Two Dry Porous Stones
7. Knife, (preferably serated)
8. Straight Edge
9. Water in Squirt Bottle with Pointed End
10. Wrenches

## C. Preparation of Sample

For the test sample, take about a pint of soil from the soil layer in which the foundation member will rest. Although samples can be tested at three relative water contents (dry, moist, or wet), it is suggested that those being tested for FHA purposes be tested in the air dried condition only. The samples can be sufficiently air dried by breaking the soil into small lumps and leaving it in the sun for a few hours. The following procedures are for soil in the air dried condition. For information about soil in other conditions, see Lambe (1960).

## D. Preparation for Compaction

Disassemble the PVC Meter with exception of the rods which can remain screwed into the base. Place proving ring and top bar where it will not be jarred during compaction. Wipe equipment with clean cloth.

## E. Compaction

**DEFINITIONS:**

Compaction ring largest ring; identified by letter "c" etched on outside periphery.

Spacer ring smallest ring; identified by letter "s" etched on outside periphery.

1. To assemble meter for compaction, fit compaction ring on base so that "c" is backwards and at the top. Align bolt holes with those inbase. Place spacer ring on compaction ring so that "s" is at the top (radial grooves are at top). Align bolt holes with those in base. Insert the 3 bolts through both rings and the base and tighten firmly to base.

2. The soil sample is to be placed in the ring assembly in 3 layers of equal amounts. Each layer is to be compacted separately. Compaction is accomplished by use of the hammer, which is a tamping device encased in a metal sleeve.

3. Compact each layer of the sample in the following manner:

a. Place 3 heaping teaspoonsfull of sample in ring assembly and smooth lightly with hammer to firm up the surface before applying the blows (This reduces the amount of soil "jumping" out of the mold during compaction). Place apparatus on a solid level floor.

b. Before each blow, lift sleeve 1/8 inch from soil and hold firmly against the inside of the spacer ring. *Make sure sleeve of hammer rests inside rings so that hammer does not damage them in falling.* Be sure to hold sleeve and hammer perpendicular and in line with supporting rods. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer. (See F for Alternate Compaction Method).

4. At completion of the compaction of both the first and second layers scratch the top surface of the layer with a knife to assure proper bond with the next layer. After compaction, the last layer should extend approximately 1/4 inch into the spacer ring. If it is significantly below this point, remove entire sample and recompact.

5. Put assembly on table and remove the 3 bolts. Rotate spacer ring (to break bond between ring and soil) and remove carefully from base. Remove compaction ring containing sample in same way. Do not tilt compaction ring or spill soil.

6. Trim top of the sample with a knife. Hold knife against the compaction ring at all times during trimming to avoid dislodging sample. Trim in a sawing motion taking off only a small amount of soil at a time. Rotate the ring as you trim. Work from the edge toward the center. When sample is almost level, do final leveling by drawing a metal straight edge over sample.

7. The final surface of the soil sample should be firm and smooth. Any voids should be filled by pressing additional soil into them with the knife or spoon.

8. Clean soil from base and from all holes in rings and base. Remove soil in the groove of the spacer ring and from the holes in the spacer ring and the compaction ring with a toothpick or paperclip.

## F. Alternate Compaction Method

1. After fitting rings to base as explained in E, paragraph 1, place one spacer on each rod, then set the plate on the spacers. Bolt these securely to the rods. Attach the clamp to the sleeve so that the sleeve extends about 1/4 inch insider the spacer ring. Place the soil sample in the ring assembly in the same manner as explained in E, paragraphs 1 and 3a.

2. Before each blow, turn the "foot" of the clamp so that it points in the direction of the spot to be compacted. The sleeve and hammer must be held perpendicular and in line with the supporting rods. To assure this, the sleeve should be held firmly against the inside of the plate and the spacer ring. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer.

3. The remaining compaction process is the same as E, paragraphs 4 through 8.

## G. Swelling

1. Place spacer ring on base with "s" (and radial grooves) on top. Align bolt holes with those on base. Place *thoroughly dry* porous stone in spacer ring. Move assembled base to edge of working table. Place thumb under base and other fingers over spacer ring and stone, holding them firmly in place. Turn base upside-down retaining firm hold on stone and spacer ring. Pick up compaction ring containing sample trimmed side up and place flush against porous stone in spacer ring aligning bolt holes in the two rings. Move compaction ring with as little disturbance of sample as possible. Turn base with rings, stone, and sample rightside-up. Bolt rings tightly to base.

2. Place a dry porous stone on top of sample inside compaction ring. Place the rubber O-ring on the base and screw the lucite container onto it tightly to insure water seal. Place metal cover on porous stone with the center indentation at the top.

3. Place top bar with proving ring on the steel rods (Be sure that the adjustable rod which extends down from the proving ring dial does not strike the cover). Add washers and nuts and tighten firmly.

4. Set proving ring dial to zero by moving the band around the dial. Tighten dial with the screw on band. Push up on proving ring dial to see that it appears to work properly. Turn adjustable rod exactly into the center of the indentation on top of the cover. Be sure that the cover is centered exactly over the stone. Tighten lock nut on adjustable rod firmly. Be sure adjustable rod does not stick in cover (receptable for adjustable rod may require slight enlargement). Turn adjustable rod until dial reads one division past zero. Tighten lock nut firmly again until adjustable rod has no play.

5. Record the time and the proving ring reading. Add water to sample by squeezing from squirt bottle into the holes located at the top of compaction ring until water level in lucite container has covered the spacer ring and tops of the bolts. (This procedure is used to reduce the amount of air entrapped in the ring assembly and thus insures that the sample has uniform access to water over its entire top and bottom surfaces).

## H. Reading

1. Allow soil to expand until completely stabilized or for a maximum of 2 hours, then read dial to obtain PVC swell index value. On the dial the number 1 equals 10 divisions, the number 2 equals 20, etc.

2. Next, find the number corresponding to the proving ring dial reading on Figure 2.5b and subtract the one division that registered on the dial prior to swell. Read horizontally to intersection with sloping line. From point of intersection, read downward to baseline which indicates pressure in lbs./sq. ft.

3. Take this figure to Figure 2.5c find the number corresponding to it on left hand side of the chart. Read horizontally to intersection with the sloping line marked "Dry and Moist". From point of intersection, red downward to the baseline, which indicates PVC category.

4. Take the reading in lbs./sq.ft. to Figure 2.5d to determine the plasticity index.

5. It is also possible to obtain the approximate PVC category and plasticity index by taking the reading from the proving ring dial directly to Table 2.3.

**TABLE 2.3**
**Table for Converting Proving Ring Readings to PVC Category and Approximate Plasticity Index**

| PROVING RING READING | SWELL INDEX (#/SF) | PVC CATEGORY | PLASTICITY INDEX (%) |
|---|---|---|---|
| 5 | 775 | 0.8 | 8.5 |
| 6 | 925 | 1.0 | 9.5 |
| 7 | 1 075 | 1.2 | 10.7 |
| 8 | 1 250 | 1.4 | 11.7 |
| 9 | 1 375 | 1.6 | 12.7 |
| 10 | 1 550 | 1.8 | 13.8 |
| 10.8 | 1 675 | 2.0 | 14.6 |
| 11 | 1 700 | 2.0 | 14.8 |
| 12 | 1 875 | 2.2 | 15.8 |
| 13 | 2 025 | 2.4 | 17.0 |
| 14 | 2 175 | 2.65 | 18.0 |
| 15 | 2 350 | 2.85 | 19.0 |
| 16 | 2 500 | 3.05 | 20.0 |
| 17 | 2 675 | 3.3 | 21.5 |
| 18 | 2 800 | 3.45 | 22.5 |
| 19 | 2 975 | 3.7 | 23.8 |
| 20 | 3 150 | 3.9 | 25.0 |
| 20.3 | 3 200 | 4.0 | 25.5 |
| 21 | 3 300 | 4.1 | 26.0 |
| 22 | 3 450 | 4.3 | 27.5 |
| 23 | 3 600 | 4,5 | 28.5 |
| 24 | 3 775 | 4.75 | 29.8 |
| 25 | 3 925 | 4.95 | 30.8 |
| 26 | 4 075 | 5.15 | 31.8 |
| 27 | 4 225 | 5.4 | 33.0 |
| 28 | 4 375 | 5.55 | 34.0 |
| 29 | 4 525 | 5.75 | 35.3 |
| 30 | 4 700 | 5.95 | 37.0 |
| 30.2 | 4 725 | 6.00 | 37.1 |
| 31 | 4 850 | 6.2 | 38.0 |
| 32 | 4 975 | 6.35 | 39.0 |
| 33 | 5 125 | 6.5 | 40.4 |
| 34 | 5 275 | 6.7 | 41.7 |
| 35 | 5 425 | 6.9 | 43.4 |
| 36 | 5 575 | 7.1 | 44.2 |
| 37 | 5 725 | 7.25 | 45.5 |
| 38 | 5 850 | 7.4 | 46.6 |
| 39 | 6 000 | 7.5 | 48.0 |
| 40 | 6 150 | 7.65 | 49.5 |
| 40.5 | 6 225 | 7.7 | 50.0 |

Prepared by the Architectural Section, Federal Housing Administration Insuring Office San Antonio, Texas.

13) *SUGGESTED METHOD OF TEST OF ONE-DIMENSIONAL EXPANSION AND UPLIFT PRESSURE OF CLAY SOILS*

(Reproduced from "Special Procedures for Testing Soil and Rock for Engineering Purposes", 5th ed., ASTM Special Technical Publication 479, June 1970).

## 1. Scope

1.1 This method explains how to make expansion tests on undisturbed or compacted clay soil samples that have no particle sizes greater than $\frac{3}{16}$ in. (passing the No. 4 standard ASTM sieve). The test is made to determine (1) magnitude of volume change under load or no-load conditions, (2) rate of volume change, (3) influence of wetting on volume change, and (4) axial permeability of laterally confined soil under axial load or no-load during expansion. Saturation (no drainage) takes place axially. Permeant water is applied axially for determining the effect of saturation and permeability. The specimens prepared for this test may also be used to determine the vertical or volume shrinkage as the water content decreases. Total volume change for expansive soils is determined from expansion plus shrinkage values for different ranges of water content.

1.2 Expansion test data may be used to estimate the extent and rate of uplift in subgrades beneath structures or in structures formed from soils, and shrinkage tests may be used to estimate the volume changes which will occur in soils upon drying, provided that natural conditions and operating conditions are duplicated.

## 2. Significance

2.1 The expansion characteristics of a soil mass are influenced by a number of factors. Some of these are size and shape of the soil particles, water content, density, applied loadings, load history and mineralogical and chemical properties. Because of the difficulty in evaluating these individual factors, the volume-change properties cannot be predicted to any degree of accuracy unless laboratory tests are performed. When uplift problems are critical, it is important to test samples from the sites being considered.

2.2. The laboratory tests described herein are primarily intended for the study of soils having no particles larger than the No. 4 standard sieve size ($\frac{3}{16}$ in.). If the test is made on the minus No. 4 fraction of soils containing gravel material (plus No. 4), some adjustment is required in any analysis. Gravel reduces volume change because it replaces the more active soil fraction.

## 3. Apparatus

3.1 *Consolidometer* – Conventional laboratory consolidometers are used for the expansion test. Consolidometers most used in the United States are of the fixed-ring and floating-ring types. Figure 2.6 illustrates the fixed-ring type. Either of these is suitable. Both types are available commercially. In the fixed-ring container, all specimen movement relative to the container is upward during expansion. In the floating-ring container, movement of the soil sample is from the top and bottom away from the center during expansion. The specimen containers for the fixed-ring consolidometer and the floating-ring consolidometer consist of brass or plastic rings, and other component parts. Sizes of container rings most commonly used vary between 4 ¼ -in. diameter by 1 ¼ -in. deep and 2 ½ -in. diameter by 3/4-in. deep, although other sizes are used. However, the diameter should be not less than 2 in. and the depth not greater than three tenths of the diameter, except

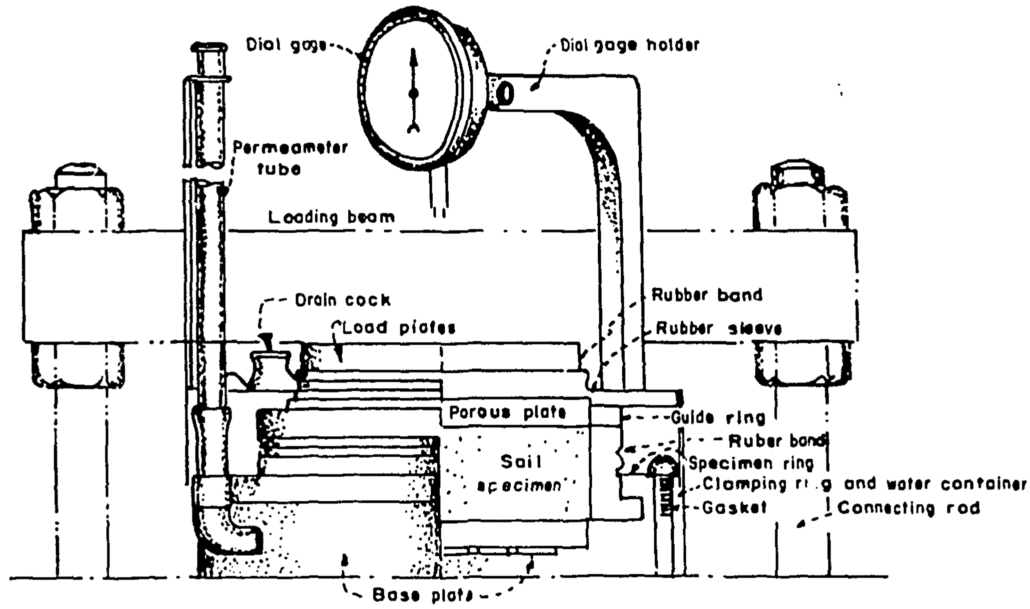



FIGURE 2.6 – FIXED-RING CONSOLIDOMETER

that the depth must not be less than 3/4 in. for specimens of small diameter. Lesser depths introduce errors caused by the magnitude of surface disturbance, while large depths cause excessive side friction. For expansion tests the larger diameter consolidation rings are preferred as they restrain the soil action to a lesser degree. In a test using the floating-ring apparatus, the friction between the soil specimen and container is smaller than with the fixed-ring apparatus. On the other hand, the fixed-ring apparatus is more suitable for saturation purposes and when permeability data are required. Porous stones are required at the top and bottom of the specimen to allow application of water. The apparatus must allow vertical movement of the top porous stone for fixed-ring consolidometers, or vertical movement for top and bottom porous stones for floating-ring consolidometers, as expansion takes place. A ring gage machined to the height of the ring container to an accuracy of 0.001 in. is required; thus, the ring gage for $1\frac{1}{4}$-in. high specimens will have a height of 1.250 in. Measure the diameter of the specimen container ring to 0.001 in.

3.2 Loading Device – A suitable device for applying vertical load to the specimen is required. The loading device may be platform scales of 1000 to 3000 lb capacity mounted on a stand and equipped with a screw jack attached underneath the frame. The jack operates a yoke which extends up through the scale platform and over the specimen container resting on the platform. The yoke is forced up or down by operating the jack, thus applying or releasing load to the soil specimen. The desired applied pressure, which is measured on the scale beam, becomes fully effective when the beam is balanced.

3.2.1 Another satisfactory loading device utilizes weights and a system of levers for handling several tests simultaneously. Hydraulic-piston or bellows-type loading apparatus are also very satisfactory if they have adequate capacity, accuracy, and sensitivity for the work being performed. Apparatus such as described in ASTM Method D 2435, Test for One-Dimensional Consolidation Properties of Soils[4] is satisfactory and may be used.

3.3 Device for Cutting Undisturbed Specimens – This apparatus consists of a cutting bit of the same diameter as the ring container of the consolidometer, a cutting stand with bit guide, and knives for trimming the soil. Wire saws or trimming lathes may be used if a uniform tight fit of the specimen to the container is obtained.

3.4 *Device for Preparation of Remolded Specimens* – Compacted soil specimens are prepared in the consolidometer ring container. In addition to the container, the apparatus consists of an extension collar about 4 in. in depth and of the same diameter as the container. A compaction hammer of the same type required in Method A of ASTM Method D 698, Test for Moisture-Density Relations of Soils, Using 5.5 lb Rammer and 12-in. Drop.[4]

## 4. Procedure-Expansion Test

4.1 *Preparation of Undisturbed Specimens* – Perform the tests on hand-cut cube samples or core samples of a size that will allow the cutting of approximately 1/2-in. of material from the sides of the consolidometer specimen. (Alternatively, obtain a core of a diameter exactly the same as the diameter of the consolidometer specimen container and extrude the core directly into the container. This procedure is satisfactory provided that the sampling has been done without any sidewall disturbance and provided that the core specimen exactly fits the container. Place the undistrubed soil block or core on the cutting platform, fasten the cutting bit to the ring container and place the assembly on the sample in alignment with the guide arms. With the cutting stand guiding the bit, trim the

4 Annual Book of ASTM Standards, Part 11.

excess material with a knife close to the cutting edge of the bit, leaving very little material for the bit to shave off as it is pressed gently downward. (Other suitable procedures to accommodate guides for wire saws, trimming lathes, or extrusion devices may be used in conformance with the use of alternative apparatus and samples). In trimming the sample, be careful to minimize disturbance of the soil specimen and to assure an exact fit of the specimen to the consolidometer container. When sufficient specimen has been prepared so that it protrudes through the container ring, trim it flush with the surface of the container ring with a straightedge cutting tool. Place a glass plate on the smooth, flat cut surface of the specimen, and turn the container over. Remove the cutting bit, trim the specimen flush with the surface of the container ring, and cover it with a second glass plate to control evaporation until it is placed in the loading device.

4.2 *Preparation of Remolded Specimens* – Use about 2 lb of representative soil that has been properly moistened to the degree desired and processed free from lumps and from which particles or aggregations of particles retained by a $\frac{3}{16}$ -in. (No. 4) sieve have been excluded. Compact the specimen to the required wet bulk density after adding the required amount of water as follows: Place the extension collar on top of the container ring and fasten the bottom of the container ring to a baseplate. Weigh the exact quantity of the processed sample to give the desired wet density when compacted to a thickness 1/4 in. greater than the thickness of the container ring. Compact the specimen to the desired thickness by the compaction hammer. Remove the extension collar and trim the excess material flush with the container ring surface with a straightedge cutting tool. Remove the ring and specimen from the baseplate and cover the specimen surfaces with glass plates until the specimen is placed in the loading device. If, after weighing and measuring the specimen and computing the wet density, as described below, the wet density is not within 1.0 lb/ft$^3$ of that required, repeat the preparation of the remolded specimen until the required accuracy is obtained.

4.3 *Calibration of Dial Gage for Height Measurements* – Prior to filling the container ring with the soil specimen, place a ring gage in the specimen container with the same arrangement of porous plates and load plates to be used when testing the soil specimen. Place the assembly in the loading machine in the same position it will occupy during the test. After the apparatus has been assembled with the ring gage in place, apply a load equivalent to a pressure of 0.35 psi (or 0.025 kgf/cm$^2$) on the soil specimen. The dial reading at this time will be that for the exact height of the ring gage. Mark the parts of the apparatus so that they can be matched in the same position for the test.

4.4 *Initial Height and Weight of Soil Specimen* – Clean and weigh the specimen container ring and glass plates and weigh them to ± 0.01 g before the ring is filled. After filling and trimming is completed, weigh the soil specimen, ring, and glass plates to ± 0.01 g. Determine the weight of the soil specimen. Assemble the specimen container and place it in the loading device. If the specimen is not to be saturated at the beginning of the test, place a rubber sleeve around the protruding porous plates and load plates to prevent evaporation. Apply the small seating load of 0.35 psi (or 0.025 kgf/cm$^2$) to the specimen. By comparing the dial reading at this time with the dial reading obtained with the ring gage in place, determine the exact height of the specimen. Use this information to compute the initial volume of the specimen, the initial density, void ratio, water content, and degree of saturation. The true water content of the specimen will be determined when the total dry weight of the specimen is obtained at the end of the test.

4.5 *Saturation and Permeability Data* – To saturate the specimen attach the percolation tube standpipe, fill it with water, and wet the specimen. Take care to remove any air that may be entrapped in the system by slowly wetting the lower porous stone and draining the stone through the lower drain cock. After the specimen is wetted, fill the pan in which the consolidometer stands with water. After saturation has been completed, permeability readings can be taken at any time during the test by filling the percolation tube standpipe to an initial reading and allow the water to percolate through the specimen. Measure the amount of water flowing through the sample in a given time by the drop in head.

4.6 *Expansion Test:*

4.6.1 *General Comments* – The expansion characteristics of an expansive-type soil vary with the loading history, so that it is necessary to perform a separate test or several specimens for each condition of loading at which *exact* expansion data are required. However, one procedure is to test only two specimens: (1) loaded-and-expanded, and (2) expanded-and-loaded. From these data, an estimate of expansion can be made for any load condition as shown by Curve C, Figure 2.7, in which Specimen No. 1 was loaded and expanded by saturation with water, (Curve B) and Specimen No. 2 was expanded by saturation with water and then loaded (Curve A).

4.6.2 *Loaded and Expanded Test* – To measure expansion characteristics where the soil specimen is saturated under full load and then allowed to expand, apply the seating load of 0.35 psi (or 0.025 kgf/cm$^2$) to Specimen No. 1, and secure initial dial readings. Then saturate the soil specimen as described in 4.5. (The permeameter tube head should be sufficiently low so that the specimen is not lifted). As the specimen begins to expand, increase the load as required to hold the specimen at its original height. Then reduce the load to 1/2, 1/4, and 1/8 of the maximum load and finally to the seating load of 0.35 psi (or

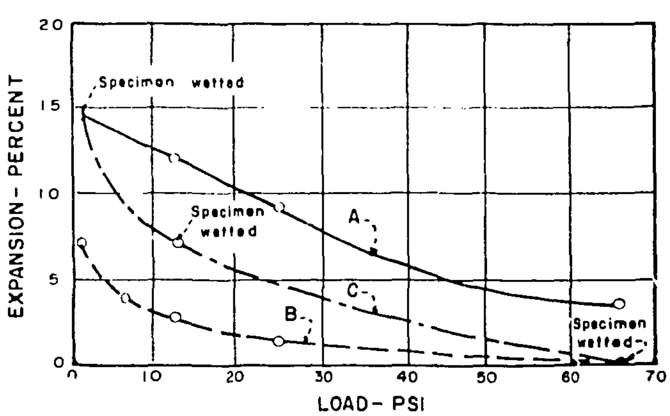



FIGURE 2.7 – LOAD-EXPANSION CURVES

0.025 kgf/cm$^2$) and measure the height with each load. Use a greater number of loadings if greater detail in the test curve is required. Maintain all loads for 24 h, or longer if needed, to obtain constant values of height. Remove the specimen from the ring container and weigh it immediately and again after drying to 105 C. From the water content, dry bulk density, and specific gravity of the specimen, calculate the volume of air and, assuming it to be the same as the volume of air following the determination of permeability, calculate the water content and degree of saturation.

4.6.3 *Expanded and Loaded Test* – To measure expansion characteristics where the soil is allowed to expand before loading, apply the seating load of 0.35 psi (or 0.025 kgf/cm$^2$) to Specimen No. 2, and secure initial dial gage readings. Then saturate the specimen as described in 4.5. Allow the specimen to expand under the seating load for 48 h or until expansion is complete. Load the specimen successively to 1/8, 1/4, 1/2 and 1 times the maximum load found in 4.6.2, to determine the reconsolidation characteristics of the soil. Use a greater number of loadings, if greater detail in the test curve is required. Follow the procedures specified in 4.6.2 for making loadings and all measurements and determinations.

4.6.4 *Individual Load-Expansion Test* – When it is desired to perform separate expansion tests for other conditions of loading aplly the seating load of 0.35 psi (or 0.025 kgf/cm$^2$) to the specimen and measure the initial height. Then load the specimen to the desired loading, saturate the specimen as described in 4.5, and allow the specimen to expand under the applied load for 48 h, or until expansion is complete. Measure the height of the expanded specimen. Reduce the load to that of the seating load. Allow the height to become constant and measure; then remove the specimen from the ring and make the determination specified in 4.6.2.

## 5. Procedure – Shrinkage Test

5.1 *Specimen Preparation* – When measurements of shrinkage on drying are needed, prepare an additional specimen as described in 4.1 or 4.2. Cut this specimen from the same undisturbed soil sample as the expansion specimens, or remolded to the same bulk density and water content conditions as the expansion specimens. Place the specimen in the container ring, and measure the initial volume and height as described in 4.4. Determine the water content of the soil specimen by weighing unused portions of the original sample of which the specimen is a part, drying the material in an oven to 105 C, and reweighing it.

5.2 *Volume and Height Shrinkage Determinations* – To measure volume shrinkage, allow the specimen in the ring to dry in air completely or at least to the water content corresponding to the shrinkage limit (ASTM Method D 427, Test for Shrinkage Factors of Soils). After the specimen has been air-dried, remove it from the ring container, and obtain its volume by the mercury-displacement method.

5.2.1 To perform the mercury displacement measurement, place a glass cup with a smoothly ground top in an evaporating dish. Fill the cup to overflowing with mercury, and then remove the excess mercury by sliding a special glass plate with three prongs for holding the specimen in the mercury over the rim. Pour the excess mercury into the original container and replace the glass cup in the evaporating dish. Then immerse the air-dried soil specimen in the glass cup filled with mercury using the special glass plate over the glass cup to duplicate the initial mercury volume determination condition. (See Method D 427 for general scheme of test and equipment.) Transfer the displaced mercury into a graduated cylinder, and measure the volume. If the shrinkage specimen is cracked into separate parts, measure the volume of each part, and add the individual volumes to obtain the total. (A paper strip wrapped around the specimen side and held by a rubber band is effective in holding the specimen intact during handling).

5.2.2 If the height of the air-dried specimen is desired, place the specimen and ring container in the loading machine. Apply the seating load of 0.35 psi (or 0.025 kgf/cm$^2$), and then read the dial gage.

## 6. Calculations

6.1 *Expansion Test Data* – Calculate the void ratio as follows:

$$e = \frac{\text{volume of voids}}{\text{volume of solids}} = \frac{h - h_o}{h_o}$$

where:

$e$ = void ratio,
$h$ = height of the specimen, and
$h_o$ = height of the solid material at zero void content

Calculate the expansion, as a percentage of the original height, as follows:

$$\Delta e \text{ percent} = \frac{h_2 - h_1}{h_1} \times 100$$

where:

$\Delta e$ = expansion in percentage of initial volume,
$h_1$ = initial height of the specimen, and
$h_2$ = height of the specimen under a specific load condition.

6.2 *Permeability Test Data* – Calculate the permeability rate by means of the following basic formula for the variable head permeameter:

$$k = \frac{A_p \times L_s}{A_s \times 12} \times \frac{1}{t} \ln \frac{H_i}{H_f}$$

where:

$k$ = permeability rate, ft/year,
$A_p$ = area of standpipe furnishing the percolation head, in.$^2$,
$A_s$ = area of the specimen, in.$^2$,
$L_s$ = length of the specimen, in.,
$H_i$ = initial head, difference in head between headwater and tailwater, in.,
$H_f$ = final head, difference in head between headwater and tailwater, in., and
$t$ = elasped time, years.

6.3 *Shrinkage Test Data* – Calculate the volume shrinkage as a percentage of the initial volume as follows:

$$\Delta_s = \frac{v_i - v_d}{v_i} \times 100$$

where:

$\Delta_s$ = volume shrinkage in percentage of initial volume,
$v_i$ = initial volume of specimen (height of specimen times area of ring container), and
$v_d$ = volume of air-dried specimen from mercury displacement method.

Calculate the shrinkage in height as follows:

$$\Delta hs = \frac{h_i - h_d}{h_i} \times 100$$

where:

$\Delta hs$ = height of shrinkage in percentage of initial height,
$h_i$ = initial height of specimen, and
$h_d$ = height of air-dried specimen.

6.3.1 To calculate the total percentage change in volume from "air-dry to saturated conditions," add the percentage shrinkage in volume on air drying $\Delta s$ to the percentage expansion in volume on saturation $\Delta e$, as described in 6.1. This value is used as an indicator of total expansion but is based on initial conditions of density and water content. Since expansion volume data are determined for several conditions of loading, the total volume change can also be determined for several conditions of loading.

6.3.2 To calculate the total percentage change in height from saturated to air-dry conditions, add the percentage shrinkage in height $\Delta hs$ to the percentage expansion $\Delta e$ when the specimen is saturated under specific load conditions.

## 7. Plotting Test Data

7.1 *Expansion Test* – The test data may be plotted as shown on Figure 2.7.

## 8. Reports

8.1 *Expansion Test* – Include the following information on the soil specimens tested in the report:

8.1.1 Identification of the sample (hole number, depth, location).

8.1.2 Description of the soil tested and size fraction of the total sample tested.

8.1.3 Type of sample tested (remolded or undisturbed; if undisturbed, describe the size and type, as extruded core, hand-cut, or other).

8.1.4 Initial moisture and density conditions and degree of saturation (if remolded, give the comparison to maximum density and optimum water content (see Methods D 698)).

8.1.5 Type of consolidometer (fixed or floating ring, specimen size), and type of loading equipment.

8.1.6 A plot load versus volume change curves as in Fig. 1. A plot of void ratio versus log of pressure curve may be plotted if desired.

8.1.7 A plot log of time versus deformation if desired.

8.1.8 Load and time versus volume-change data in other forms if specifically requested.

8.1.9 Final water content, bulk dry density, and saturation degree data.

8.1.10 Permeability data and any other data specifically requested.

8.2 *Shrinkage Test* – For the report on shrinkage, include data on the decrease in volume from the initial to air-dried condition and, if desired, other information such as the total change in volume and total change in height. Report the load conditions under which the volume change measurements were obtained. Include also Items 8.1.1 through 8.1.5 and 8.1.9.

## INTRODUCTION

The design procedure described in this chapter is developed from the analysis of pavement sections in the tropics. The procedure is based on the relationship between performance and deflections and the relationship between deflection and the structural strength of each component layer within the pavement structure.

The design procedure has been termed "Tropical Design Procedures For Flexible Pavements" because it was developed principally for red tropical soils which form the basic structural layers, i.e., base, subbase and subgrade. However, the procedure has wider application and can be used in most temperate climates where pavements are not subjected to frost penetration.

The design procedure is to be used by qualified pavement engineers with the experience and background necessary to determine the appropriate design parameters. The design procedure has been simplified through the use of Design Tables only to reduce the computational task in dimensioning the various pavement layers. The engineer can then devote more of his effort in the analysis of the strength characteristics of the materials he has selected for the component layers.

## STRUCTURAL DESIGN CURVES

The structural design curve is a basic element of the flexible pavement design procedure described below. It illustrates the relationship between the design traffic and the structural index, a measure of pavement strength (see Figure 3.1).

In the figure, the design traffic is given in terms of standard axle applications in both directions and can be determined in Brazilian designs from Figure 3.2 by multiplying the design daily equivalent axle loading by two and again by the appropriate design period in days. Similar relationships should be developed for the area in which the design procedure is to be used. The coefficient of variation is a reflection of local construction practices and is determined by analysing the actual deflections of existing roads in the proposed design area. The Appendix to Chapter 3 describes the method used to determine the coefficient of variation. The structural index (SI) represents the minimum required strength for a standard 90 cm pavement section and, once determined, it forms the basis for a flexible pavement design.

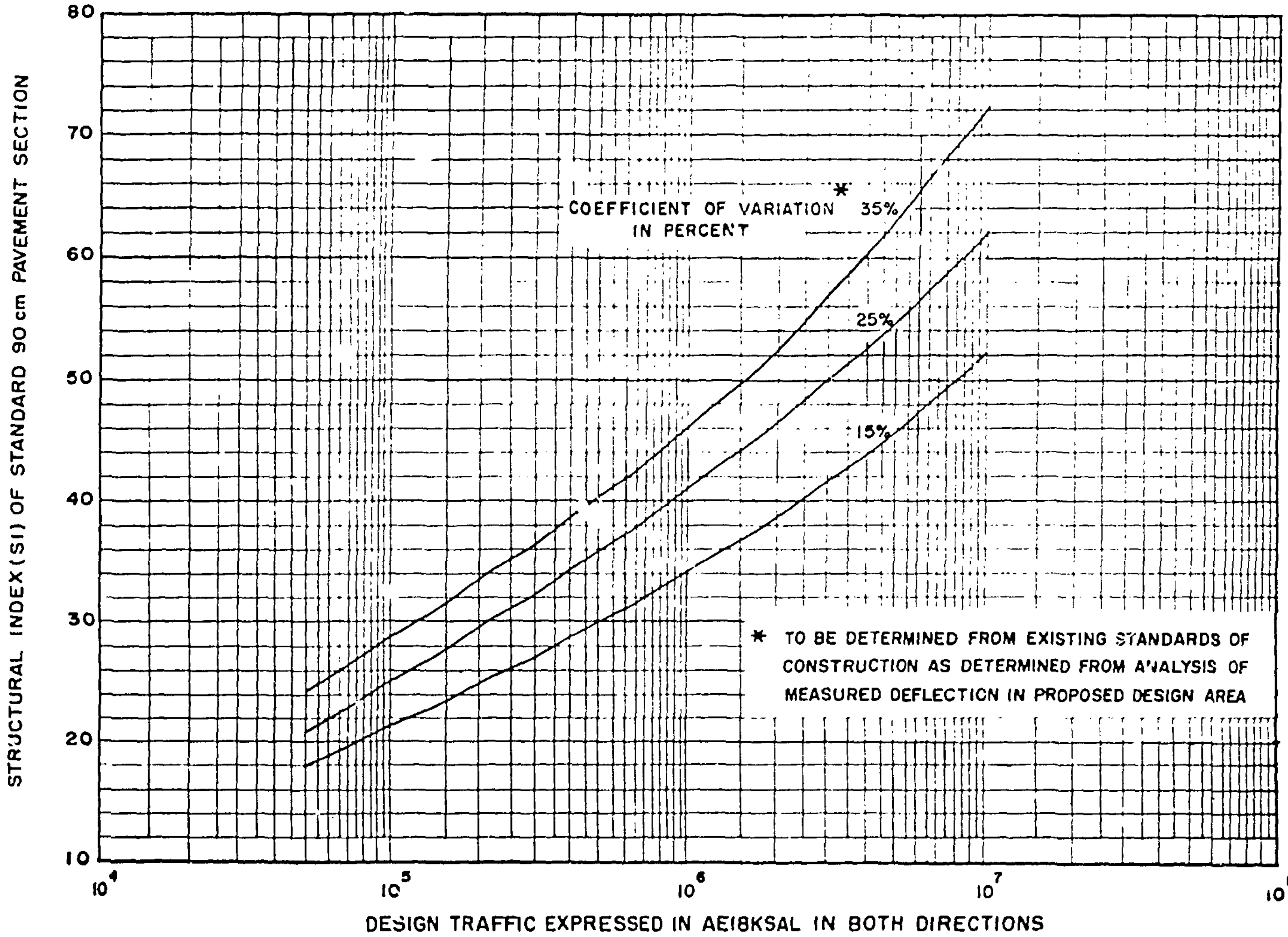



FIGURE 3.1 – STRUCTURAL DESIGN CURVES FOR DETERMINING THE REQUIRED STRUCTURAL INDEX OF THE STANDARD PAVEMENT SECTION

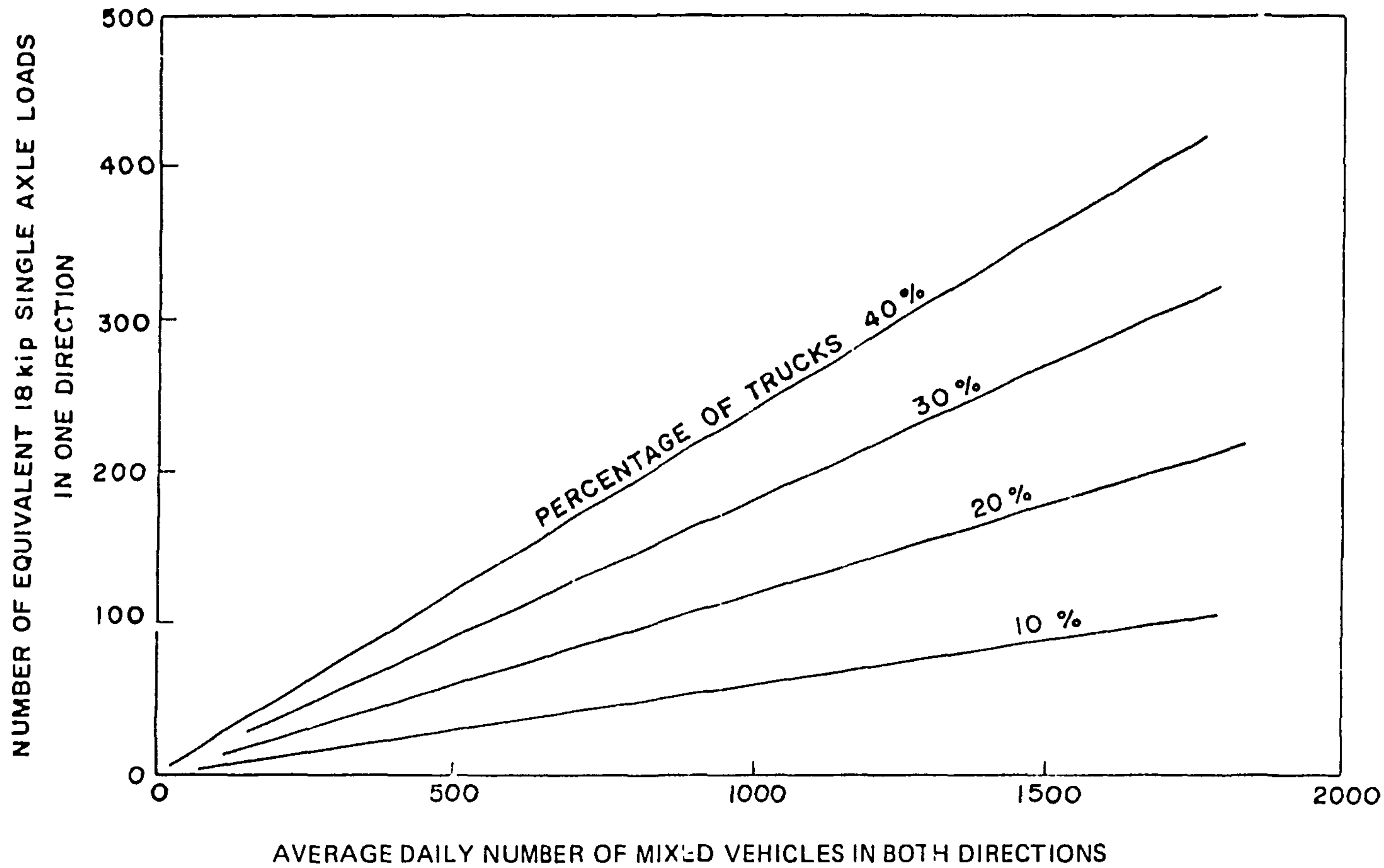



FIGURE 3.2 – GENERAL RELATIONSHIP BETWEEN TOTAL DAILY NUMBER OF VEHICLES AND EQUIVALENT 18KSAL–CURVES (BASED ON VEHICLE AND LOAD DISTRIBUTION OBTAINED FROM LOADOMETER STUDY IN BRAZIL)

## STRUCTURAL DESIGN PROCESS

The objective of the structural design process is to use the structural index to determine the thickness of each layer of a pavement section, i.e., the surface, base course and subbase course. All that is required to use the procedure are the design traffic, the CBR of the base, subbase and subgrade, the coefficient of variation and the figures and tables provided in this chapter.

The structural design equations are shown below. The equations show that the structural index (SI) of a pavement section is determined by summing the structural indexes of each layer in a standard 90 cm pavement section. The equations are:

1) Structural Index (SI) = $a_1 t_1 + a_2 t_2$ ..... $a_n t_n$; and
2) $t_1 + t_2 +$ .... $t_n = 90$ cm.

In the first equation $a_1$, $a_2$, etc., are the structural coefficients of the materials used in succeeding layers of the pavement section. Structural coefficients for various unbound soils are summarized in Table 3.1. In addition, Table 3.1 shows the structural coefficients for crushed stone, cement-treated soils and lime-treated soils. The coefficient for crushed stone was estimated from test sections with macadam base courses. The coefficients reflect the load-spreading characteristics of the materials. Table 3.1 shows that the structural coefficient of crushed stone (open graded) is less than that of higher quality concretionary gravels. The better load-spreading characteristics of the concretionary gravel (with CBR's greater than 80) are due to cohesion. The structural coefficients for cement-treated and lime-treated soils are estimated from relationships given in the AASHO Interim Design Guide (1972).

In the above equation, $t_1$, $t_2$, etc., represent the thickness of each succeeding pavement layer of a standard 90 cm design section. Design thicknesses are linked to CBR values of the underlying pavement layer except for the subgrade. The design CBR–thickness relationship is given in Figure 3.3.

The strength of the subgrade, initially expressed as the subgrade CBR, is a key element in the design of the pavement structural layers, e.g., surface, base and subbase. Its importance in the design is demonstrated in Figure 3.4. This figure which is partially replotted from Figure 3.3 shows that the thickness of cover (the pavement structural layers) varies inversely with subgrade CBR values. The figure also shows the relationship between subgrade CBR and structural index. Since the structural index of the subgrade influences the design thickness of the structural layers it is referred to simply as subgrade support. Therefore, higher strength subgrades furnish greater support and permit the use of thinner structural layers. The combined structural index for the structural layer (surface, base and subbase) is equivalent to the required structural index (from Figure 3.1) less the subgrade support.

TABLE 3.1
**Pavement Coefficients for Flexible Pavement Design**

| Pavement Component Strength | Coefficient |
|---|---|
| **Surface Course** | |
| Asphalt Concrete-Sand Asphalt | |
| 5 cm | Figure 3.5 |
| 10 cm | Figure 3.6 |
| 15, 20 and 25 cm | Figure 3.7a, b, & c |
| Note: Bituminous treated macadam included in asphalt layer thickness | |
| **Base Course** | |
| Crushed Stone (Macadam hydraulic) | 1.037 open graded |
| | 1.394 graded |
| Cement Treated | |
| Compressive strength 7 days | |
| 650 psi or more | 2.400* |
| 400 psi to 650 psi | 2.100* |
| 400 psi or less | 1.600* |
| Lime Treated | 1.400 - 1.600* |
| Concretionary Gravels | |
| CBR (Design) | |
| + 100 | 1.394 |
| 90 | 1.232 |
| 85 | 1.167 |
| 80 | 1.102 |
| 75 | 1.037 |
| 70 | 0.940 |
| 60 | 0.552 |
| 50 (min)[1] | 0.383 |
| **Subbase Course** | |
| CBR (Design) | |
| + 40 | 0.576 |
| 35 | 0.290 |
| 30 | 0.205 |
| 25 (min) | 0.075 |
| **Subgrade Layer** | |
| CBR (Design) | |
| + 20 | 0.481 |
| 15 | 0.357 |
| 10 | 0.212 |
| 9 | 0.183 |
| 8 | 0.133 |
| 7 | 0.084 |
| 6 | 0.053 |
| 5 | 0.033 |
| 4 | 0.020 |
| 3 | 0.015 |
| 2 (min) | 0.010 |

**Design Coefficient Limits**

**Base Course refers to materials to a depth of 25 cm.**
**Subbase Course refers to material layers between 25-50 cm.**
**Subgrade Layer refers to material layers between 50-90 cm.**

* Values estimated from structural coefficient relationships given in AASHO Interim Guide for Design of Pavement Structures 1972.

1 Material with a CBR of 40 can be used between the depth intervals of 10 and 25 cm and assigned the same coefficient.

Note: Coefficients are for the metric system.

## FLEXIBLE PAVEMENT DESIGN

It may take more than one trial to design a pavement using the flexible pavement design process outlined herein. The first trial begins with the selection of a surface type and concludes with the determination of the base and subbase thickness.

The surface type and thickness is determined from Table 3.2 and is based on the base course CBR values and the design traffic.

The minimum design thickness of the unbound structural layers (base and subbase) in a double bituminous surface treatment (DBST) pavement design is equal to the minimum thickness of cover derived from Figure 3.4. The thickness of the DBST pavement course is not considered since it does not contribute strength to the pavement section. However, an asphalt concrete (AC) surface course provides a load-spreading characteristics so it is considered part of the minimum thickness of cover. Therefore in an AC pavement design the minimum design thickness of the unbound structural layers equals the minimum thickness of cover less the thickness of AC.

The minimum base course thickness is determined from Figure 3.3 and is based on the CBR of the subbase materials. The subbase course is the remaining thickness of cover for the subgrade. Neither the base or the subbase course should be designed for less than 10 cm, since it is impractical to construct a thinner layer. Therefore it may be necessary to over-design the base or subbase course in order to account for this practical problem. Finally, the subgrade thickness for design purposes is the difference between the standard 90 cm design section and the thickness of cover.

The adequacy of this first trial design section to carry the design traffic is determined by summing the structural indexes of each pavement layer and comparing it to the required structural index from the Structural Design Curves, Figure 3.1. The structural index of each pavement layer is equivalent to the product of its thickness and its structural coefficient (Table 3.1) but only insofar as the thickness and CBR value fall within the strength and design coefficient limits provided in the Table 3.1. Those portions of the pavement section not falling within these limits do not provide strength to the pavement and therefore are excluded from the summation of the structural index of the entire pavement section even though they are a part of the 90 cm pavement section.

### Double Bituminous Surface Treatment

DBST pavement design is the least complex flexible pavement design since the DBST surface does not impart any strength to the pavement. Design example 1 below utilizes the principles, figures and tables described above to solve a simple pavement design problem.

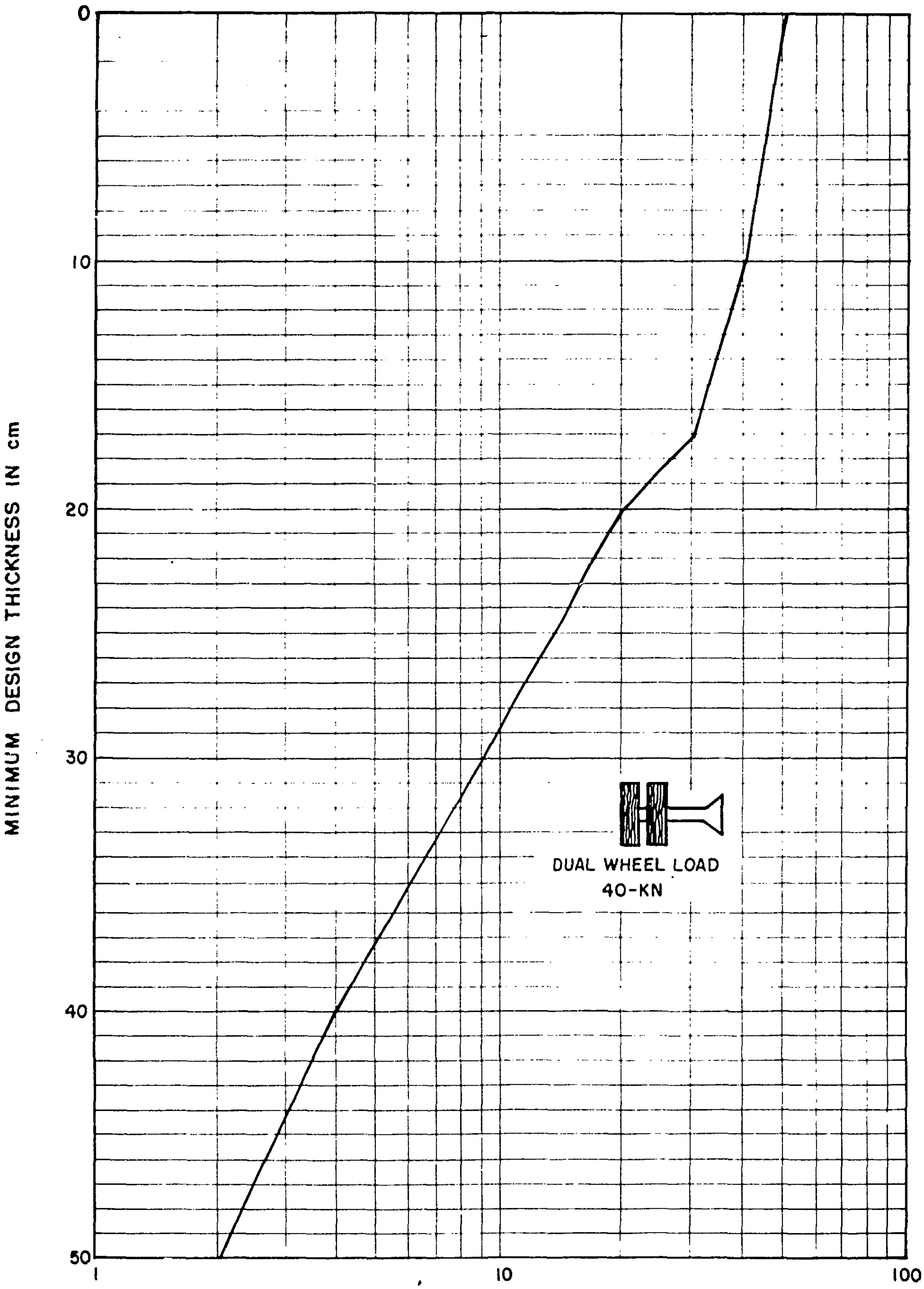



FIGURE 3.3 – MINIMUM DESIGN THICKNESS FOR ALL PAVEMENT COURSES BASED ON CBR VALUES OF UNDERLYING LAYERS

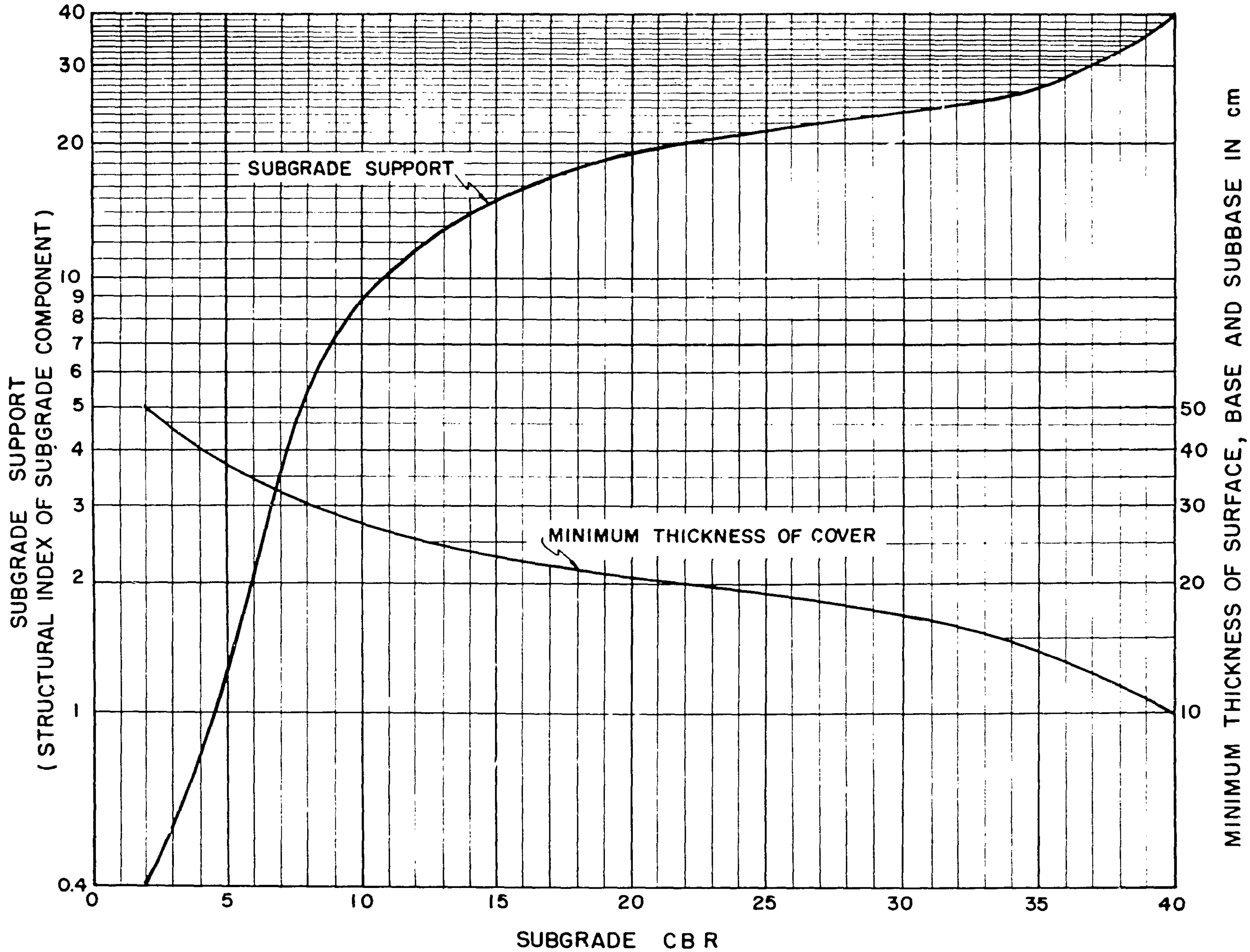



FIGURE 3.4 – SUBGRADE SUPPORT AND MINIMUM THICKNESS OF COVER BASED ON SUBGRADE CBR VALUES

## TABLE 3.2
**Recommended Type and Thickness of Surface Courses – cm**

| Total Equivalent Standard Axle Load Applications Both Directions | Strength of Base Course – CBR | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | + 100 | 90 | 85 | 80 | 75 | 70 | 60 | 50 |
| 100,000 | ST[1] | ST | ST | ST | ST | ST | ST | ST |
| 200,000 | ST | ST | ST | ST | ST | ST | ST | 10[2] |
| 300,000 | ST | ST | ST | ST | ST | ST | 15 | 15 |
| 400,000 | ST | ST | ST | ST | ST | 10 | 15 | 15 |
| 500,000 | ST | ST | ST | ST | 10 | 10 | 15 | 15 |
| 600,000 | ST | ST | ST | 10 | 10 | 10 | 15 | 15 |
| 700,000 | ST | ST | 5 | 10 | 10 | 10 | 20[3] | 20[3] |
| 800,000 | ST | 5 | 5 | 10 | 10 | 10 | 20 | 20 |
| 900,000 | ST | 5 | 5 | 10 | 10 | 10 | 20 | 20 |
| 1,000,000 | ST | 5 | 5 | 10 | 10 | 10 | 20 | 20 |
| 2,000,000 | 5 | 5 | 5 | 10 | 10 | 10 | 20 | 20 |
| 3,000,000 | 5 | 5 | 5 | 15 | 15 | 15 | 20 | 20 |
| 4,000,000 | 5 | 5 | 5 | 15 | 15 | 15 | 20 | 20 |
| 5,000,000 | 10 | 10 | 10 | 15 | 15 | 15 | 20 | 20 |
| 6,000,000 | 10 | 10 | 10 | 15 | 15 | 15 | 20 | 20 |
| 7,000,000 | 10 | 10 | 10 | 15 | 15 | 15 | 20 | 20 |
| 8,000,000 | 10 | 10 | 10 | 15 | 15 | 15 | 20 | 20 |
| 9,000,000 | 10 | 10 | 10 | 15 | 15 | 15 | 20 | 20 |
| 10,000,000 | 10 | 10 | 10 | 20[3] | 20[3] | 20[3] | 25 | 25 |

Notes: 1 ST denotes a double bituminous surface treatment (2) numbers denotes the thickness of asphalt concrete in cm and (3) to be used only if higher quality base base material is not available and stabilization or modification proves to be too expensive.

## DESIGN EXAMPLE 1

Given:
- a) Estimated design traffic = 125,000 SAL (both directions)
- b) Measured construction variation of nearby highways = 35 percent
- c) Subgrade CBR = 9
- d) Available base material CBR = 80 and Subbase material CBR = 30.

Find: Practical pavement design

Solutions:

**Step 1: Determine the required Structural Index (SI)**

The design curves Figure 3.1 are used to determine the SI. Enter the horizontal axis at the 125,000 design traffic and turn to the vertical axis upon intersection with 35 percent coefficient of variation curve. The SI equals 30.

**Step 2: Determine Surface Type and Thickness**

The recommended thickness of surface conditions are given in Table 3.2. A DBST surface is recommended for a 125,000 SAL design traffic when the base course CBR is 80.

**Step 3: Determine Pavement Layer Thicknesses**

The minimum thicknesses of cover over each pavement layer are obtained from Figure 3.3. They are as follows:

Min. Cover over subgrade (surface, base & subbase) = 30 cm for subgrade CBR of 9.

Min. Cover over subbase (surface and base) = 17 cm for subbase CBR of 30.

Min. Cover over base (surface) = 0 for base CBR of 80.

Therefore assume a irst trial design thickness for each layer as follows:

DBST Surface = 0 (since DBST does not impart strength to the pavement).

Base course = 17 – 0 = 17 say 20 cm

Subbase course = 30 – 20 = 10 cm

Subgrade = 90 (standard section) – 30 = 60 cm

**Step 4: Determine the Subgrade Support Index**

The overall required structural index is first reduced by an amount equal to the subgrade SI which can be obtained directly from Figure 3.4. Enter the figure at CBR 9 on the horizontal axis and turn to the left vertical axis upon intersection with the subgrade support curve.

The subgrade support SI = 7.

Step 5: **Determine the Structural Index of the Unbound Soil Layers**

The unbound soil layer SI is the summation of the product of the structural coefficients of the materials used in a "standard" base and subbase course. Table 3.1 provides coefficients for various materials.

Base course coefficients are applicable only between 0 and 25 cm from the surface. Subbase course coefficients are applicable only between 25 and 50 cm below the surface. The base and subbase course thicknesses used in determining the SI are referred to as the "standard" base and subbase course and are not to be confused with base and subbase design thicknesses. Determination of the unbound soil layer SI follows:

SI unbound soil layer = [Subbase course SI $_{(50\text{-}25\ cm)}$] + [Base course SI$_{(25\text{-}0\ cm)}$]

= [(Coefficient of CBR 9 x 20 cm) + (Coefficient of CBR 30 x 5 cm)] +

= [(Coefficient of CBR 30 x 5 cm) + (Coefficient of CBR 80 x 20 cm)]

= [(0 x 20) + (0.205 x 5)] + [(0 x 5) + (1.102 x 20)]

= [1.02] + [22.04]

= 23.06

Step 6: **Determine the Adequacy of the First Trial Design**

The required SI (Step 1) = 30.

The calculated SI (Step 4 + Step 5) = 7 + 23.06 = 30.06.

Therefore the first trial design is adequate to serve the estimated design traffic in a region where the coefficient of variation in construction is 35 percent.

Design: DBST surface 20 cm Base and 10 cm Subbase.

### Asphalt Concrete

The Asphalt Concrete (AC) pavement design is more complex because the AC surface influences the design. As explained earlier, the AC surface provides for a better load-spreading characteristic in the underlying unbound soil layers. Consequently, the strength component of the underlying layers does not have to be as great as it would if it supported a DBST surface. The benefit of the load-spreading characteristics of asphalt concrete is illustrated in Figures 3.5, 3.6, 3.7a, 3.7b, and 3.7c for 5, 10, 15, 20 and 25 cm AC surfaces respectively.

It should be noted that the load-spreading benefit extends only 50 cm below the surface for the 5 and 10 cm AC surfaces. It extends throughout the entire 90 cm design section for the 15, 20 and 25 cm AC surface. Design example 2 below shows how these figures are utilized.

## DESIGN EXAMPLE 2

Given: a) Estimated design traffic = 1,250,000 SAL (both directions)

b), c) and d) same as Example 1

Find: A practical pavement design

Solution:

Step 1: **Determine the Required Structural Index (SI)**

From Figure 3.1 a design traffic of 1,250,000 SAL and a coefficient of variation of 35 percent require a SI of 48.

Step 2: **Determine Surface Type and Thickness**

From Table 3.2 asphalt concrete (AC) surface is required for a SAL between a 1,000,000 and 2,000,000 when the base course CBR is 80.

Step 3: **Determine Pavement Layer Thicknesses**

Same as example 1 except the 10 cm AC surface contributes strength to the pavement and replaces the top 10 cm of base course material. Therefore the first trial design thicknesses are as follows:

| | | |
|---|---|---|
| AC surface | = | 10 cm |
| Base | = | 10 cm |
| Subbase | = | 10 cm |
| Subgrade | = | 60 cm |

Step 4 : **Determine the Subgrade Support Index**

Same as Example 1, i.e. subgrade support SI = 7 from Figure 3.4.

Step 5 : **Determine the Structural Index of the Unbound Soil Layers** SI unbound soil layer

SI unbound soil layer = [subbase course SI $_{(50\text{-}25\ cm)}$ + [Base course SI $_{(25\text{-}10\ cm)}$]

= [(Coefficient of CBR 9 x 20 cm) + (Coefficient of CBR 30 x 5 cm)] +

[(Coefficient of CBR 30 x 5 cm) + (Coefficient of CBR 80 x 10 cm)]

= [(0 x 20) + (0.205 x 5)] + [(0 x 5) + (1.102 x 10)]

= [(1.02] + [11.02]

= 12.04, or 12

Step 6 : **Determine the Combined Structural Index of all Structural Layers**

The AC surface provides direct strength to the pavement and also imparts a special load-spreading characteristic to the unbound soil layers. Therefore the AC concurrently increases the strength of the pavement and reduces the stress on the unbound structural layer. The combined structural index which accounts for these phenomena in a 10 cm AC surface is shown in Figure 7.6. Enter the figure with the SI computed in Step 5 (12) in the horizontal axis and turn to the vertical axis upon intercepting the curve. The combined structural index is 41.

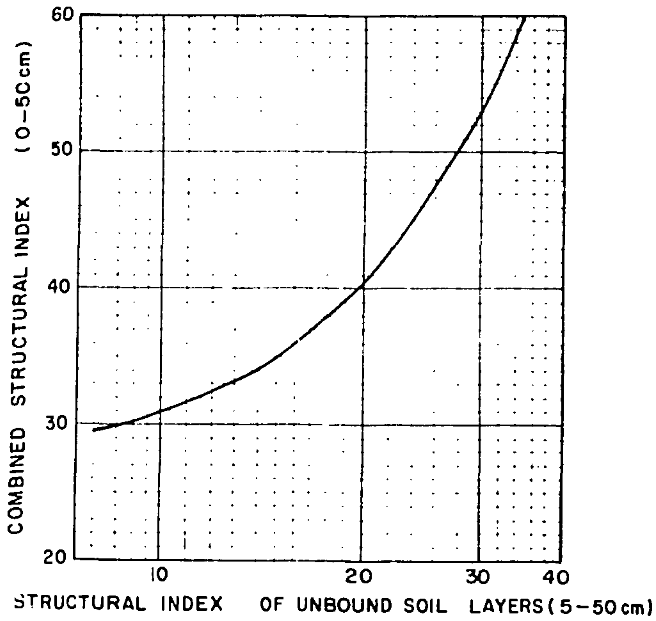



FIGURE 3.5 – COMBINED STRENGTH OF ASPHALT SECTION (0–50 cm) OF ASPHALT AND UNBOUND SOIL LAYERS–5 cm ASPHALT CONCRETE

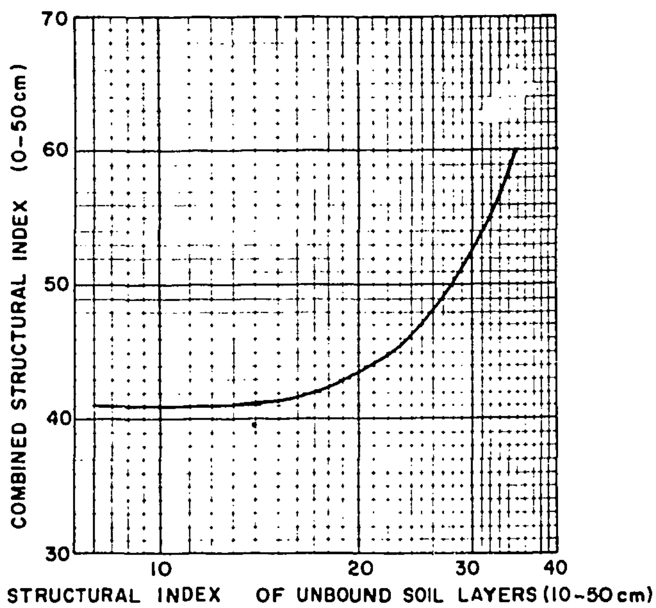



FIGURE 3.6 – COMBINED STRENGTH OF ASPHALT SECTION (0–50 cm) OF ASPHALT AND UNBOUND SOIL LAYERS–10 cm ASPHALT CONCRETE

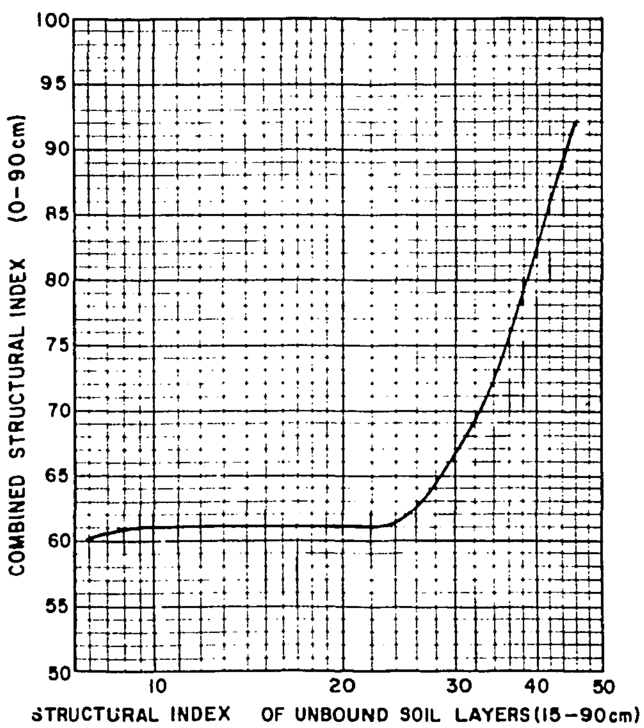



FIGURE 3.7a – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS–15 cm ASPHALT CONCRETE

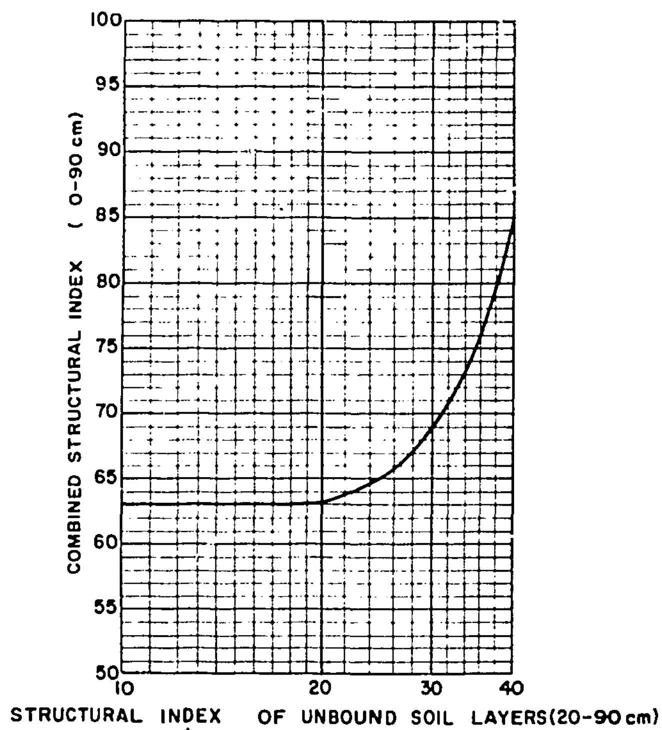



FIGURE 3.7b – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS–20 cm ASPHALT CONCRETE

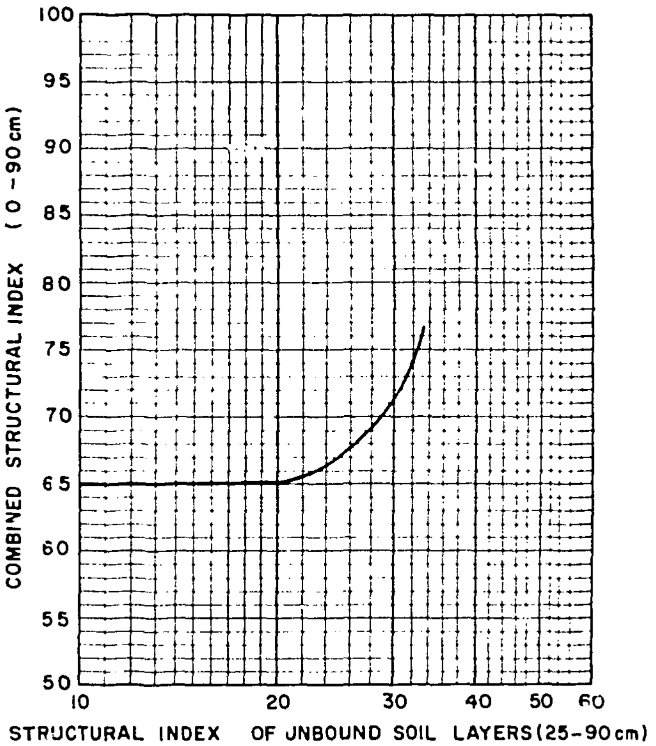



FIGURE 3.7c – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS–25 cm ASPHALT CONCRETE

Step 7 : **Determine the Adequacy of the First Trial Design**

The required SI (Step 1) = 48.
The calculated SI (Steps 4 and 6) = 7 + 41 = 48.
Therefore the first trial design is adequate.

Design: 10 cm AC, 10 cm Base and 10 cm Subbase.

The design examples show pavement designs that meet the structural index requirement. However, if the calculated structural index had exceeded the required index this would have indicated that the section was over-designed. In such an event it may be desirable to re-design the pavement for economic reasons; otherwise the design is considered satisfactory. Should a re-design be undertaken for an over-designed pavement section it is possible that some savings may be realized by using materials with lower CBR values. Such a re-design must provide 1) that the minimum design thicknesses (Figure 3.3) are met; and 2) that the re-design does not result in a required change from DBST surface to AC surface or to an increase of the AC surface thickness, since such changes would result in higher cost. It should also be noted that if the CBRs of the lower strength materials fall below those shown in Table 3.1, then these materials can not be assigned a structural coefficient for the purpose of determining the structural index.

If the computed structural index in the design examples were lower than the required structural index (Figure 3.1) then five basic alternatives are available to the design engineer to improve the strength of the pavement design. They are:

- Increase the subbase thickness. This increases the minimum thickness of cover but does not affect the surface and base courses.
- Increase the base thickness and reduce the subbase by a like amount. This design retains the minimum thickness of cover.
- Use an AC surface instead of a DBST surface or increase the AC surface thickness and reduce the thickness of the base or subbase course by a like amount. This design also retains the minimum thickness of cover.
- Stabilize the base course to increase the base course coefficient. This retains all of the first trial thicknesses.
- Any combination of the above.

The engineering decision to undertake one of the above alternatives will be based on economic considerations, local conditions and the availability of the materials required to increase the structural index.

## DESIGN TABLES

Design tables have been developed using procedures described above. Sixteen structural design tables follow that provide the engineer with an easy procedure for designing pavement sections with various combinations of surface types and subgrade, subbase and base course CBR's. Four standard thicknesses of base course and three standard thicknesses of asphalt are provided. The thickness of the subbase is fixed by the required minimum thickness of cover (Figure 3.4). For a given thickness of asphalt, a corresponding thickness of base course is shown where the combined thickness of asphalt and base course is equal to 25 cm. Since the structural coefficients change at a depth of 25 cm, an additional thickness of base course material would be uneconomical in design. A high-quality subbase material does as well.

Numerous design combinations are available in the tables. As the tables are based on minimum thickness of cover, the strength of many sections can be increased by adding an incremental thickness of subbase. Exceptions are the 15 cm asphalt surface tables. In all the tables the pavement combinations represent a reduction in deflection through a reduction in stress. However in the 15 cm tables this apparently reaches a maximum value as additional reduction in deflection for many of the sections in the lower half of the tables can only be obtained by increasing the shearing resistance of the component layers, i.e., by moving upward in the table (stabilization of the subgrade) and then to the right (stabilization of the base course). Examples of using the Structural Design Tables follow.

# DESIGN TABLE 1

**DBST – Subbase CBR-25**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thicness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR= +100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 20 | | | 28 | 30 | | | 31 | 34 | | | 39 | 43 | | | 42 | 47 | | | 45 | 51 | | | 48 | 55 | 10 | 5 | – | – |
| 15 | | | 22 | 24 | | | 25 | 28 | | | 33 | 38 | | | 36 | 42 | | | 39 | 45 | | | 42 | 49 | 13 | 8 | 3 | – |
| 10 | | | 16 | 18 | | | 20 | 23 | | | 28 | 32 | | | 31 | 36 | | | 33 | 40 | | | 37 | 44 | 19 | 14 | 9 | 4 |
| 9 | | | 15 | 17 | | | 19 | 21 | | | 26 | 31 | | | 30 | 35 | | | 32 | 38 | | | 36 | 43 | 20 | 15 | 10 | 5 |
| 8 | | | | 15 | | | 17 | 20 | | | 25 | 29 | | | 28 | 33 | | | 30 | 37 | | | 34 | 41 | 21 | 16 | 11 | 6 |
| 7 | | | | | | | | 17 | | | 22 | 27 | | | 26 | 31 | | | 28 | 34 | | | 31 | 38 | 23 | 18 | 13 | 8 |
| 6 | | | | | | | | 17 | | | 22 | 26 | | | 25 | 30 | | | 28 | 34 | | | 31 | 38 | 25 | 20 | 15 | 10 |
| 5 | | | | | | | | 16 | | | 21 | 26 | | | 24 | 30 | | | 27 | 33 | | | 30 | 37 | 28 | 23 | 18 | 13 |
| 4 | | | | | | | | 16 | | | 21 | 26 | | | 24 | 30 | | | 27 | 33 | | | 30 | 37 | 30 | 25 | 20 | 15 |
| 3 | | | | | | | | 16 | | | 21 | 26 | | | 24 | 30 | | | 27 | 33 | | | 30 | 37 | 34 | 29 | 24 | 19 |
| 2 | | | | | | | | 16 | | | 21 | 26 | | | 24 | 30 | | | 27 | 33 | | | 30 | 37 | 40 | 35 | 30 | 25 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 2

AC 5 cm – Subbase CBR-25

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | | | | | | | | | | | | | | | | | | | | | | | | 2 | – | – | – |
| 20 | | – | 46 | | | 46 | 50 | | | 53 | 58 | | | 56 | 63 | | | 57 | 65 | | | 60 | 65 | | 5 | – | – | – |
| 15 | | – | 44 | | | 44 | 45 | | | 49 | 54 | | | 51 | 57 | | | 52 | 60 | | | 55 | 64 | | 8 | 3 | – | – |
| 10 | | – | 38 | | | 38 | 41 | | | 43 | 48 | | | 46 | 51 | | | 47 | 55 | | | 50 | 58 | | 14 | 9 | 4 | – |
| 9 | | – | 37 | | | 37 | 39 | | | 41 | 47 | | | 45 | 50 | | | 46 | 54 | | | 49 | 57 | | 15 | 10 | 5 | – |
| 8 | | – | 35 | | | 35 | 37 | | | 40 | 45 | | | 43 | 49 | | | 44 | 52 | | | 47 | 56 | | 16 | 11 | 6 | 1 |
| 7 | | – | 33 | | | 33 | 35 | | | 38 | 43 | | | 41 | 47 | | | 43 | 50 | | | 45 | 54 | | 18 | 13 | 8 | 3 |
| 6 | | – | 32 | | | 32 | 34 | | | 37 | 42 | | | 40 | 46 | | | 42 | 49 | | | 44 | 52 | | 20 | 15 | 10 | 5 |
| 5 | | – | 31 | | | 31 | 33 | | | 37 | 41 | | | 39 | 45 | | | 41 | 48 | | | 44 | 52 | | 23 | 18 | 13 | 8 |
| 4 | | – | 31 | | | 31 | 33 | | | 36 | 41 | | | 38 | 45 | | | 41 | 48 | | | 44 | 52 | | 25 | 20 | 15 | 10 |
| 3 | | – | 31 | | | 31 | 33 | | | 36 | 41 | | | 39 | 45 | | | 41 | 48 | | | 44 | 52 | | 29 | 24 | 19 | 14 |
| 2 | | – | 31 | | | 31 | 34 | | | 36 | 42 | | | 39 | 45 | | | 41 | 48 | | | 44 | 52 | | 35 | 30 | 25 | 20 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 3

**AC 10 cm — Subbase CBR-25**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade 63 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 40 | 63 | 63 | | | 64 | 64 | | | 67 | 70 | | | 69 | 73 | | | 70 | 76 | | | 72 | 79 | | | – | – | – | – |
| 30 | 60 | 60 | | | 60 | 60 | | | 60 | 62 | | | 61 | 64 | | | 62 | 65 | | | 62 | 67 | | | – | – | – | – |
| 20 | – | – | | | – | 60 | | | 60 | 61 | | | 60 | 61 | | | 60 | 62 | | | 61 | 63 | | | – | – | – | – |
| 15 | – | – | | | – | 55 | | | 55 | 56 | | | 55 | 56 | | | 55 | 57 | | | 56 | 59 | | | 3 | – | – | – |
| 10 | – | – | | | – | 50 | | | 50 | 50 | | | 50 | 51 | | | 50 | 51 | | | 50 | 53 | | | 9 | 4 | – | – |
| 9 | – | – | | | – | 48 | | | 48 | 49 | | | 48 | 49 | | | 48 | 50 | | | 49 | 51 | | | 10 | 5 | – | – |
| 8 | – | – | | | – | 46 | | | 46 | 47 | | | 46 | 47 | | | 46 | 48 | | | 47 | 50 | | | 12 | 7 | 2 | – |
| 7 | – | – | | | – | 44 | | | 44 | 45 | | | 44 | 45 | | | 44 | 46 | | | 45 | 48 | | | 13 | 8 | 3 | – |
| 6 | – | – | | | – | 43 | | | 43 | 44 | | | 43 | 44 | | | 43 | 45 | | | 44 | 46 | | | 15 | 10 | 5 | – |
| 5 | – | – | | | – | 42 | | | 42 | 43 | | | 42 | 44 | | | 42 | 44 | | | 43 | 46 | | | 18 | 13 | 8 | 3 |
| 4 | – | – | | | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 44 | | | 42 | 46 | | | 20 | 15 | 10 | 5 |
| 3 | – | – | | | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 44 | | | 42 | 46 | | | 24 | 19 | 14 | 9 |
| 2 | – | – | | | – | 41 | | | 41 | 42 | | | 41 | 43 | | | 42 | 44 | | | 42 | 46 | | | 30 | 25 | 20 | 15 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 4

**AC 15 cm – Subbase CBR-25**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade 76 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 30 | Asphalt Placed Directly on Subgrade 62 | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 20 | 61 | | | | 62 | | | | 64 | | | | 67 | | | | 67 | | | | 69 | | | | – | – | – | – |
| 15 | 61 | | | | 61 | | | | 62 | | | | 63 | | | | 64 | | | | 64 | | | | – | – | – | – |
| 10 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 4 | – | – | – |
| 9 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 5 | – | – | – |
| 8 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 6 | 1 | – | – |
| 7 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 8 | 3 | – | – |
| 6 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 10 | 5 | – | – |
| 5 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 13 | 8 | 3 | – |
| 4 | – | | | | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 15 | 10 | 5 | – |
| 3 | – | | | | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 19 | 14 | 9 | 4 |
| 2 | – | | | | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 25 | 20 | 15 | 10 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 5

**DBST – Subbase CBR-30**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | | 32 | 34 | | | 35 | 38 | | | 43 | 48 | | | 46 | 52 | | | 49 | 55 | | | 52 | 59 | 7 | 2 | – | – |
| 20 | | | 28 | 30 | | | 31 | 34 | | | 39 | 43 | | | 42 | 47 | | | 5 | 51 | | | 48 | 55 | 10 | 5 | – | – |
| 15 | | | 22 | 24 | | | 25 | 28 | | | 33 | 38 | | | 36 | 42 | | | 39 | 45 | | | 42 | 49 | 13 | 8 | 3 | – |
| 10 | | | 17 | 19 | | | 20 | 23 | | | 28 | 33 | | | 31 | 37 | | | 34 | 40 | | | 37 | 44 | 19 | 14 | 9 | 4 |
| 9 | | | 16 | 18 | | | 19 | 22 | | | 27 | 32 | | | 30 | 36 | | | 33 | 39 | | | 36 | 43 | 20 | 15 | 10 | 5 |
| 8 | | | – | 16 | | | 18 | 20 | | | 25 | 30 | | | 29 | 34 | | | 31 | 37 | | | 34 | 41 | 21 | 16 | 11 | 6 |
| 7 | | | – | 15 | | | 16 | 19 | | | 24 | 29 | | | 27 | 33 | | | 29 | 36 | | | 33 | 40 | 23 | 18 | 13 | 8 |
| 6 | | | – | – | | | 15 | 18 | | | 23 | 28 | | | 26 | 32 | | | 28 | 35 | | | 32 | 39 | 25 | 20 | 15 | 10 |
| 5 | | | – | – | | | 15 | 18 | | | 23 | 27 | | | 26 | 31 | | | 28 | 35 | | | 32 | 39 | 28 | 23 | 18 | 13 |
| 4 | | | – | – | | | 15 | 18 | | | 23 | 27 | | | 26 | 31 | | | 28 | 35 | | | 32 | 39 | 30 | 25 | 20 | 15 |
| 3 | | | – | – | | | 15 | 18 | | | 23 | 28 | | | 27 | 32 | | | 29 | 35 | | | 32 | 39 | 34 | 29 | 24 | 19 |
| 2 | | | – | – | | | 17 | 19 | | | 24 | 29 | | | 28 | 33 | | | 30 | 36 | | | 33 | 40 | 40 | 35 | 30 | 25 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 6

### AC 5 cm – Subbase CBR-30

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+ 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | 51 | 52 | | | 53 | 56 | | | 59 | 64 | | | 61 | 68 | | | 64 | 72 | | | 67 | 76 | | 2 | – | – | – |
| 20 | | – | 46 | | | 46 | 50 | | | 53 | 58 | | | 56 | 63 | | | 57 | 65 | | | 60 | 65 | | 5 | – | – | – |
| 15 | | – | 44 | | | 44 | 45 | | | 49 | 54 | | | 51 | 57 | | | 52 | 60 | | | 55 | 64 | | 8 | 3 | – | – |
| 10 | | – | 38 | | | 39 | 42 | | | 44 | 49 | | | 47 | 53 | | | 49 | 56 | | | 52 | 61 | | 14 | 9 | 4 | – |
| 9 | | | 37 | | | 38 | 39 | | | 42 | 47 | | | 45 | 51 | | | 47 | 54 | | | 50 | 59 | | 15 | 10 | 5 | – |
| 8 | | – | 36 | | | 36 | 38 | | | 41 | 45 | | | 43 | 49 | | | 45 | 53 | | | 47 | 57 | | 16 | 11 | 6 | 1 |
| 7 | | – | 34 | | | 34 | 3[illegible] | | | 39 | 44 | | | 42 | 48 | | | 44 | 51 | | | 46 | 55 | | 18 | 13 | 8 | 3 |
| 6 | | 31 | 33 | | | 33 | 35 | | | 38 | 43 | | | 41 | 47 | | | 43 | 50 | | | 46 | 55 | | 20 | 15 | 10 | 5 |
| 5 | | 31 | 32 | | | 33 | 35 | | | 38 | 43 | | | 40 | 47 | | | 42 | 50 | | | 46 | 55 | | 23 | 18 | 13 | 8 |
| 4 | | 30 | 30 | | | 33 | 35 | | | 38 | 43 | | | 40 | 47 | | | 43 | 50 | | | 46 | 54 | | 25 | 20 | 15 | 10 |
| 3 | | 31 | 32 | | | 33 | 36 | | | 39 | 44 | | | 41 | 48 | | | 44 | 51 | | | 47 | 55 | | 29 | 24 | 19 | 14 |
| 2 | | 32 | 33 | | | 34 | 37 | | | 40 | 45 | | | 43 | 49 | | | 45 | 52 | | | 48 | 57 | | 35 | 30 | 25 | 20 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 7

**AC 10 cm — Subbase CBR-30**

| SG CBR | Base Course CBR and Thickness | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR=50 | | | | CBR=60 | | | | CBR=70 | | | | CBR=80 | | | | CBR=90 | | | | CBR=+ 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade = 63 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | 63 | 63 | | | 64 | 64 | | | 67 | 70 | | | 69 | 73 | | | 70 | 76 | | | 72 | 79 | | | – | – | – | – |
| 30 | 60 | 60 | | | 60 | 60 | | | 60 | 62 | | | 61 | 64 | | | 62 | 65 | | | 62 | 67 | | | – | – | – | – |
| 20 | – | – | | | – | 60 | | | 60 | 61 | | | 60 | 61 | | | 60 | 62 | | | 61 | 63 | | | – | – | – | – |
| 15 | – | – | | | – | 55 | | | 55 | 56 | | | 55 | 56 | | | 55 | 57 | | | 56 | 58 | | | 3 | – | – | – |
| 10 | – | – | | | – | 50 | | | 50 | 50 | | | 50 | 51 | | | 50 | 52 | | | 50 | 53 | | | 9 | 4 | – | – |
| 9 | – | – | | | | 48 | | | 48 | 49 | | | 48 | 50 | | | 48 | 50 | | | 49 | 52 | | | 10 | 5 | – | – |
| 8 | – | – | | | – | 46 | | | 46 | 47 | | | 46 | 48 | | | 47 | 49 | | | 47 | 50 | | | 12 | 7 | 2 | – |
| 7 | – | – | | | – | 44 | | | 44 | 45 | | | 44 | 46 | | | 45 | 47 | | | 45 | 48 | | | 13 | 8 | 3 | – |
| 6 | – | – | | | – | 43 | | | 43 | 44 | | | 43 | 45 | | | 43 | 46 | | | 44 | 47 | | | 15 | 10 | 5 | – |
| 5 | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 44 | | | 43 | 45 | | | 43 | 47 | | | 18 | 13 | 8 | 3 |
| 4 | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 44 | | | 42 | 45 | | | 43 | 47 | | | 20 | 15 | 10 | 5 |
| 3 | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 44 | | | 43 | 46 | | | 43 | 47 | | | 24 | 19 | 14 | 9 |
| 2 | 41 | 41 | | | 41 | 41 | | | 42 | 43 | | | 42 | 44 | | | 43 | 46 | | | 44 | 48 | | | 30 | 25 | 20 | 15 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 8

**AC 15 cm – Subbase CBR-30**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade, S1 = 76 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | Asphalt Placed Directly on Subgrade, S1 = 62 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | 64 | | | | 67 | | | | 71 | | | | 73 | | | | 75 | | | | 77 | | | | – | – | – | – |
| 20 | 61 | | | | 62 | | | | 64 | | | | 67 | | | | 67 | | | | 69 | | | | – | – | – | – |
| 5 | 61 | | | | 61 | | | | 62 | | | | 63 | | | | 64 | | | | 64 | | | | – | – | – | – |
| 10 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 4 | – | – | – |
| 9 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 5 | – | – | – |
| 2 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 6 | 1 | | – |
| 7 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 8 | 3 | – | – |
| 6 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 10 | 5 | – | – |
| 5 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 13 | 8 | 3 | – |
| 4 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 15 | 10 | 5 | – |
| 3 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 19 | 14 | 9 | 4 |
| 2 | – | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 25 | 20 | 15 | 10 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 9

### DBST – Subbase CBR-35

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | | 30 | 32 | 34 | | 33 | 35 | 38 | | 38 | 43 | 48 | | 41 | 46 | 52 | | 43 | 49 | 55 | | 45 | 52 | 59 | 7 | 2 | – | – |
| 20 | | 26 | 28 | 30 | | 28 | 31 | 34 | | 34 | 39 | 43 | | 36 | 42 | 47 | | 39 | 45 | 51 | | 41 | 48 | 55 | 10 | 5 | – | – |
| 15 | | 20 | 22 | 24 | | 23 | 25 | 28 | | 28 | 33 | 38 | | 31 | 36 | 42 | | 33 | 39 | 45 | | 35 | 42 | 49 | 13 | 8 | 3 | – |
| 10 | | 15 | 17 | 19 | | 18 | 21 | 23 | | 24 | 28 | 33 | | 26 | 32 | 37 | | 28 | 34 | 40 | | 31 | 38 | 45 | 19 | 14 | 9 | 4 |
| 9 | | 15 | 16 | 18 | | 17 | 20 | 23 | | 23 | 28 | 32 | | 25 | 31 | 36 | | 27 | 33 | 40 | | 30 | 37 | 44 | 20 | 15 | 10 | 5 |
| 8 | | | | 17 | | 15 | 18 | 21 | | 21 | 26 | 31 | | 24 | 29 | 35 | | 26 | 32 | 38 | | 28 | 35 | 42 | 21 | 16 | 11 | 6 |
| 7 | | | | 15 | | | 17 | 19 | | 20 | 25 | 29 | | 22 | 28 | 33 | | 24 | 30 | 36 | | 27 | 34 | 41 | 23 | 18 | 13 | 8 |
| 6 | | | | | | | 16 | 19 | | 19 | 24 | 29 | | 22 | 27 | 33 | | 24 | 30 | 36 | | 26 | 33 | 40 | 25 | 20 | 15 | 10 |
| 5 | | | | | | | 16 | 19 | | 19 | 24 | 29 | | 22 | 27 | 33 | | 24 | 30 | 36 | | 26 | 33 | 40 | 28 | 23 | 18 | 13 |
| 4 | | | | | | | 16 | 19 | | 19 | 24 | 29 | | 22 | 27 | 33 | | 24 | 30 | 36 | | 26 | 33 | 40 | 30 | 25 | 20 | 15 |
| 3 | | | | 16 | | | 17 | 20 | | 20 | 25 | 30 | | 23 | 28 | 34 | | 25 | 31 | 37 | | 27 | 34 | 41 | 34 | 29 | 24 | 19 |
| 2 | | | 15 | 17 | | 16 | 19 | 21 | | 22 | 26 | 31 | | 24 | 29 | 35 | | 26 | 32 | 38 | | 29 | 36 | 43 | 40 | 35 | 30 | 25 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 10

**AC 5 cm – Subbase CBR-35**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | 50 | 51 | 52 | | 50 | 53 | 56 | | 52 | 59 | 64 | | 56 | 61 | 68 | | 61 | 64 | 72 | | 63 | 67 | 76 | | 2 | – | – | – |
| 20 | – | – | 46 | | – | 46 | 50 | | 40 | 53 | 58 | | 51 | 56 | 63 | | 52 | 57 | 65 | | 54 | 60 | 65 | | 5 | – | – | – |
| 15 | – | – | 44 | | – | 44 | 45 | | 45 | 49 | 54 | | 46 | 50 | 57 | | 47 | 54 | 60 | | 48 | 61 | 64 | | 8 | 3 | – | – |
| 10 | – | – | 38 | | – | 39 | 41 | | 39 | 44 | 49 | | 41 | 46 | 53 | | 42 | 48 | 55 | | 44 | 51 | 59 | | 14 | 9 | 4 | – |
| 9 | – | 37 | 37 | | – | 38 | 40 | | 39 | 43 | 48 | | 40 | 46 | 52 | | 41 | 48 | 55 | | 43 | 49 | 59 | | 15 | 10 | 5 | – |
| 8 | – | 35 | 36 | | – | 36 | 39 | | 37 | 41 | 46 | | 38 | 44 | 51 | | 39 | 46 | 53 | | 41 | 48 | 57 | | 16 | 11 | 6 | 1 |
| 7 | – | 33 | 34 | | 33 | 34 | 38 | | 35 | 40 | 45 | | 37 | 42 | 48 | | 38 | 45 | 52 | | 40 | 47 | 56 | | 18 | 13 | 8 | 3 |
| 6 | – | 32 | 33 | | 32 | 34 | 36 | | 35 | 39 | 44 | | 36 | 42 | 48 | | 37 | 44 | 51 | | 39 | 47 | 56 | | 20 | 15 | 10 | 5 |
| 5 | 31 | 32 | 33 | | 31 | 33 | 36 | | 35 | 39 | 45 | | 36 | 42 | 49 | | 37 | 44 | 51 | | 39 | 47 | 56 | | 23 | 18 | 13 | 8 |
| 4 | 31 | 32 | 33 | | 31 | 34 | 36 | | 34 | 39 | 45 | | 36 | 42 | 48 | | 38 | 44 | 52 | | 39 | 48 | 57 | | 25 | 20 | 15 | 10 |
| 3 | 31 | 32 | 34 | | 32 | 34 | 37 | | 36 | 40 | 46 | | 38 | 43 | 50 | | 38 | 46 | 53 | | 40 | 49 | 58 | | 29 | 24 | 19 | 14 |
| 2 | 32 | 34 | 35 | | 33 | 36 | 39 | | 37 | 42 | 48 | | 39 | 43 | 52 | | 40 | 47 | 56 | | 42 | 51 | 61 | | 35 | 30 | 25 | 20 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 11

**AC 5 cm — Subbase CBR-35**

| SG CBR | Base Course CBR and Thickness | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade, SI = 63 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | 63 | 63 | | | 64 | 64 | | | 67 | 70 | | | 69 | 73 | | | 70 | 76 | | | 72 | 79 | | | – | – | – | – |
| 30 | 60 | 60 | | | 60 | 60 | | | 60 | 62 | | | 61 | 64 | | | 62 | 65 | | | 62 | 67 | | | – | – | – | – |
| 20 | – | – | | | – | 60 | | | 60 | 61 | | | 60 | 61 | | | 60 | 62 | | | 61 | 63 | | | – | – | – | – |
| 15 | – | – | | | – | 55 | | | 55 | 56 | | | 55 | 56 | | | 55 | 57 | | | 56 | 58 | | | 3 | – | – | – |
| 10 | – | – | | | – | 50 | | | 50 | 50 | | | 50 | 51 | | | 50 | 52 | | | 51 | 53 | | | 9 | 4 | – | – |
| 9 | – | – | | | – | 48 | | | 48 | 49 | | | 48 | 50 | | | 49 | 51 | | | 49 | 52 | | | 10 | 5 | – | – |
| 8 | – | 46 | | | – | 46 | | | 46 | 47 | | | 46 | 48 | | | 47 | 50 | | | 47 | 51 | | | 12 | 7 | 2 | – |
| 7 | – | 44 | | | – | 44 | | | 44 | 45 | | | 45 | 46 | | | 45 | 47 | | | 45 | 49 | | | 13 | 8 | 3 | – |
| 6 | – | 43 | | | 43 | 43 | | | 43 | 44 | | | 43 | 45 | | | 44 | 46 | | | 44 | 48 | | | 15 | 10 | 5 | – |
| 5 | – | 42 | | | 42 | 42 | | | 42 | 43 | | | 43 | 45 | | | 43 | 46 | | | 44 | 48 | | | 18 | 13 | 8 | 3 |
| 4 | 42 | 42 | | | 42 | 42 | | | 42 | 43 | | | 42 | 46 | | | 43 | 46 | | | 43 | 48 | | | 20 | 15 | 10 | 5 |
| 3 | 42 | 42 | | | 42 | 42 | | | 42 | 44 | | | 43 | 46 | | | 43 | 47 | | | 44 | 49 | | | 24 | 19 | 14 | 9 |
| 2 | 42 | 42 | | | 42 | 42 | | | 42 | 45 | | | 43 | 47 | | | 43 | 48 | | | 44 | 51 | | | 30 | 25 | 20 | 15 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 12

**AC 15 cm – Subbase CBR-35**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade, SI = 76 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | Asphalt Placed Directly on Subgrade, SI = 62 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | 64 | | | | 67 | | | | 71 | | | | 73 | | | | 75 | | | | 77 | | | | – | – | – | – |
| 20 | 61 | | | | 62 | | | | 64 | | | | 67 | | | | 67 | | | | 69 | | | | – | – | – | – |
| 15 | 61 | | | | 61 | | | | 62 | | | | 63 | | | | 64 | | | | 64 | | | | – | – | – | – |
| 10 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 4 | – | – | – |
| 9 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 5 | – | – | – |
| 8 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 6 | 1 | – | – |
| 7 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 8 | 3 | – | – |
| 6 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 10 | 5 | – | – |
| 5 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 13 | 8 | 3 | – |
| 4 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 15 | 10 | 5 | – |
| 3 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 19 | 14 | 9 | 4 |
| 2 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 25 | 20 | 15 | 10 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

# DESIGN TABLE 13

**Surface Treatment – Subbase CBR-40**

**DBST – Subbase CBR-40**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | 43 | | | | 45 | 46 | 47 | | 49 | 51 | 54 | | 51 | 54 | 58 | | 52 | 56 | 60 | | 53 | 58 | 63 | | – | – | – | – |
| 30 | 31 | 31 | 32 | | 33 | 33 | 35 | | 36 | 39 | 43 | | 38 | 42 | 46 | | 39 | 44 | 49 | | 41 | 46 | 52 | | 7 | 2 | – | – |
| 20 | 28 | 28 | 28 | | 29 | 30 | 31 | | 33 | 36 | 39 | | 35 | 38 | 42 | | 36 | 40 | 45 | | 38 | 43 | 48 | | 10 | 5 | – | – |
| 15 | 23 | 23 | 23 | | 25 | 26 | 26 | | 29 | 31 | 34 | | 30 | 34 | 37 | | 32 | 36 | 40 | | 33 | 38 | 43 | | 13 | 8 | 3 | – |
| 10 | 20 | 20 | 20 | | 22 | 23 | 24 | | 26 | 29 | 32 | | 28 | 31 | 35 | | 29 | 33 | 37 | | 30 | 36 | 41 | | 19 | 14 | 9 | 4 |
| 9 | 20 | 20 | 20 | | 21 | 22 | 23 | | 25 | 28 | 31 | | 27 | 31 | 34 | | 28 | 33 | 37 | | 30 | 35 | 40 | | 20 | 15 | 10 | 5 |
| 8 | 18 | 18 | 18 | | 20 | 21 | 22 | | 24 | 27 | 30 | | 26 | 29 | 33 | | 27 | 31 | 35 | | 28 | 34 | 39 | | 21 | 16 | 11 | 6 |
| 7 | 18 | 18 | 18 | | 19 | 20 | 21 | | 23 | 26 | 29 | | 25 | 28 | 32 | | 26 | 31 | 35 | | 28 | 33 | 38 | | 23 | 18 | 13 | 8 |
| 6 | 17 | 17 | 17 | | 19 | 20 | 21 | | 23 | 26 | 29 | | 25 | 28 | 32 | | 26 | 31 | 34 | | 28 | 33 | 38 | | 25 | 20 | 15 | 10 |
| 5 | 18 | 18 | 18 | | 20 | 21 | 22 | | 24 | 27 | 30 | | 26 | 29 | 33 | | 27 | 31 | 35 | | 29 | 34 | 39 | | 28 | 23 | 18 | 13 |
| 4 | 19 | 19 | 19 | | 21 | 22 | 22 | | 25 | 27 | 30 | | 26 | 30 | 33 | | 28 | 32 | 36 | | 29 | 34 | 39 | | 30 | 25 | 20 | 15 |
| 3 | 21 | 21 | 21 | | 21 | 23 | 24 | | 25 | 27 | 29 | | 28 | 32 | 36 | | 30 | 34 | 38 | | 31 | 36 | 41 | | 34 | 29 | 24 | 19 |
| 2 | 24 | 24 | 24 | | 26 | 27 | 28 | | 30 | 33 | 36 | | 32 | 35 | 39 | | 33 | 37 | 41 | | 34 | 40 | 45 | | 40 | 35 | 30 | 25 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 14

**AC 5 cm — Subbase CBR-40**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | 62 | | | | 64 | 65 | 66 | | 69 | 72 | 77 | | 71 | 76 | 81 | | 72 | 79 | 86 | | 75 | 83 | 90 | | – | – | – | – |
| 30 | 50 | 51 | 52 | | 51 | 53 | 56 | | 54 | 59 | 64 | | 56 | 61 | 68 | | 57 | 64 | 72 | | 59 | 67 | 76 | | 2 | – | – | – |
| 20 | – | – | 46 | | – | 46 | 50 | | 51 | 53 | 58 | | 53 | 56 | 63 | | 54 | 57 | 65 | | 55 | 60 | 65 | | 5 | – | – | – |
| 15 | – | – | 44 | | 44 | 44 | 45 | | 47 | 50 | 54 | | 49 | 52 | 57 | | 50 | 54 | 60 | | 51 | 57 | 64 | | 8 | 3 | – | – |
| 10 | 40 | 40 | 40 | | 41 | 41 | 42 | | 44 | 47 | 50 | | 46 | 50 | 54 | | 47 | 52 | 57 | | 49 | 55 | 61 | | 14 | 9 | 4 | – |
| 9 | 38 | 38 | 38 | | 40 | 41 | 41 | | 44 | 47 | 50 | | 45 | 49 | 53 | | 47 | 51 | 57 | | 48 | 55 | 60 | | 15 | 10 | 5 | – |
| 8 | 37 | 37 | 37 | | 38 | 39 | 40 | | 42 | 45 | 50 | | 44 | 48 | 52 | | 45 | 50 | 55 | | 47 | 53 | 60 | | 16 | 11 | 6 | 1 |
| 7 | 36 | 36 | 36 | | 38 | 38 | 39 | | 41 | 44 | 48 | | 43 | 47 | 51 | | 44 | 50 | 55 | | 47 | 52 | 60 | | 18 | 13 | 8 | 3 |
| 6 | 36 | 36 | 36 | | 37 | 38 | 39 | | 41 | 44 | 48 | | 43 | 47 | 52 | | 45 | 50 | 55 | | 47 | 53 | 60 | | 20 | 15 | 10 | 5 |
| 5 | 37 | 37 | 37 | | 38 | 39 | 40 | | 42 | 46 | 49 | | 44 | 49 | 53 | | 46 | 51 | 57 | | 48 | 54 | 62 | | 23 | 18 | 13 | 8 |
| 4 | 37 | 37 | 37 | | 39 | 40 | 43 | | 43 | 47 | 50 | | 45 | 50 | 55 | | 47 | 52 | 58 | | 49 | 56 | 63 | | 25 | 20 | 15 | 10 |
| 3 | 39 | | 39 | | 41 | 42 | 45 | | 46 | 49 | 53 | | 48 | 52 | 57 | | 49 | 55 | 62 | | 52 | 59 | 67 | | 29 | 24 | 19 | 14 |
| 2 | 43 | 43 | 43 | | 45 | 46 | 49 | | 50 | 54 | 58 | | 52 | 57 | 63 | | 54 | 60 | 67 | | 56 | 64 | 72 | | 35 | 30 | 25 | 20 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 15

### AC 10 cm – Subbase CBR-40

| SG CBR | Base Course CBR and Thickness | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade, SI = 62 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | 63 | 63 | | | 64 | 64 | | | 67 | 70 | | | 69 | 73 | | | 70 | 76 | | | 72 | 79 | | | – | – | – | – |
| 30 | 60 | 60 | | | 60 | 60 | | | 60 | 62 | | | 61 | 64 | | | 62 | 65 | | | 62 | 67 | | | – | – | – | – |
| 20 | – | – | | | – | 60 | | | 60 | 61 | | | 60 | 61 | | | 60 | 62 | | | 61 | 63 | | | – | – | – | – |
| 15 | – | – | | | – | 55 | | | 55 | 56 | | | 55 | 56 | | | 55 | 57 | | | 56 | 59 | | | 3 | – | – | – |
| 10 | 50 | 50 | | | 50 | 50 | | | 50 | 50 | | | 50 | 51 | | | 51 | 53 | | | 51 | 54 | | | 9 | 4 | – | – |
| 9 | 48 | 48 | | | 48 | 48 | | | 49 | 49 | | | 49 | 50 | | | 50 | 52 | | | 50 | 53 | | | 10 | 5 | – | – |
| 8 | 46 | 46 | | | 46 | 46 | | | 47 | 48 | | | 47 | 49 | | | 48 | 50 | | | 49 | 52 | | | 12 | 7 | 2 | – |
| 7 | 44 | 44 | | | 44 | 44 | | | 45 | 46 | | | 46 | 47 | | | 46 | 49 | | | 47 | 51 | | | 13 | 8 | 3 | – |
| 6 | 43 | 43 | | | 43 | 43 | | | 44 | 45 | | | 45 | 47 | | | 45 | 48 | | | 47 | 50 | | | 15 | 10 | 5 | – |
| 5 | 42 | 42 | | | 43 | 43 | | | 44 | 46 | | | 45 | 47 | | | 46 | 49 | | | 47 | 52 | | | 18 | 13 | 8 | 3 |
| 4 | 42 | 42 | | | 43 | 43 | | | 44 | 46 | | | 45 | 48 | | | 46 | 50 | | | 47 | 53 | | | 20 | 15 | 10 | 5 |
| 3 | 43 | 43 | | | 43 | 44 | | | 46 | 48 | | | 47 | 50 | | | 48 | 53 | | | 49 | 55 | | | 24 | 19 | 14 | 9 |
| 2 | 44 | 44 | | | 46 | 46 | | | 48 | 51 | | | 50 | 54 | | | 51 | 57 | | | 53 | 61 | | | 30 | 25 | 20 | 15 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN TABLE 16

**AC 15 cm – Subbase CBR-40**

| SG CBR | Base Course CBR and Thickness (cm) | | | | | | | | | | | | | | | | | | | | | | | | Subbase Thickness (cm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | CBR = 50 | | | | CBR = 60 | | | | CBR = 70 | | | | CBR = 80 | | | | CBR = 90 | | | | CBR = + 100 | | | | Base Thickness | | | |
| | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 | 10 | 15 | 20 | 25 |
| 40 | Asphalt Placed Directly on Subgrade, S1 = 76 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 30 | Asphalt Placed Directly on Subgrade, S1 = 62 | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| | | | | | | | | | | | | | | | | | | | | | | | | | – | – | – | – |
| 40 | 76 | | | | 79 | | | | 85 | | | | 88 | | | | 91 | | | | 94 | | | | – | – | – | – |
| 30 | 64 | | | | 67 | | | | 71 | | | | 73 | | | | 75 | | | | 77 | | | | – | – | – | – |
| 20 | 61 | | | | 62 | | | | 64 | | | | 67 | | | | 67 | | | | 69 | | | | – | – | – | – |
| 15 | 61 | | | | 61 | | | | 62 | | | | 63 | | | | 64 | | | | 64 | | | | – | – | – | – |
| 10 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 4 | – | – | – |
| 9 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 5 | – | – | – |
| 8 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 6 | 1 | – | – |
| 7 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 8 | 3 | – | – |
| 6 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 10 | 5 | – | – |
| 5 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 13 | 8 | 3 | – |
| 4 | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 15 | 10 | 5 | – |
| 3 | 61 | | | | 61 | | | | 61 | | | | 62 | | | | 62 | | | | 62 | | | | 19 | 14 | 9 | 4 |
| 2 | 61 | | | | 61 | | | | 62 | | | | 63 | | | | 63 | | | | 64 | | | | 25 | 20 | 15 | 10 |

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

## DESIGN EXAMPLE 3

Given: Same as Example 1.
Find: Practical design by using the design table.

Solution:
Step 1: Same as Example 1.
Step 2: Same as Example 1.
Step 3: *Determine Pavement Design from Table*

The type of surfacing and the subbase CBR determines the tables to be used. For a DBST surfacing and a subbase material with a CBR of 30 use Design Table 5. From Table 5, locate the subgrade CBR in the first column on the left, move to the right to the columns captioned by a CBR of 80 for the base course. The structural index is indicated for various applicable thicknesses of base course layers. Locate a structural index equal to or slightly greater than 30. Move up the column to determine the base course thickness (20 cm), move to the right to the columns captioned as subbase thickness, stopping at the previously determined base course thickness (20 cm). Move down the column to a subgrade CBR of 9. The number (10) indicates a 10 cm thickness of subbase which is based on the minimum thickness of cover for the subgrade CBR value.
Design: DBST, 20 cm base and 10 cm subbase as in Example 1.

## DESIGN EXAMPLE 4

Given: Same as Example 2.
Find: Practical design by using design tables
Solution:
Step 1: Same as Example 2.
Step 2: Same as Example 2.
Step 3: *Determine Pavement Design from Table*

Design Table 7, is used for a surfacing of 10 cm of asphalt concrete and a subbase material with a CBR of 30. Follow the procedure outlined in Step 3 of Example 3.

Design: 10 cm AC, 10 cm base and 10 cm subbase.

Examples 5 and 6 are similar to Examples 3 and 4 respectively but the design traffic is different and special problems are introduced which require the joint use of design equations and design tables.

## DESIGN EXAMPLE 5

Design traffic period 350,000 SAL (both directions).

Step 1:
From Figure 3.1 required SI = 38 for design traffic of 350,000 SAL and coefficient of variation of 35 percent.

Step 2:
From Table 3.2 the recommended surfacing for a base course CBR of 80 and design traffic period of 350,000 SAL (both directions) is a DBST.

Step 3:
As shown in Design Table 5. the maximum SI is 36 for minimum thickness design. An addition SI of 2 is required to satisfy the requirements in Step 1. A decision is made to increase the SI of the section by increasing the subbase thickness.

Step 4:
An increase in subbase thickness of 10 cm greater than the thickness required by minimum thickness design (5 cm) will increase the structural index by 2 (0.205 x 10 cm = 2.05).

Check with structural equation.

$$SI = (1.102)(25)+(.205)(15)+(0)(10)+(.183)(40) = = 37{,}95 \sim 38$$

Design: DBST, 25 cm base and 15 cm subbase.

## DESIGN EXAMPLE 6

Design traffic period of 2,100,000 SAL (both directions).

Step 1:
From Figure 3.1, required structural index = 53 for a design traffic of 2,100,000 SAL and a coefficient of variation of 35 percent.

Step 2:
From Table 3.2, the recommended surfacing for a base course CBR of 80 and designed traffic period of 2,100,000 SAL (both directions) is 10 cm of asphalt concrete.

Step 3:
As shown in Design Table 7, the maximum SI is 50 for minimum thickness design. An addition SI of 3 is required to satisfy the requirements in Step 1. A decision is made to increase the SI of the section by increasing the subbase thickness.

Step 4.
Required SI (53) – Subgrade SI (7) = 46.

Step 5:
From Figure 3.6, the required structural index of the unbound soil layers between the depth of 10 and 50 cm is 24. An additional thickness of subbase of 20 cm provides an increase in the structural index of 4.1 (.205 x 20 cm). The total SI of the unbound soil layers between 10 and 50 cm would be 21.66 (rounding off) 22. From Figure 3.6 the combined structural number would be 45 $\sim$ 46.

Design: 10 cm AC, 15 cm base and 25 cm subbase.

## DESIGN LIMITATIONS

Pavement performance is not governed by total axle loads alone but also by the number of loads applied within a given time period. Several maximum intensities of standard axle loads are recommended. These are provided in Figure 3.8. If higher maximum intensities are required in design, consideration should be given to adding a second traffic lane.

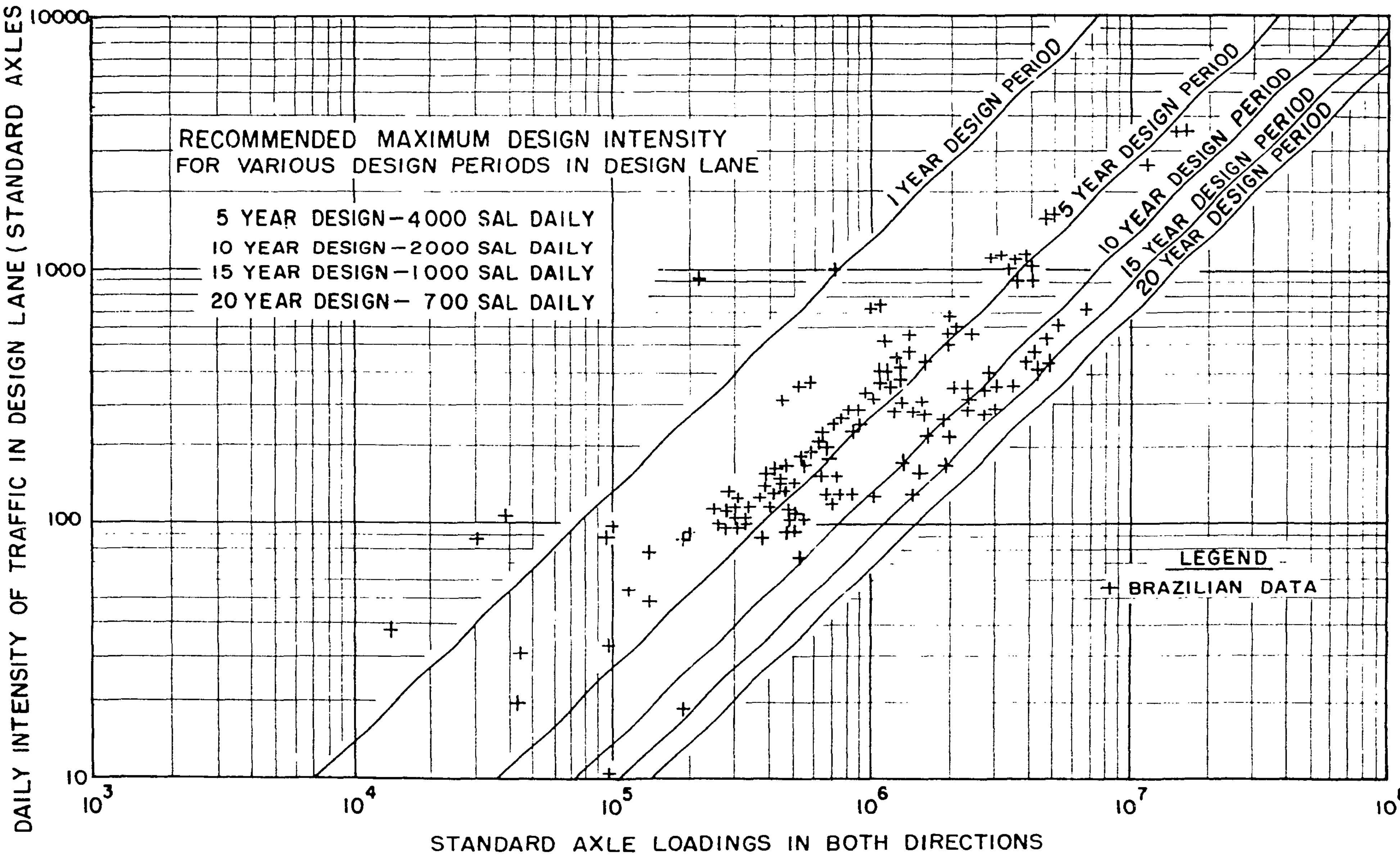



FIGURE 3.8 – DAILY INTENSITY OF STANDARD AXLE LOADINGS VS TOTAL STANDARD AXLE LOADINGS

## PAVEMENT OVERLAY CONCEPTS

The percent reduction in deflection for a given overlay thickness is dependent upon whether the pavement receiving the overlay has a surface treatment or an asphalt concrete surface. The curves in Figures 3.9 and 3.10 are recommended for use in designing pavement overlays. In any major overlay program where the combined thickness of asphalt after overlay is equal to or greater than 15 cm, it is advisable to evaluate the strength of the existing pavement prior to the final decision of overlay thickness.

## ASPHALT EQUIVALENTS

The Asphalt Institute (1969) recommends the use of a "Substitution Ratio" of 2 for high-quality bases where CBR is 100 or more and 2.7 for low-quality bases where CBR is more than 20.

Asphalt equivalents based on Table 3.1 coefficients are shown in Figure 3.12. These provide asphalt equivalents that are in the same range as those ratios recommended by the Asphalt Institute.

Asphalt equivalents are not constant for all design schemes. The Asphalt Institute states:

> "Analytical studies of extensive data available from road tests, laboratory experiments, and theoretical analyses show that there is no simple, constant factor for converting a given thickness of asphalt layer into a thickness of untreated granular base that will provide equivalent load-spreading capacity. This conversion is a variable that depends principally on the amount of traffic, the magnitude of the wheel loads, and the strength properties of the untreated granular base and subgrade".

However, the use of material equivalents may be appropriate in the planning and design stages. For example, equivalent pavement designs Table 3.3 are shown for three thicknesses of asphalt surfacing for a given design period and subgrade CBR. The equivalents are in terms of the structural index which can be readily computed for various traffic and subgrade conditions. A cost analysis can be made to determine the most economical section. The thickness of the base and subbase would depend upon the availability of materials, whereas the minimum thickness of asphalt is governed by the base course CBR and traffic.

## DESIGN OF UNPAVED ROADS

The deterioration of unpaved roads results from the loss of surface material; formation of corrugations and potholes; softening of the surface during rain; and the loss of fines as dust during the dry season. It would be impossible to design an unpaved road that required no maintenance. It is better to correlate the performance of unpaved roads to the maintenance required rather than to a thickness-CBR relationship.

Maintenance can be held to a minimum by the proper selection of surface materials and by controlling the deflection by selecting an adequate thickness of material based upon the strength of the subgrade. One observation that was made in the study of unpaved roads in Africa is that very little attention has been given to the thickness of the section above the subgrade. In many cases thickness requirements are determined from maintenance records of the loss of surface material during a given time period. The thickness then simply determines the interval between resurfacing. The difficulty in maintaining untreated gravel roads is related to the subgrade soil and the thickness of select material above the subgrade. This indicates that a structural thickness requirement is appropriate in designing unpaved roads. The minimum thicknesses for various subgrade CBR values shown in Figure 3.4 are recommended for design of unpaved roads. It is recommended that the CBR of the select material be 40 or above.

### TABLE 3.3

**Equivalent Pavement Designs**

| Design | Surface Thickness cm | Structural Index of Base |
|---|---|---|
| 1 | 25 | 0 |
| 2 | 15 | 22 |
| 3 | 10 | 28 |

Subgrade CBR 15; Design traffic = 3.75 x $10^6$ SAL.

Material desirable for the surface course of an unpaved road will not always be suitable as a base course under a paved road. Experience in Africa indicates that material with far higher Atterberg limits than normally acceptable for base courses under paved roads perform better as surface courses of unpaved roads.

If stage construction is desirable, the surface material should comply with specifications for the base course of a paved road. Two alternatives are available. One is to use the more suitable surface material and cover it with an adequate thickness of base material prior to adding an asphalt concrete surface. The other is to stabilize the surface material with an additive prior to applying the asphalt concrete surfacing. In Nigeria, for example, it has been the practice to remove the top six inches and replace it with a more suitable material or stabilize the top six inches with lime prior to placing an asphalt concrete surface.

Experience in other countries has shown that 10-15 cm of untreated surface material will last four to five years for traffic in the range of 100 vehicles per day. The loss of material can be taken into consideration in the design where the loss would be replaced with suitable base material prior to paving. In this case the design grade of the unpaved road would be the design grade of the base course in the second stage. Therefore, drainage structures such as side ditches can be initially incorporated into the first stage without major modification during the second phase.

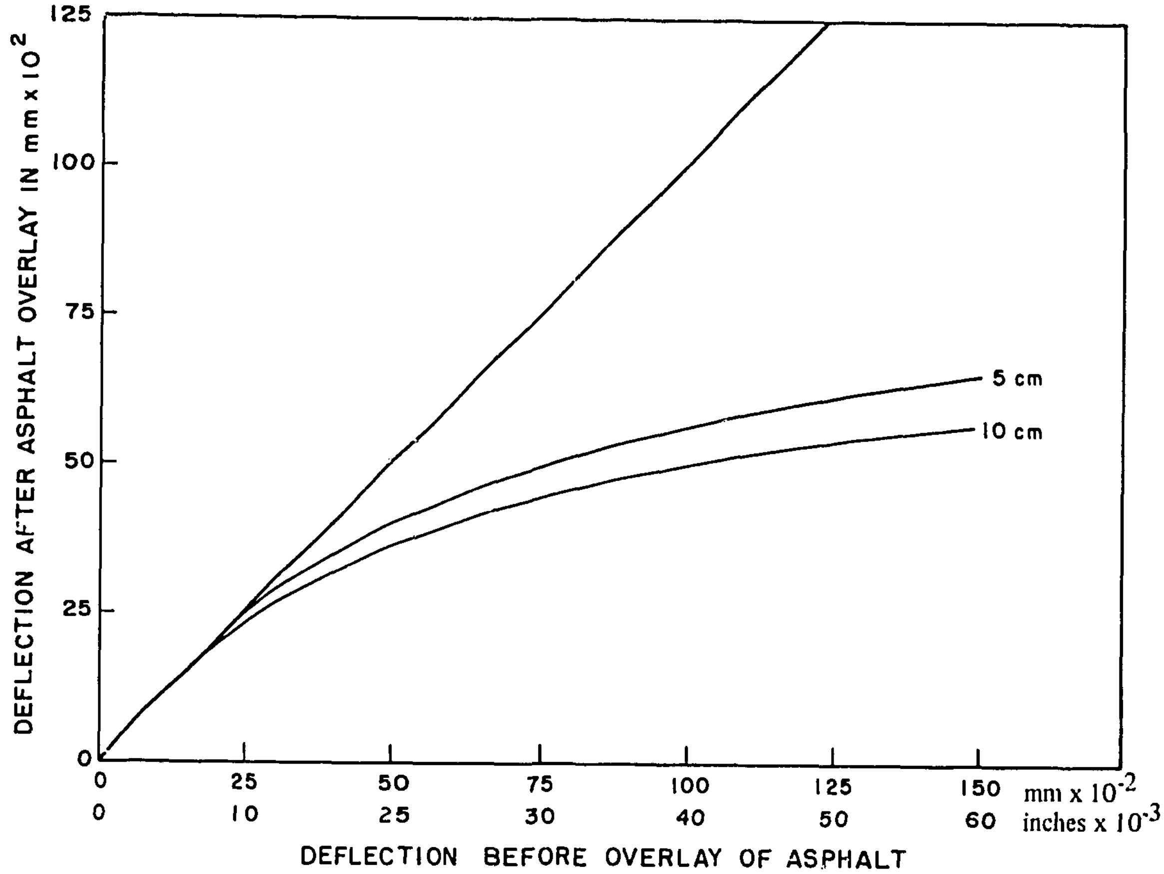



FIGURE 3.9 – REDUCTION IN DEFLECTION ACHIEVED BY OVERLAYS OF DIFFERENT THICKNESSES PLACED ON AN EXISTING SURFACE TREATMENT

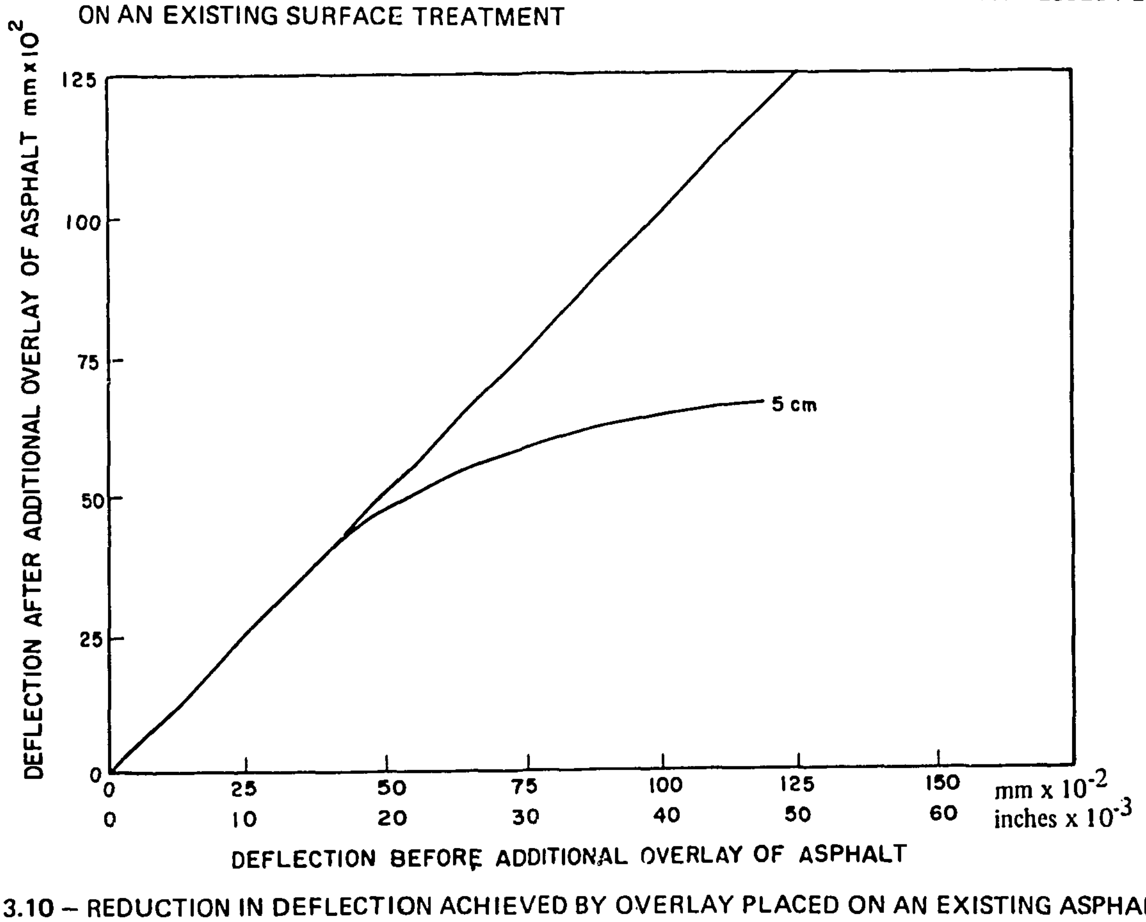



FIGURE 3.10 – REDUCTION IN DEFLECTION ACHIEVED BY OVERLAY PLACED ON AN EXISTING ASPHALT CONCRETE SURFACE

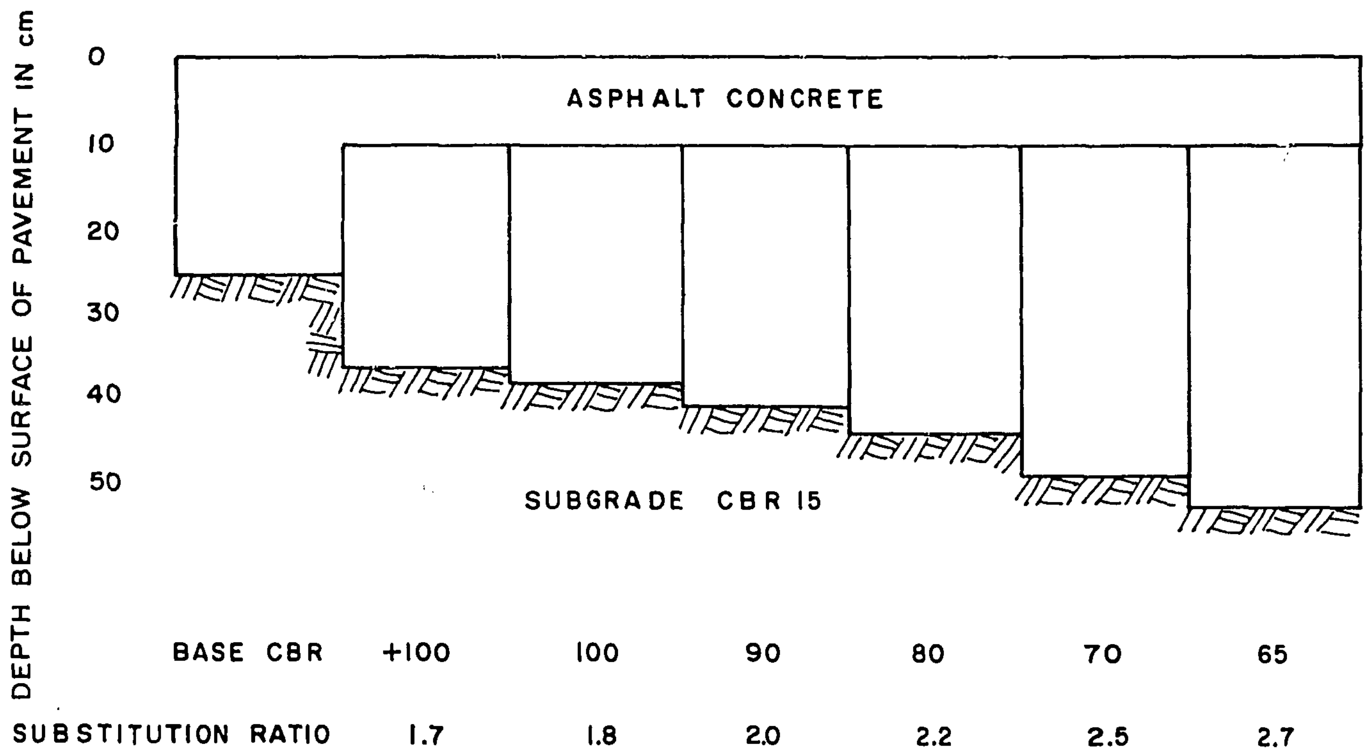



FIGURE 3.11 – SUBSTITUTION RATIOS FOR VARIOUS BASE COURSE MATERIALS

## APPENDIX TO CHAPTER 3
*DETERMINATION OF COEFFICIENT OF VARIATION*

The coefficient of variation is a significant factor in the pavement design procedure described in Chapter 3. The coefficient should be determined for the study area prior to using the design curves. It is determined from pavement deflection measurements of several existing roads in the vicinity of a proposed project. It reflects the variation in the quality of construction materials and the consistency in applying construction techniques in that region. The following is a detail description of deflection equipment, measurement procedures and derivation of the coefficient of variation.

### Equipment

#### DEFLECTION BEAMS:

Deflection beams are available in two designs; (1) with a lever arm ratio of two-to-one and (2) with a lever arm of four-to-one. Both designs have an eight foot probe beam which is supported at its fulcrum by a ball bearing pivot.

The beams are equipped with dial indicators that measure the movement of the probe beam. The dial indicators are graduated in inches or in millimeters. The dial indicators graduated in inches are in 0.001 inch divisions. One dial division represents .002 inch deflection when using beams with lever arm ratios of 2:1 and .004 inch deflection when using beams with a lever arm ratio of 4:1. Compensated dial indicators are available that provide a direct reading without having to compute a leverage factor.

A buzzer unit is mounted to the main instrument section of the beam which eliminates or reduces stickiness and friction in operating parts of the beam and the dial indicator. The beams are equipped with probe beam locks to prevent damage to the micrometer extension arms when moving the beams from one location to another.

#### LOAD VEHICLES:

The load is applied to the pavement by a two axle truck with the rear axle equipped with dual wheel assemblies. The weight of the truck is adjusted by adding sufficient ballast to produce an 8182 kg. rear axle loading. Rear tire pressures are maintained between 5.9 and 6.3 $kg/cm^2$. Some type of trucks require shims between the rear wheels of each assembly to provide proper clearance for operation of beams. Figure A3.1 shows a typical load-vehicle used in the Brazilian study. The short wheel base truck, as shown, requires less total weight than longer wheel base trucks to obtain the 8182 kg. rear axle loading.

As a safety precaution the load vehicle should be equipped with a reverse movement alarm system.

#### PROCEDURE

One of two procedures are commonly used in determining the deflection characteristics of flexible pavements. In one method the maximum deflection is determined as

FIGURE A3.1 – TYPICAL LOAD VEHICLE USED IN THE BRAZILIAN STUDY the load-vehicle moves over the test location and the rebound and residual deflection measured as the load-vehicle moves off the test location. The test vehicle is placed behind the test location. The beams are positioned behind the rear axle with the beam arm between the tires of the two dual wheel assemblies. The beam is moved forward until the toe of the probe beam is on the test location. The load vehicle then moves forward. The maximum deflection is recorded as the rear tires passes over the test location. The rebound deflection is measured when the load vehicle moves off the test location a distance of approximately 6 meters. The residual deflection is considered as the difference between the maximum deflection and the rebound deflection. The "line-up" procedure leaves some doubt in the results obtained due to the possibility of the front supports located within the deflection basin at the beginning of the test (Figure A3.2). Another procedure sometimes used is to back the load-vehicle onto the test location with the beams in place on the pavement. This procedure eliminates the possibility of the front supports being located within the deflection basin at the beginning of the test. The procedure requires very experienced drivers otherwise the beams may be damaged. The interpretation of the measured residual deflection often can be misleading.

Due to the uncertainties of the procedure described above a procedure of measuring only the rebound deflection, is recommended. In this procedure the load-vehicle is placed over the test location. Two deflection beams are positioned behind the rear axle with each beam toe between the tires and in the center of one of the two dual wheel assemblies. An initial reading is recorded. After the initial zero reading is recorded, the truck is advanced forward a distance of 6 m and a second reading recorded. The difference between the two readings is the uncorrected rebound deflection.

**TEMPERATURE CORRECTION:**

The deflection of an asphaltic concrete pavement is dependent upon the temperature of the pavement as well as the magnitude of the imposed load. Pavement deflections are corrected to a standard temperature to provide a basis of comparison between deflection data obtained elsewhere. The standard temperature normally used is 21°C. Although the usual pavement temperature in many tropical countries is greater it is recommended that pavement deflections be corrected to 21°C to facilitate correlation with research conducted in other countries. The temperature correction procedure of the Asphalt Institute (USA) is recommended. The temperature correction factors are shown in Figure A3.3. Curve *A* includes data primarily for pavements over granular base materials and represents strong support to the asphalt layer (less than 10 cm). Curve *B* represents data from thick asphalt pavements (10 cm or more) laid directly on weak subgrades. The Asphalt Institute recommends the use of Curve *A* in most circumstances. The temperature adjustment factors given in Figure for Curve *A* are presented in tabular form in Table A3.1. The use of the temperature adjustment factor is shown in equation A3.1.

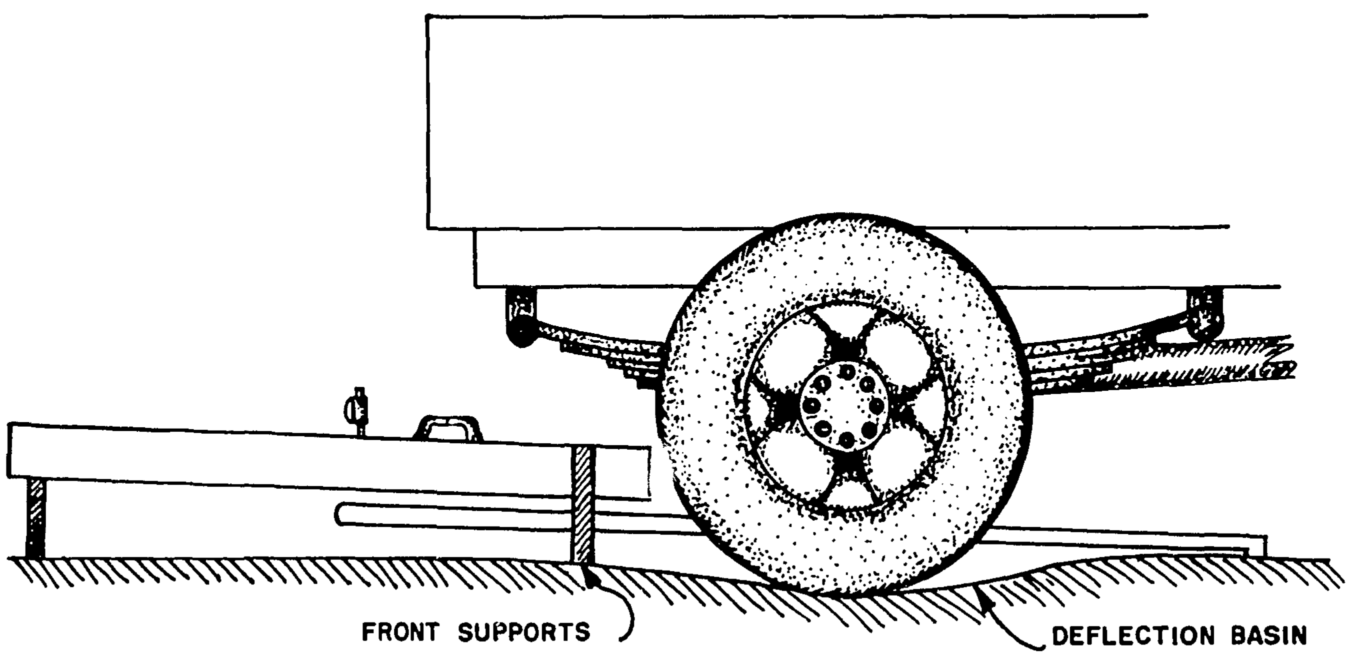



FIGURE A3.2 – "LINE-UP" PROCEDURE WITH POSSIBILITY OF FRONT SUPPORTS LOCATED WITHIN THE DEFLECTION BASIN

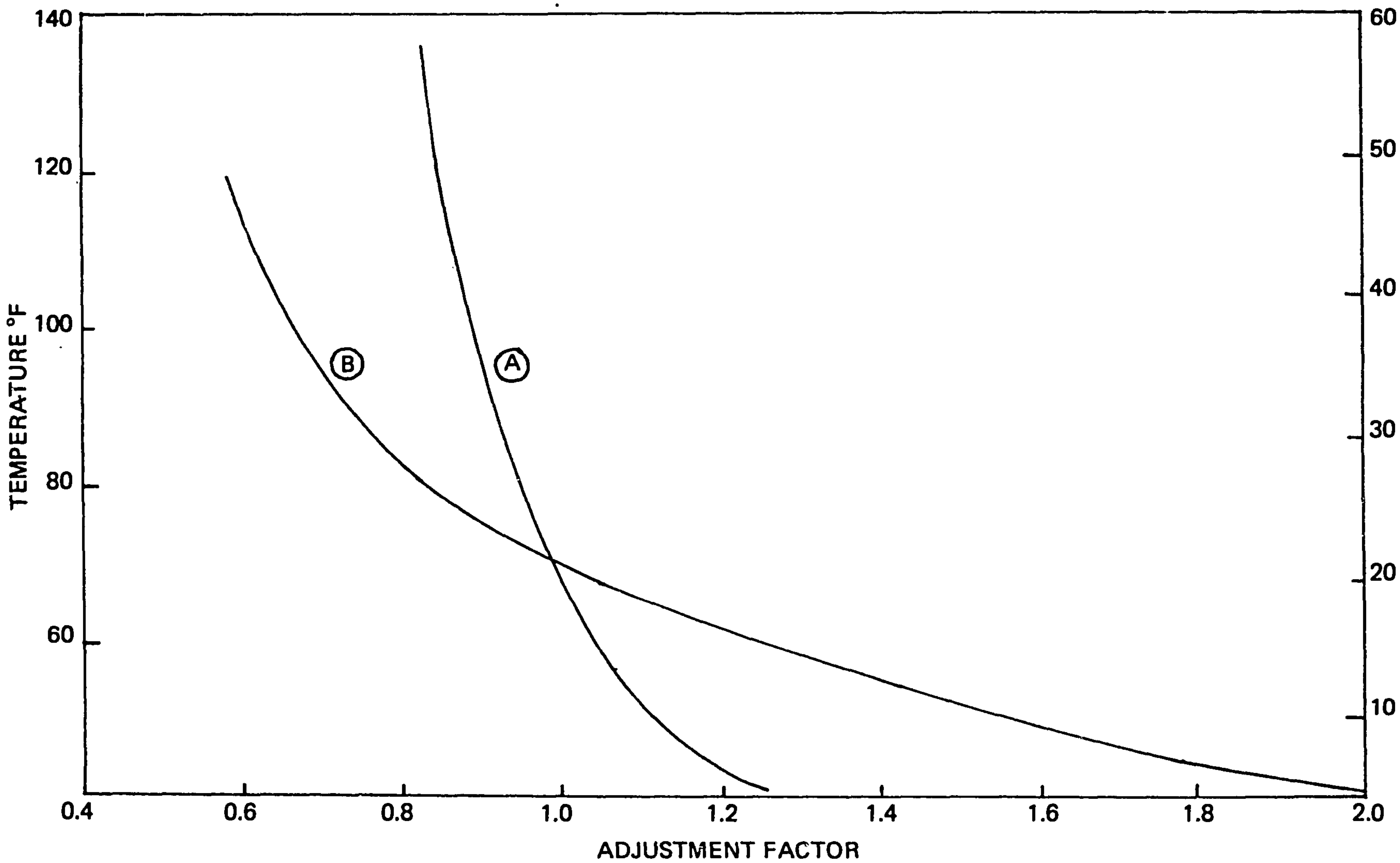



FIGURE A3.3 – ADJUSTMENT FACTOR CURVES FOR USE IN CORRECTING BEAM DEFLECTION TO A STANDARD TEMPERATURE OF 21°C

## Computations

The rebound deflection is the difference between the intial reading on the dial indicator and the final reading. The difference is multiplied by the beam constant to obtain the rebound deflection.

Rebound deflection = $(R_f - R_o)\,KT_c$ ...... (A3.1)

Where:

$R_f$ = the final reading on the micrometer

$R_o$ = the initial reading on the micrometer

K = the beam constant, either 2 or 4 depending upon beam design.

$T_c$ = temperature correction factor

### TABLE A3.1

**Adjustement Factors for Use in Correcting Beam Deflections to a Standard Temperature of 21°C. (After Kingham 1969)**

**Temperature Adjustement Factors**

| Temp. °C | Adjustement Factor | +1° | +2° | +3° | +4° |
|---|---|---|---|---|---|
| 5 | 1.22 | 1.202 | 1.184 | 1.166 | 1.148 |
| 10 | 1.13 | 1.114 | 1.098 | 1.082 | 1.066 |
| 15 | 1.05 | 1.042 | 1.034 | 1.026 | 1.018 |
| 20 | 1.01 | 1.00 | 0.993 | 0.985 | 0.978 |
| 25 | 0.97 | 0.962 | 0.954 | 0.946 | 0.938 |
| 30 | 0.93 | 0.923 | 0.917 | 0.910 | 0.904 |
| 35 | 0.898 | 0.893 | 0.888 | 0.883 | 0.878 |
| 40 | 0.873 | 0.869 | 0.866 | 0.863 | 0.860 |
| 45 | 0.857 | 0.854 | 0.850 | 0.847 | 0.843 |
| 50 | 0.840 | 0.838 | 0.836 | 0.834 | 0.833 |
| 55 | 0.831 | 0.829 | 0.827 | 0.824 | 0.822 |
| 60 | 0.820 | | | | |

## Coefficient of Variation

The coefficient of variation is defined as the standard deviation ($\sigma$) divided by the average deflection of a given set of measurements. It represents the degree of variation of the pavement strength that reflects the construction and control practices that were employed in the construction of the pavement section. The coefficient of variation is a significant factor in pavement design. In determining the coefficient of variation to be used in the design procedure outlined in Chapter 3 the deflection measurements are taken at 30 meter intervals along a 150 meter section of pavement.

Calculation of the Coefficient of Variation:

Given; The following pavement deflection measurements were obtained at six 30 meter stations along a given 150 meter section of highway for the outside wheel path.
(.075 – .083 – .088 – .063 – .095 – .073) measurements were obtained in cm.

Find; The coefficient of variation of the given deflection measurements.
The coefficient of variation is defined as the standard deviation ($\sigma$) divided by the average deflection ($\bar{X}$).

Step 1 Determine the average deflection ($\bar{X}$)

Average Deflection =

$$\frac{.075 + .083 + .088 + .063 + .095 + .073}{6}$$

$$= \frac{0.477}{6} = 0.0795 \text{ cm}$$

Step 2 Determine the standard deviation ($\sigma$)
When the standard deviation ($\sigma$) is expressed by the following equation:

$$\sigma = \sqrt{\frac{\Sigma (X_i - \bar{X})^2}{N - 1}}$$

Where $X_i$ is an individual measurement;
$\bar{X}$ is the average of all measurements and
N is the number of measurements.

| $X_i$ | $(X_i - \bar{X})$ | $(X_i - \bar{X})^2$ |
|---|---|---|
| .075 | - 0.0045 | 0.00002025 |
| .083 | 0.0035 | 0.00001225 |
| .088 | 0.0085 | 0.00007225 |
| .063 | - 0.0165 | 0.00027225 |
| .095 | 0.0155 | 0.00024025 |
| .073 | - 0.0065 | 0.00004225 |

The sum ($\Sigma$) of $(X_i - \bar{X})^2$ is 0.0006595

$$\sigma = \sqrt{\frac{0.0006595}{5}} = \sqrt{.0001319} = \pm 0.01148$$

Step 3; Coefficient of variation $= \frac{\sigma}{\bar{X}}$ (100)

$$= \frac{0.01148}{0.0795} \times 100 = 14.4\%$$

The designer would use the 15 percent curve in Figure 3.1.

The purpose of stabilizing a soil is to alter its physical properties, increase its strength and increase its durability in order to provide a satisfactory foundation material. The admixtures most commonly used today in road construction throughout the world are cement, lime, asphalt and sand. Cement appears to be most common additive used to date in Africa and South America Lime has been used as well, but not as commonly as cement. The use of asphalt has been limited to lateritic soils of a sandy nature. Stabilization by admixture of sand has been investigated and successfully used in locations where sand is readily available such as in coastal regions.

There has been a certain amount of controversy between those advocating stabilization with cement and those advocating stabilization with lime. This is true in temperate climates involving both residual and transported soils. It is also true in tropical climates where the soils are mostly residual. Those that believe only in cement stabilization state that there is little in tropical residual soils with which the lime can react, and unless there are clay-lime reactions, there is no benefit in adding lime. Those who believe in lime stabilization state that most clayey materials are more effectively stabilized with lime than cement. There are also those who believe in a combination of both lime and cement for tropical red soils. Studies have found many highly plastic soils which do not increase in strength upon the addition of lime but which always show a reduction in plasticity. The soils become more friable and easier to mix creating ideal soils for cement stabilization.

Under field conditions it is very hard and expensive to dry and break down a plastic soil mechanically. In a high rainfall area it is impossible. It has been recommended that cement be used to stabilize those soils having a plasticity index of 15 or less and a percentage passing the No. 200 sieve of no more than 25 percent. Soils with these physical characteristics can usually be mixed with most available construction equipment. It is often practical to cement stabilize even heavy clays after pre-conditioning with hydrated lime.

Lime is almost always more practical to use when the plasticity index is greater than 15 and the percent passing the No. 200 sieve is greater than 25 percent. However, lime has little effect in highly organic soils, or in soils with little reactive clay content. If the clay material is mostly kaolinitic the reaction with lime may be very slow and if the clay material is mostly iron minerals, lime may not be reactive. Cement should be used in these cases. However, soils with a plasticity index greater than 15 tend to "ball up" when a pulvermixer is used and lime may improve the workability. To insure that a soil will be properly blended, at least 60 percent of the soil binder must pass a No. 4 sieve after final mixing. This insures that the cement and the soil has formed a homogeneous mixture. The mixture must be compacted within a two hour period after the cement is added. This is done so that the mixture does not set before compaction. If the mixture does set and is disturbed after bonding begins there will be an unnecessary loss in strength.

Cementing reactions that take place between the lime and soil proceed slowly and, therefore, the final mixing can be delayed for considerable time. If the soil is very plastic, the lime can be mixed very expeditiously at first and compacted only enough to protect the lime from carbonation and subgrade from becoming saturated in case of a heavy rain until the lime has time to react with the clays. At the time of final mixing the pH of the soil should be determined and if needed enough lime added to bring the pH to 12.4 before compaction

If the material to be stabilized has a plasticity index greater than 15 and laboratory strength tests show that the lime treated soil will not give a bearing value which will meet the design standards, lime and cement should be used in combination. The soil or base material should be mixed with enough lime to give a pH of 11.0±0.2. The lime should be added first to disaggregate the soil by reducing the plasticity. At the time of final mixing, the cement should be added. The mixing and compaction of the cement and lime treated soil should be accomplished in the two hour time limit.

The general applicability of all stabilization methods is given in Table 4.1. The general ranges for cement and lime are given along with those for bitumen, polymeric-organic materials and mechanical stabilization. Thermal stabilization is a technique applied mainly to expansive clays, usually for localized foundation problems.

A broad comparison of the stabilization techniques available in highway construction is given in Table 4.2. This may be used as a general guide, but other factors should be considered for tropical soils. The reactivity of the soils must be checked with each project.

Generally the sandy soils or those with high contents of iron will be most effectively stabilized with cement. This would include most of the luvisols and arenosols of South America, the ferruginous soils and sandy ferrallitic soils of Africa and the comparable soils of Southeast Asia. The acrisols and ferralsols of South America and the ferrallitic soils and ferrisols of Africa and the comparable soils of Southeast Asia will usually be more effectively stabilized with lime. With tropical soils a few chemical and mineralogical analyses are always advisable.

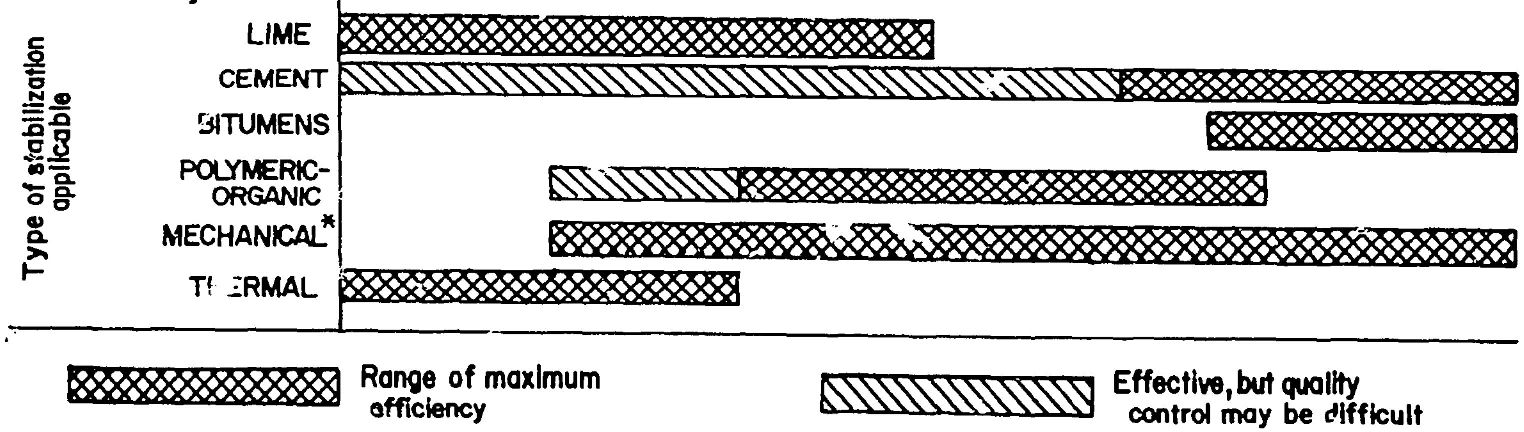
Lime needs clay minerals to react with, and kaolinite-lime reactions may be very slow. Lime is reactive with the aluminum minerals which are common in the ferralsols and acrisols. If none of these materials are present, lime is of little value outside possibly to help dry the soil during construction. These minerals or materials are commonly present in ferralsols and acrisols of South America and the ferrallitic soils of Africa and the comparable soils of Southeast Asia.

The results of the asphalt stabilization tests in Africa and Southeast Asia indicate that asphalt is an effective and practical stabilizer in relatively clean sands.

The Tropical Design Procedure can be applied if the stabilized materials do not exceed reasonable limits. Structural coefficients are given in Chapter 3 for CBR values greater than 100; however, there is no distinction made between these higher CBR values, because CBR values over 100 are meaningless. In using the design procedure outlined in Chapter 3 appropriate maximum CBR values for the

TABLE 4.1
Applicability of Stabilization Methods
(After Ingles and Metcalf 1972)

| Designation | Fine clays | Coarse clays | Fine silts | Coarse silts | Fine sands | Coarse sands |
|---|---|---|---|---|---|---|
| SOIL Particle size (mm) | <.0006 | .0006-.002 | .002-.01 | .01-.06 | .06-.4 | .4-2.0 |
| SOIL Volume stability | V. poor | Fair | Fai | Good | V. good | V. good |




* i.e. improvement of soil grading by mixing-in gravels, sands or clays as appropriate

### TABLE 4.2
**Broad Comparison of Stabilization Techniques (After Ingles and Metcalf, 1972)**

| In situ Material | Pavement Thickness | Mechanical | Cement | Lime | Bitumen |
|---|---|---|---|---|---|
| Natural gravel | Min. 10 cm (4 in.) | Fines may be needed to prevent ravelling | Probably not necessary except if plastic. 2-4 per cent | Not necessary except if plastic. 2-4 per cent | Not necessary unless lacking fines 3 per cent residual bitumen use medium or-slow curing cut-back or emulsion |
| Clean sand | Min. 10 cm (4 in.) | Coarse material for strength and fines to prevent ravelling | Unsuitable: produces brittle material | Unsuitable: no reaction | Most suitable 3 per cent residual bitumen Rapid curing cut-backs may be used Add 2 per cent lime for wet sand. |
| Clayey sand loam | 15-25 cm (6-10 in.) | Coarse material for strength and seal adhesion | 4-8 per cent | May be suitable depending on clay content | May be suitable 3-4 per cent |
| Sandy clay | 15-35 cm (6-14 in.) | Not usually suitable | 4-12 per cent | 4-8 per cent depending on clay content | May be suitable for light traffic 3-4 per cent |
| Heavy clay | 25 cm (10 in.) | Unsuitable | Unsuitable Mixing may be assisted by pre-treatment with 2 per cent lime then 8-15 per cent cement | Most suitable 4-8 percent depending on clay content | Not usually suitable |

design of stabilized layers would be the maximum values shown in Table 3.1 for the various depth intervals. Stabilization can be used to improve a material from a CBR of 50 to a CBR of 80. Or the designer can use stabilization as a means of improving materials that are otherwise unsatisfactory. As an example, a material which exhibits a CBR of 30 can be used as a base course material if, through stabilization, the CBR can be increased to over 50. Increased strengths with CBR values in the range of 100 and over should be evaluated in terms of unconfined compressive strengths.

The results of such tests can then be applied in a design procedure which takes into consideration the difference between the load-spreading characteristics of stabilized layers and unbound layers. The design procedure outlined in Chapter 3 can accommodate stabilized base course on the basis of unconfined compressive strength, but, these coefficients are conservative values.

One of the more advanced flexible pavement design procedures which incorporates stabilization of the component layers is founded on the basic concept that a reduction in the overall required thickness of pavement is possible due to its acquiring higher flexural strength with stabilization. The original thickness is determined assuming the structural layers to be unbound material. Stabilization reduces the required thickness. The amount of reduction depends upon the tensile strength of the stabilized layer and the original required thickness.

## INTRODUCTION

Tropical black clays, often called black cotton soils, are the major problem soils of the tropics. They are an unsuitable material to use in highway or air-field construction because they contain a large percentage of plastic clay and swell or expand as they absorb water.

The expansiveness of tropical black clays is a notorious characteristic of this soil. Swelling clays and clay shales are found in many parts of the world in temperate as well as tropical regions. The causes are usually high content of montmorillonite in the clay fraction.

In many areas where these soils occur, there are no suitable natural gravels or aggregates. Most tropical black clay deposits cover such large areas that avoiding or by-passing them is not feasible. Few roads constructed through these soils have proved satisfactory. Some, in Africa and Southeast Asia, have failed completely.

The physical properties of the tropical black clays are distinctly different from those of arenosols and luvisols but there can be some overlap in properties with ferralsols, nitosols and ferrisols.

The gradation of the soils varies considerably. There is little or no gravel size in most soils and those which show appreciable percentages, such as those in India and Ethiopia, may include rock fragments not yet decomposed. The amount of sand varies from 0 to about 40 percent and the silt varies from 0 to about 50 percent. The clay size generally ranges from 20 to 100 percent.

Atterberg limits of the black clays are high. Liquid limits are usually over 50 percent; plasticity indexes range between 20 and 60. Shrinkage limits vary from 7 to 28 percent. The AASHO classification for the black clays is A-7-5 or A-7-6. Organic contents are usually faily low. CBR values are very low when reported after the normal 4 day soaking period.

## DESIGN CONSIDERATIONS FOR ROADS OVER TROPICAL BLACK CLAYS

### General

Three problems are associated with black clay soils in highway construction. First, the material is predominantly clay, which is potentially expansive and may have a very low bearing capacity. Secondly, in the larger residual deposits there is a lack of a suitable natural aggregate where these soils occur, although alluvial deposits generally include layers of sandy materials within the profile. Finally, road construction alters the moisture pattern in the subgrade soils in all but very arid or very humid climates. Surface evaporation is prevented or reduced and the moisture content of the subgrade soil invariably rises. In a montmorillonitic soil, this causes swelling. The swelling, in turn, reduces the bearing capacity. A pavement structure that is adequately designed for the as-constructed subgrade density may fail as the density is reduced through expansion.

There are no inexpensive means of utilizing these soils satisfactorily. No "easy way" has been found for dealing with these or any other expansive soils that occur throughout the world. The obvious solution is to avoid them wherever possible, or if shallow, to excavate and waste the material. This is rarely possible in large deposits.

A very great thickness of base and surface is usually required, as dictated by low CBR values even in the absence of swelling. Economizing on the thickness of the pavement structure invariably results in greater expenditures later. This has been the experience all to often in many countries. Inadequate designs have resulted in extensive failures; large sections have had to be rebuilt within one or two years.

### Recent Design Practices

Recent work on expansive soils, shows that swelling of expansive clays can be reduced by: (1) accepting lower densities and higher moisture contents; (2) allowing some initial expansion to occur; (3) by surcharging; and (4) preventing changes in moisture content.

There are four major approaches to pavement design practices over expansive clays. They are:

1. Avoid expansive clays by either realignment or undercutting and backfilling.
2. Reduce expansive characteristics.
3. Confine expansive clays under embankments.
4. Minimize moisture changes in the expansive clay after paving.

The first, avoiding the clays, is not often possible in many of the locations where these soils occur. However, a routine swell test, such as the PVC, would at least provide a basis for classifying the expansive sections.

Expansive characteristics can be reduced by compaction control and by stabilization. Compacting to lower densities reduces swell and to compact wet of optimum further reduces swell. The method of compaction that produces greater dispersion also causes less swell.

Ponding water over or pre-wetting the subgrade soil has been successful in the U.S. and South Africa. Pre-wetting expansive clays may provide a solution to prevent localized heaving around culverts. The loss of strength and highly plastic condition of the soil after ponding may require lime treatment of the upper 15-20 cm to provide a working platform.

Treatments of expansive subgrades include clearing the vegetation and allowing the soil to "fallow" which reduces water loss due to evapotranspiration and allows moisture to accumulate from rainfall. The moisture content of the subgrade is raised significantly and a large part of the swelling occurs prior to covering with structural courses of the highway. Pre-wetting for some months, with a grid of auger holes, is advisable for culvert sites.

Fallowing to increase the subgrade moisture content and induce heave is a simple and inexpensive procedure to eliminate a major part of the problem of expansive soils. It is particularly adaptable to highways and stage construction. Placing a layer of filter material or a subbase course is particularly beneficial. For example a 15 cm sand cover over black clay induced most of the heave in one season by permitting the moisture content to increase. The sand prevented evapotranspiration and evaporation and conserved the moisture from precipitation. Sand was found to be more effective than a plastic cover. Also a sand filter was found to be effective in reducing drying at the edge of the pavement by retaining rainwater and reducing evaporation through capillary rise. It may require 5 or 6 years for heave to reach equilibrium under structures. However, allowing the soil to fallow for one year, particularly with a granular cover, can accelerate heave by conserving moisture, and that it is quite likely that most of the heave will occur in 1 or 2 years.

Expansive characteristics can be reduced by chemical stabilization. In plastic clays, hydrated lime is usually more successful than cement. Lime improves the workability of the soil while cement does not; moreover, montmorillonitic clays retard the hydration of cement. If the black clays have appreciable organic matter, about 5 percent or more, stabilization may not be possible. Lime stabilization is beneficial for a number of reasons: (1) it reduces potential swelling in the top layer of the subgrade which would contribute most toward total heave: (2) more water has to be added than for the untreated soil, which helps to raise the moisture content nearer to final equilibrium moisture content; (3) the treated layer is more impermeable, so moisture variations are reduced; and (4) the workability of the soil is improved, particularly during the wet seasons. Deep-plow stabilization with lime has been effectively used on recent highway projects. Stabilized depths of 60 cm have been successfully completed and it may be possible to go as deep as 90 cm. Quality control is important on such projects. No other chemical is as effective as lime for expansive soils.

The third point, confining clays under embankment material, assumes that suitable embankment material is readily available. On the Ethiopian Plateau in Africa, for example, suitable material is scarce and much of the readily available embankment material may itself be expansive. Surcharge does reduce swell by an amount greater than its weight. It is estimated that heave can be reduced by about 30 percent for every 1 m of fill.

Minimizing moisture changes is undoubtedly a very important aspect. A recent investigation showed the importance of eliminating any design features which allow the accumulation of moisture in the pavement structure. Adequate provisions must be made for surface drainage; cracks must be filled and the surface maintained.

Drainage is particularly important in highways over black clays. If drainage is poor, seasonal variations in the subgrade moisture are unavoidable and heaving results. However, if drainage ditches are too deep, or too close to the pavement structure, seasonal drying or partial desiccation will occur along the shoulders. It is recommended that the shoulders be extended to a width equal to the depth of the active zone and locating the surface drainage as far away from the pavement as practicable. Recent studies have shown by means of the linear diffusion equation that seasonal moisture changes penetrate under the pavement as much as 3/4 of the thickness of the swelling layer.

The compactive effort applied to expansive soils and the corresponding optimum moisture content should correspond to Standard AASHO (or British Standard) rather than to more severe standards.

> Field measurements have indicated that either the optimum moisture content in the standard compaction test or the plastic limit is a useful guide to the moisture conditions under bituminous surfaced pavements, at least in these areas not governed by the water-table or where the surface drainage is not defective.

It is recommended that compaction be to 90 percent Modified AASHO maximum dry density. Swelling can be minimized by compacting clay above the plastic limit.

The final moisture content beneath pavements depends on soil conditions. Soil conditions are divided into three categories:

1. groundwater level close to ground level.
2. deep groundwater level and a yearly rainfall of over 250 mm;
3. arid climatic conditions and a yearly rainfall of less than 250 mm.

In the first category are pavements over clay subgrades with a groundwater depth of 5 to 6 m. In this case the water table controls the moisture conditions in the subgrade. For clays in the second category, field measurements have shown that the final moisture contents are close to the plastic limit or to the optimum moisture content at Standard AASHO compaction, which is usually close to the plastic limit. In the final category are those areas that are so arid that there is no moisture accumulation in the subgrade from below. Similarly, in those areas with very humid climates, the subgrade is usually close to the equilibrium moisture content.

The use of membranes as impervious blankets has increased in recent years in the western U.S. with generally good results. Catalytically blown asphalt about 4.8 mm thick is extended all the way across the subgrade from ditch line to ditch line.

Lime stabilization also serves the same purpose of waterproofing. Such treatments should also extend completely across the shoulders. Otherwise seasonal settlement of the shoulders due to shrinkage causes longitudinal cracks to form which readily admit water.

The CBR values for black clay soils are always low; most are under 2 after the normal four-day soaking period. It has been found in South Africa that the in-situ CBR values improve with time. Measurements on a number of black clay subgrade soils in roads up to 15 years old were always 6 or over. An explanation of this gain in strength with time is that the initial water content introduced during compaction partially fills voids and is adsorbed on mineral and soil particle surfaces. With time, some of the adsorbed water becomes adsorbed or structurally-held water within the caly mineral lattice. Also, adsorbed water, being a dipole molecule, becomes electrically oriented with time and, once oriented, is more firmly held on the clay mineral surfaces. In either event, the strength increases. The tendency to gain strength appears to be another advantage to stage construction, whereby subgrade CBR values can be expected to improve somewhat with time.

## RECOMMENDED DESIGN PROCEDURE

It is recommended that the following additions be made to the normal soil survey procedures for highways over black clay soils and to the design considerations.

The depth of the troublesome clay layer should be determined at sufficiently close intervals to provide a reasonably accurate picture of its stratigraphic position. This may be accomplished by normal hand or machine auger borings. The expansiveness of the clay should be determined along the length of alignment and with depth. The index properties can be used for this purpose, using (Figure 5.1) as an approximation of the potential expansiveness. The expansiveness of a few selected samples can be checked by means of the free swell and PVC tests.

The natural moisture contents and water table depths should be determined. These are important in all soil surveys but particularly important in expansive soils. Investigations for the water table should be carried to depths of at least 6 m. Low lying, poorly-drained areas with high water tables should be avoided.

The potential total heave can be approximated with the method which makes use of Figure 5.1. The potential unit heave at ground surface is assumed to be 25 mm for each 30 cm of depth of very highly expansive soil; 12.5 mm for each 30 cm of highly expansive soil; 6.2 mm for each 30 cm of medium expansive soil; and 0 mm for low or non-expansive soil. The potential heave can be estimated from:

$$H = \sum_{D=30\text{ cm}}^{D=n} F_D(\text{P.E.})$$

where

PE = potential expansiveness from Figure 5.1

$F_D$ = the factor by which heave decreases with depth.

The values of F are given in Table 5.1.

The potential expansiveness of all the soil layers are then added to determine the total potential heave. As an example, if only depths 0.3 to 1.2 m are of concern and the soil is highly expansive, the total heave could be estimated by:

$$\begin{aligned}
0.3\text{-}0.6\text{ m}, F_2 &= 0.842 \times 12.5 = 10.525\\
0.6\text{-}0.9\text{ m}, F_3 &= 0.750 \times 12.5 = 9.375\\
0.9\text{-}1.2\text{ m}, F_4 &= 0.668 \times 12.5 = 8.350\\
\text{total heave} &= 28.25 \text{ mm}
\end{aligned}$$

This approximation does not take into account the natural moisture content or what the ultimate moisture content might be. If the water table is below 6 m and the natural moisture content is near optimum, the approximation is too conservative. Nevertheless, a total potential heave of over 2.5 cm should be considered detrimental for highways and special design procedures or relocation of the alignment should be investigated.

Stage construction and allowing the subgrade to lie fallow for one year should be adopted whenever possible. A granular cover should also be provided not only to conserve moisture, but also to act as a filter. Very coarse material should not be placed directly on expansive soil, or any plastic clay for that matter. An overlying sand layer should be placed to prevent pumping of the fines into the structural layers. Quarry material in 1 cm to No. 10 sieve size is the most suitable filter size over cohesive soils.

Standard AASHO requirements for compaction and optimum moisture content, rather than Modified AASHO requirements, should be specified. The lower density and higher moisture content will reduce potential swelling. The lower density does not appreciably affect bearing capacity. The CBR values at both compactive efforts are very low. Moisture contents above optimum do not cause lower CBR values. Therefore, there is no advantage in specifying greater compaction.

The Tropical Design Procedure described in Chapter 3 can be applied to black clay soils. The minimum CBR value in these curves is 2 and this value can be used for soils having CBR's below this minimum. From the pavement performance survey, it was concluded that subgrades with CBR's less than 2 contribute no appreciable strength to the pavement structure.

For expansive soils the minimum cover should be 1 m. The minimum thickness design applies only to the subbase, base and surface thicknesses in those cases, or to the structural courses. For example, a non-expansive subgrade exhibiting a design CBR of 2 requires a minimum cover of 50 cm. However, an additional 50 cm of select material is required if the subgrade is expansive. The select material may be of any quality as long as it is not expansive. The minimum thickness of the overlying courses are then based on the CBR value of the select material.

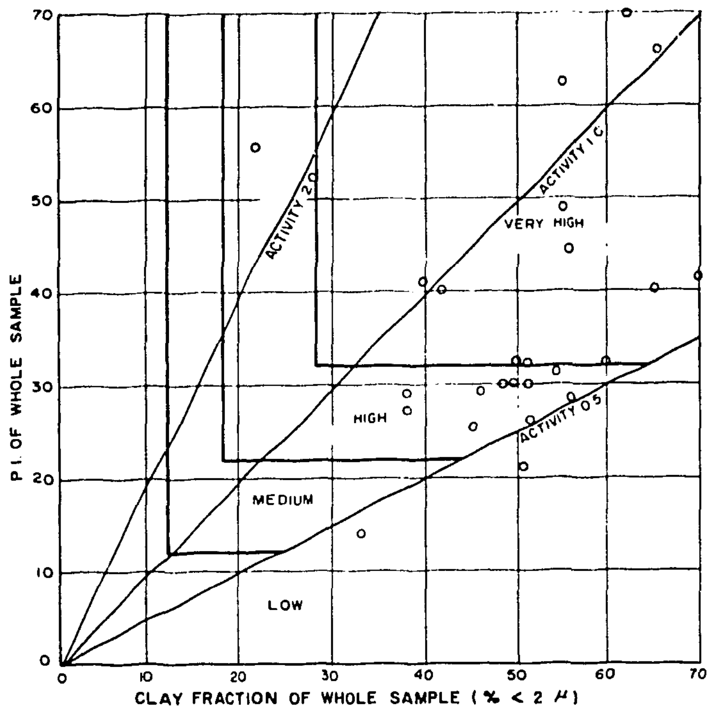



FIGURE 5.1 – POTENTIAL EXPANSIVENESS OF SOILS

## TABLE 5.1

Value of Factor F With Depth D From the Relation D = 20 log F

| Depth in m. | Depth in ft. | Mean Value of F | Depth in m. | Depth in ft. | Mean Value of F |
|---|---|---|---|---|---|
| 0-0.3 | 0-1 | $F_1$ 0.943 | 4.6-4.9 | 15-16 | $F_{16}$ 0.168 |
| 0.3-0.6 | 1-2 | $F_2$ 0.842 | 4.9-5.2 | 16-17 | $F_{17}$ 0.150 |
| 0.6-0.9 | 2-3 | $F_3$ 0.750 | 5.2-5.5 | 17-18 | $F_{18}$ 0.133 |
| 0.9-1.2 | 3-4 | $F_4$ 0.668 | 5.5-5.8 | 18-19 | $F_{19}$ 0.119 |
| 1.2-1.5 | 4-5 | $F_5$ 0.595 | 5.8-6.1 | 19-20 | $F_{20}$ 0.106 |
| 1.5-1.8 | 5-6 | $F_6$ 0.531 | 6.1-6.4 | 20-21 | $F_{21}$ 0.094 |
| 1.8-2.1 | 6-7 | $F_7$ 0.473 | 6.4-6.7 | 21-22 | $F_{22}$ 0.084 |
| 2.1-2.4 | 7-8 | $F_8$ 0.422 | 6.7-7.0 | 22-23 | $F_{23}$ 0.075 |
| 2.4-2.7 | 8-9 | $F_9$ 0.376 | 7.0-7.3 | 23-24 | $F_{24}$ 0.067 |
| 2.7-3.0 | 9-10 | $F_{10}$ 0.335 | 7.3-7.6 | 24-25 | $F_{25}$ 0.060 |
| 3.0-3.3 | 10-11 | $F_{11}$ 0.298 | 7.6-7.9 | 25-26 | $F_{26}$ 0.053 |
| 3.3-3.6 | 11-12 | $F_{12}$ 0.266 | 7.9-8.2 | 26-27 | $F_{27}$ 0.047 |
| 3.6-4.0 | 12-13 | $F_{13}$ 0.237 | 8.2-8.5 | 27-28 | $F_{28}$ 0.042 |
| 4.0-4.3 | 13-14 | $F_{14}$ 0.211 | 8.5-8.8 | 28-29 | $F_{29}$ 0.038 |
| 4.3-4.6 | 14-15 | $F_{15}$ 0.188 | 8.8-9.1 | 29-30 | $F_{30}$ 0.034 |

CHAPTER 6

*MATERIAL AND CONSTRUCTION SPECIFICATION*

Specifications are used to insure proper construction and to minimize or eliminate all basis for disputes between designers and contractors. Ideally, specifications should be used as a guide, for both the engineer and contractor in the performance of work outlined in the contract documents.

## SPECIFICATIONS FOR SUBBASE; BASE AND SURFACE COURSE MATERIALS

### Subbase and Base Courses

Recommended specifications for subbase and base course materials are given in Table 6.1. Six classifications of base course materials are shown in the table. The required base classification is determined by the design period which is the accumulated equivalent standard axle loadings in both directions (single lane design) used in the design computations. The six classifications also provide efficient use of available materials since thickness requirements are dependent upon the CBR of the component layers.

Limits for grading and Atterberg limits are shown for each classification. These are used as criteria in construction control testing. If a surface treatment is used in design it is recommended that the durability requirements given for surface course materials be included in the specifications for the base course material.

The design CBR is determined by a statistical analysis of samples obtained from the proposed borrow area. An illustrated example is shown in Figure 6.1. The percentage of the samples greater than a given CBR value is computed and plotted as shown on the lower right of Figure 6.1. An acceptable practice of selecting an appropriate CBR value is that which corresponds to a 90 percent confidence limit, i.e., the CBR value where 90 percent of all samples tested are greater than the selected value. For statistical purposes the minimum number of tests is 6. However, more tests may be required to evaluate a given borrow area. The number required would depend upon variations in the material and size of the borrow pit. In the illustrated example the CBR would be 55 or a Class VI base course. An alternative would be to use "selective" excavation which would eliminate areas exhibiting the lower CBR values. The borrow area would then receive a higher classification.

### Determination of CBR Values

A common practice is to soak the CBR sample for four days prior to testing. The reasoning is that the CBR should be determined at the worst possible condition that will exist during the design period. In high rainfall regions such a pretreatment prior to testing is warranted. The in-situ moisture contents within the various component layers were examined in Brazil. Figures 6.2 and 6.3 show the relationship between optimum moisture content at AASHO Modified compaction and in-situ moisture content for base, and subbase layers. The annual rainfall has been indicated for the individual test sections. Examination of these plots shows that the in-situ moisture content of the base materials does not exceed the optimum moisture content of the material until the annual rainfall exceeds 1500 mm. The in-situ moisture content of the subbase materials does not exceed the optimum moisture content of the material until the annual rainfall exceeds 800 mm. Figure 6.4 shows the relationship between annual rainfall and coefficient of equilibrium moisture content (c) for the subgrade materials. The equilibrium moisture content represents the amount of moisture the soil can accumulate which in subgrades under pavements is close to the plastic limit. The coefficient of equilibrium moisture content relates the plastic limit to the equilibrium moisture;

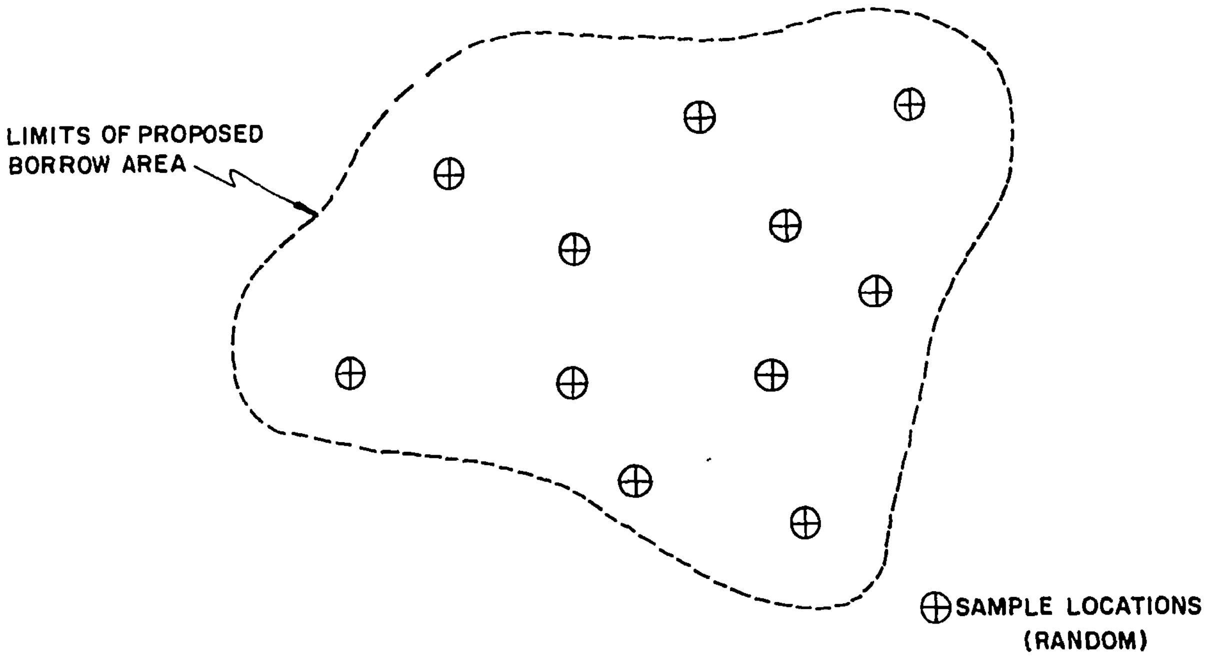
Equilibrium moisture = (c) (PL) .................. (6.1)

The band in Figure 6.4 represents the range of moisture contents which should be considered in the determination of the design CBR of the subgrade. Table 6.2 gives the range of moisture contents to be used in examining the CBR value of the respective layers for design purposes.

**TABLE 6.1**
**Specifications, Base and Subbase Materials**

| Criteria | Base Classification I | II | III | IV | V | VI | Subbase |
|---|---|---|---|---|---|---|---|
| Design CBR | +100 | 90 | 80 | 70 | 60 | 50 | 25 - 40 |
| Maximum traffic AESALBD | $10^7$ | $10^7$ | $10^7$ | $2 \times 10^6$ | $9 \times 10^5$ | $5 \times 10^5$ | – |
| Gradation | 1 | 1 | 1 | 2 | 2 | 2 | 3 |
| Maximum LL X (-200) | 600 | 900 | 900 | 900 | 1250 | 1250 | 1600 |
| Maximum PI X (-200) | 200 | 400 | 400 | 400 | 600 | 600 | 800 |
| Maximum Granularmetric Modulus | 490 | 525 | 550 | 580 | 600 | 615 | 630 |

AESALBD – Accumulated Equivalent Standard Axle Loadings in Both Directions
Granularmetric Modulus – Percent passing 1, 3/4, 1/2, 3/8, 4, 10, 40, 200 – sieves.

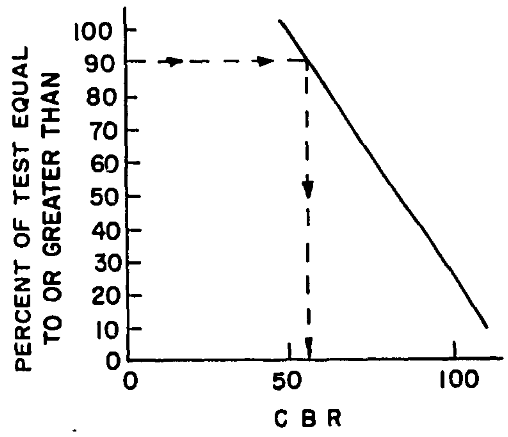



RESULTS OF LABORATORY TESTS

CBR = 50-60-65-70-75-80-80-90-90-90-110

STATISTICAL TEST

| CBR | NUMBER | PERCENT ≥ |
|---|---|---|
| 50 | 11 | 100 |
| 65 | 9 | 82 |
| 70 | 8 | 73 |
| 75 | 7 | 64 |
| 80 | 6 | 55 |
| 90 | 4 | 30 |
| 110 | 1 | 9 |




CBR = 55 ~ CLASS VI

FIGURE 6.1 – CLASSIFICATION OF BASE COURSE MATERIAL

## TABLE 6.2
**Recommended Moisture Range for Evaluation of Design CBR Values**

| Structural Layer | Annual Rainfall | | |
|---|---|---|---|
| | <800 mm<br>30 in | 800 to 1500 mm<br>30 to 60 in | >1500 mm<br>60 in |
| Base | OMC | OMC - 1.25 OMC | 4 day soak |
| Subbase | OMC | OMC - 1.5 OMC | 4 day soak |
| Subgrade | 0.4 - 0.6 PL | 0.7 - 1.2 PL | 0.9 - 1.5 PL |

**Conditions**: Water table at least 1 meter below pavement surface and good surface drainage.

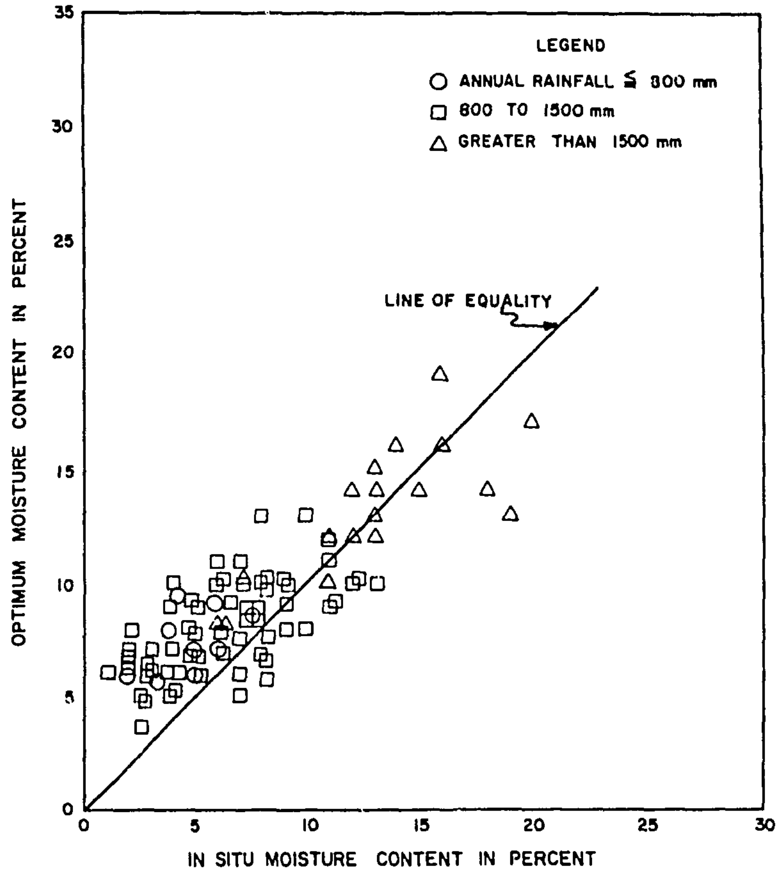



FIGURE 6.2 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND IN SITU MOISTURE CONTENT – BASE COURSES

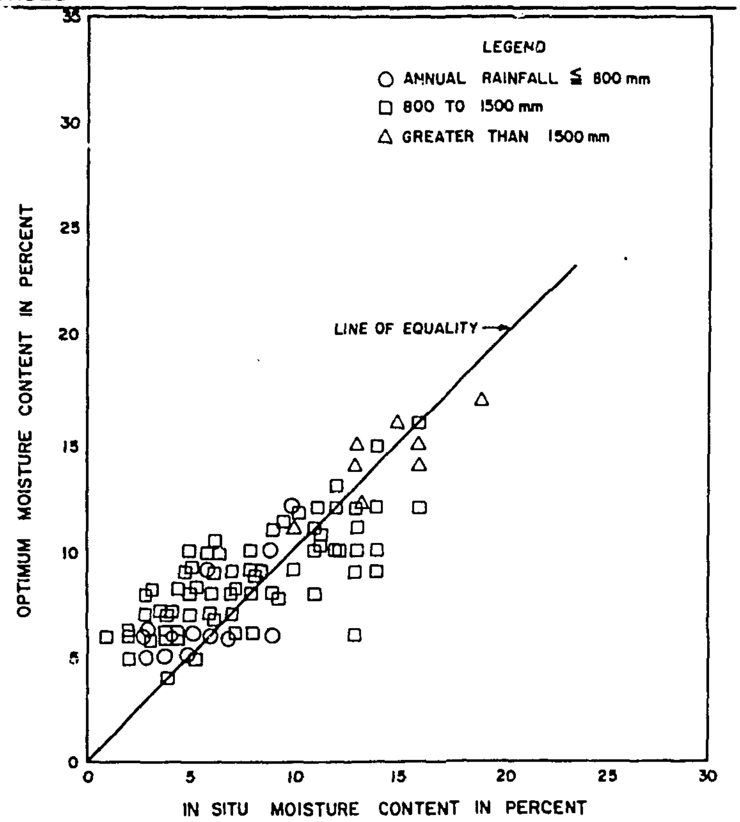



FIGURE 6.3 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND IN SITU MOISTURE CONTENT – SUBBASE COURSES

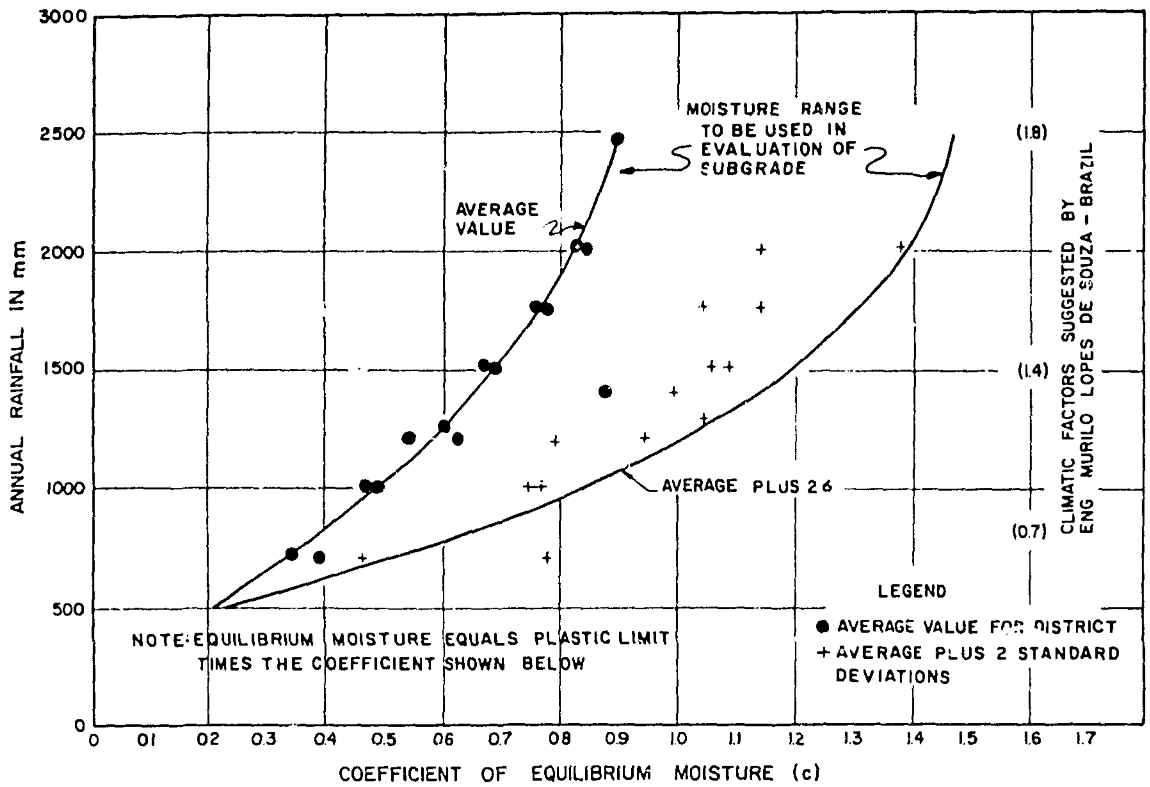



FIGURE 6.4 – ANNUAL RAINFALL VS COEFFICIENT OF EQUILIBRIUM MOISTURE

## Surface Course Materials

The selection of surface course materials is governed by gradation. Table 6.3 shows the various gradings used in selecting materials for the component layers. Grading 4 represents the recommended gradation for untreated surface course materials. The grading envelope was arrived at by studies of reports covering the performance of untreated gravel roads. The minimum CBR value for surface course materials is 40. The physical properties given for Class VI base courses or subbases can be used for control purposes.

The quality of ironstones and concretionary gravels should be evaluated before these materials are utilized for surface course materials.

Several types of durability tests were evaluated in Volume I Chapter 8. The most promising of these is the slake durability test. A tentative specification of a minimum slake durability value of 95 is recommended for ironstones and concretionary gravels when these materials are to be used in the surface course.

**TABLE 6.3**
Recommended Gradations

| Sieve | | 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|
| ISO* | ASTM-E11 | | | | |
| 50 mm | 2 in | 100 | 100 | 100 | 100 |
| 37.5 | 1 1/2 | 80 - 100 | 90 - 100 | 100 | 100 |
| 25 | 1 | 50 - 100 | 70 - 100 | 85 - 100 | 100 |
| 19 | 3/4 | 45 - 100 | 60 - 100 | 75 - 100 | 100 |
| 12.5 | 1/2 | 35 - 100 | 45 - 100 | 65 - 100 | 90 - 100 |
| 9.5 | 3/8 | 30 - 90 | 40 - 100 | 55 - 100 | 80 - 100 |
| 4.75 | No. 4 | 20 - 70 | 30 - 75 | 45 - 80 | 55 - 85 |
| 2 | 10 | 10 - 50 | 25 - 60 | 35 - 65 | 35 - 65 |
| 425 μm | 40 | 5 - 35 | 20 - 45 | 30 - 45 | 20 - 40 |
| 75 | 200 | 0 - 25 | 15 - 35 | 25 - 40 | 15 - 30 |

*ISO International Standards Organization, Geneva, Switzerland.

## SPECIFICATIONS FOR EXCAVATION OF BORROW AREAS, COMPACTION EQUIPMENT AND COMPACTION REQUIREMENTS

### Excavation of Borrow Areas*

In developing the layout of a borrow site, area utilization and drainage (especially during the rainy season) are key factors to consider. If scrapers are to be used, the borrow pit should be excavated from an uphill position down, and the furrows made by the scrapers should be continuous and provide for drainage away from the pit. However, if power shovels are to be used, the pit should be excavated from a down hill position up. This technique will permit natural drainage and prevent local ponding. All stripped soil should be placed in a location outside the borrow area. While the ironstone or concretionary gravel is being excavated, care should be taken to prevent excavation into underlying clayey silt. If silt is mixed in with the borrow material during excavation, it must be washed out prior to placement, otherwise serious local failure may be expected. Most lateritic soil can be excavated with a scraper or a scraper pushed by a bulldozer. However, the ironstone must be excavated with a bulldozer with a 35 to 45 cm ripper tooth. Blasting to excavate laterite is relatively impractical because the laterite has a high natural porosity.

### Compaction Equipment*

*(1) Ironstone or concretionary gravel (Laterite)*

During excavation, transportation, and compaction, an effort should be made to prevent unnecessary structural degradation of the laterite; therefore, compaction should be light and shaping kept to a minimum to avoid high shear stresses. The reference to "light compaction" is to indicate that lighter rollers and good moisture control should be employed to obtain the specified density and that heavy compactors should not be used as panacea to all compaction processes. For the wormhole laterite, the 7,258 to 9,720 kg vibratory steel wheel roller gives the best result. For pellet laterite, the 4,536 to 7,258 kg steel wheeled or pneumatic tire rollers are the most effective.

*(2) Lateritic Soils*

In Thailand, a variation of a sheepsfoot roller was used effectively in compacting a lateritic soil. The roller is similar to a sheepsfoot roller except that the feet are flatter and have a larger surface area. It has the capability of compacting thick lifts; for example, a 30 cm loose lift can be compacted to a 15 cm. However, the loose lift thickness is usually limited to 15 cm and is compacted to a thickness of 8 to 10 cm. Contrary to popular opinion, the sheepsfoot roller can be used effectively if the weight of the roller is reduced (usually by only half filling the drum) and the roller is pulled slowly to avoid high shear stresses. The 45,360 kg roller can be used, but the load and tire pressure are critical; they must be adjusted to approximately 22,680 kg and 6.3 to 7.7 kg/cm $^2$ respectively. A versatile roller which provides the "compaction action" of both a steel wheel and a sheepsfoot is the "Hyster" Grid Roller. This type of roller should prove to be very effective in the compaction of both laterite and laterite soils.

Lateritic soils compacted on the wet side of OMC often give a spongy section instead of a suitable compacted layer. A good rule of thumb for the field is to apply water at 2 percent less than the lab optimum moisture content.

### Compaction Requirements

The specified compaction requirements have been developed through analysis of the deflection and compaction relationships discussed in Volume 1 Chapter 12. The recommended density requirements shown in Table 6.4 are expressed as a function of the depth beneath the surface of the pavement. The requirements are applicable to natural subgrades in which case the required compaction will prevent excessive deflection of the layer but do not necessarily provide for the development of shear strengths required in the stability of high fills.

**TABLE 6.4**
**RECOMMENDED COMPACTION REQUIREMENT FOR COMPONENT STRUCTURAL LAYERS AND NATURAL SUBGRADE**

| Depth Below Surface | Compaction |
|---|---|
| 0 – 25 cm ( 0 – 10 in) | 100% AASHO MOD |
| 25 – 45 cm (10 – 18 in) | 95% AASHO MOD |
| 45 – 60 cm (18 – 24 in) | 100% AASHO STD |
| 60 – 90 cm (24 – 36 in) | 95% AASHO STD |

* From U.S. Army Corps of Engineers (1968).

## SPECIFICATIONS FOR MATERIALS AND CONSTRUCTION IN TROPICAL CLIMATES

The standard specifications for asphalt, Portland cement and lime as well as specifications for surface treatment pavements are listed below. Specifications for asphalt concrete surfacing are not presented. The specifications provided are given in the following list.

### LIST OF DETAILED SPECIFICATIONS

## TABLE 6.5
### Specifications for Rapid-Curing (RC) Liquid Asphalts-Asphalt Institute

| Characteristics | AASHO Test Method | ASTM Test Method | Grades | | | |
|---|---|---|---|---|---|---|
| | | | RC-70 | RC-250 | RC-800 | RC-3000 |
| Kinematic Viscosity at 140°F., cs | T 201 | D2170 | 70-140 | 250-500 | 800-1600 | 3000-6000 |
| Flash Point (Tag. Open Cup), F.1 | T 79 | D1310 | ......... | 80 + | 80 + | 80 + |
| Distillation | | | | | | |
| Distillate (% by Volume of Total Distillate to 680°F.) | | | | | | |
| To 374° F. | | | 10 + | ........ | ........ | ........ |
| To 437° F. | T78 | D402 | 50 + | 35 + | 15 + | ........ |
| To 500° F. | | | 70 + | 60 + | 45 + | 25 + |
| To 600° F. | | | 85 + | 80 + | 75 + | 70 + |
| Residue from Distillation to 680° F., F., % Volume by Difference | | | 55 + | 65 + | 75 + | 80 + |
| Tests on Residue from Distillation | | | | | | |
| Penetration, 77°F., 100 g., 5 sec. | T 49 | D5 | 80-120 | 80-120 | 80-120 | 80-120 |
| Ductility, 77°F., cms. | T 51 | D113 | 100 + | 100 + | 100 + | 100 + |
| Solubility in Carbon Tetrachloride, %[1] | T 44 | D2042 | 99.5 + | 99.5 + | 99.5 + | 99.5 + |
| Water, %T 55 | D55 | D95 | 0.2 – | 0.2 – | 0.2 – | 0.2 – |

General Requirement – The material shall not foam when heated to application temperature recommended by The Asphalt Institute.

## TABLE 6.6
### Specifications For Medium-Curing (MC) Liquid Asphalts-Asphalt Institute

| Characteristics | AASHO Test Method | ASTM Test Method | Grades | | | | |
|---|---|---|---|---|---|---|---|
| | | | MC-30 | MC-70 | MC-250 | MC-800 | MC-3000 |
| Kinematic Viscosity at 140° F., cs. | T 201 | D2170 | 30,60 | 70,140 | 250-500 | 800-1600 | 3000-6000 |
| Flash Point (Tag. Open Cup) ° F. | T 79 | D1310 | 100 + | 100 + | 150 + | 150 + | 150 + |
| Distillation | | | | | | | |
| Distillate (% by Volume et Total Distillate to 630° F.) | | | | | | | |
| To 437° F. | | | 25 – | 20 – | 0-10 | | |
| To 500° F. | T 70 | D402 | 40-70 | 20-60 | 15-55 | 35 – | 15 – |
| To 600° F. | | | 75-93 | 63-90 | 60-87 | 45-80 | 15-75 |
| Residue from Distillation to 680°F., % Volume by Difference | | | 50 + | 55 + | 67 + | 75 + | 80 + |
| Testes on Residue from Distillation | | | | | | | |
| Penetration, 77°F., 100 g., 5 sec. | T 49 | D5 | 120-250 | 120-250 | 120-250 | 120-250 | 120-250 |
| Ductility, 77°F., cms[3] | T51 | D113 | 100 + | 100 + | 100 + | 100 + | 100 + |
| Solubility in Carbon Tetrachloride, %[3] | T44 | D2042 | 99.5 + | 99.5 + | 99.5 + | 99.5 + | 99.5 + |
| Water, % | T55 | D95 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |

General requirement – The material shall not foam when heated to application temperature recommended by The Asphalt Institute.

## TABLE 6.7

### Specifications for Anionic Emulsified Asphalts – Asphalt Institute

| Characteristics | AASHO Test Method | ASTM Test Method | Grades: Rapid Setting[2] | | Medium Setting | Slow Setting[3] | |
|---|---|---|---|---|---|---|---|
| | | | RS-1 | RS-2 | MS-2 | SS-1 | SS-1h |
| Tests on Emulsion[1] | | | | | | | |
| Furol Viscosity at 77°F., sec. | | | 20-100 | – | 100+ | 20-100 | 20-100 |
| Furol Viscosity at 122°F., sec. | | | – | 75-400 | – | – | – |
| Residue from Distillation, % by weight | | | 57+ | 62+ | 62+ | 57+ | 57+ |
| Settlement, 5 days, % difference | T59 | D244 | 3- | 3- | 3- | 3- | 3- |
| Demulsibility: | | | | | | | |
| 35 ml. of 0.02 N $CaCl_2$, % | | | 60+ | 50+ | – | – | – |
| 50 ml. of 0.10 N $CaCl_2$, % | | | – | – | 30- | – | – |
| Sieve Test (Retained on No. 20), % | | | 0.10- | 0.10- | 0.10- | 0.10- | 0.10- |
| Cement Mixing Test, % | | | – | – | – | 2.0 - | 2.0 - |
| Tests on Residue | | | | | | | |
| Penetration, 77°F, 100 g., 5 sec. | T49 | D5 | 100-200 | 100-200 | 100-200 | 100-200 | 40-90 |
| Solubility in Carbon Tetrachloride, % | T44 | D4 | 97.5+ | 97.5+ | 97.5+ | 97.5+ | 97.5+ |
| Ductility 77°F., cms. | T51 | D113 | 40+ | 40+ | 40+ | 40+ | 40+ |

NOTE: 1 To be Used with Calcareous Materials Only
2 Rapid Setting Emulsions for Surface Treatments
3 Slow Setting Emulsions for Slurry-Seal

## TABLE 6.8

### Specifications for Cationic Emulsified Asphalts – Asphalt Institute

| Characteristics | AASHO Test Method | ASTM Test Method | Grades: Rapid Setting | | Medium Setting | | Slow Setting[2] | |
|---|---|---|---|---|---|---|---|---|
| | | | RS-2K | RS-3K | SM-K | CM-K | SS-K | SS-Kh |
| Tests on Emulsion[1] | | | | | | | | |
| Furol Viscosity at 77°F., sec. | T59 | D244 | – | – | – | – | 20-100 | 20-100 |
| Furol Viscosity at 122°F., sec. | T59 | D244 | 20-100 | 100-400 | 50-500 | 50-500 | – | – |
| Residue from Distillation: | | | | | | | | |
| Residue, % by weight | T59 | D244 | 60+ | 65+ | 60+ | 65+ | 57+ | 57+ |
| Oil Distillate, % by volume of Emulsion | T59 | D244 | 5- | 5- | 20- | 12- | – | – |
| Settlement, 7 days, % differ. | T59 | D244 | 3- | 3- | 3- | 3- | 3- | 3- |
| Sieve Test (Retained on No. 20) % | T59 | D244 | 0.10- | 0.10- | 0.10- | 0.10- | 0.10- | 0.10- |
| Aggregate Coating Water Resistance Test | – | D244 | – | – | – | – | – | – |
| Dry Aggregate (Job), % Coated | | | – | – | 80+ | 80+ | – | – |
| Wet Aggregate (Job), % Coated | | | – | – | 60+ | 60+ | – | – |
| Cement Mixing Test, % | T59 | D244 | – | – | · | – | 2- | 2- |
| Particle Charge Test | T59A | D244 | Positive | Positive | Positive | Positive | – | – |
| pH | T200 | E70 | – | – | – | – | 6.7- | 6.7- |
| Tests on Residue | | | | | | | | |
| Penetration, 77° F. 100 g. 5 sec | T49 | D5 | 100-250 | 100-250 | 100-250 | 100-250 | 100-200 | 40-90 |
| Solubility in Carbon Tetrachloride, % | T44 | D4 | 97.0+ | 97.0+ | 97.0+ | 97.0+ | 97.0+ | 97.0+ |
| Ductility, 77°F., cm. | T51 | D113 | 40+ | 40+ | 40+ | 40+ | 40+ | 40+ |

Note: 1 Rapid Setting Emulsions for Surface Treatments
2 Slow Setting Emulsions for Slurry-Seal

## STANDARD SPECIFICATION FOR PORTLAND CEMENT

*AASHO DESIGNATION: M 85-70*

### SCOPE

1.1 This specification covers five types of portland cement, as follows:

*Type I.* For use in general concrete construction when the special properties specified for types II, III, IV and V are not required.

*Type II.* For use in general concrete construction exposed to moderate sulfate action, or where moderate heat of hydration is required.

*Type III.* For use when high early strength is required.

*Type IV.* For use when a low heat of hydration is required. (Note 1)

*Type V.* For use when high sulfate resistance is required. (Note 1)

Note 1 Attention is called to the fact that cements conforming to the requirements for type IV and type V are not usually carried in stock. In advance of specifying their use, purchasers or their representatives should determine whether these types of cement are, or can be made available.

### Basis of Purchase

2.1 The purchaser should specify the type or types desired. When no type is specified, the requirements of type I shall govern.

### Definition

3.1 Portland cement is the product obtained by pulverizing clinker consisting essentially of hydraulic calcium silicates to which no additions have been made subsequent to calcination other than water and/or untreated calcium sulfate, except that additions of other nondeleterious materials may be added at the option of the manufacturer in an amount not to exceed 0.1 per cent.

### Chemical Limits

4.1 Portland cement of each of the five types shown in Section 1 shall conform to the requirements prescribed in Table 1.

### Physical Requirements

5.1 Portland cement of each of the five types shown in Section 1 shall conform to the requirements prescribed in Table 2.

### Packaging and Marking

6.1 When the cement is delivered in packages, the name and brand of the manufacturer and the type shall be plainly identified thereon, except that, in the case of type I cement, the type need not be identified. When the cement is delivered in bulk this information shall be contained in the shipping advices accompanying the shipment. A bag shall contain 94 lb. net. A barrel shall consist of 376 lb. net. All packages shall be in good condition at the time of inspection.

### Storage

7.1 The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weathertight building that will protect the cement from dampness and minimize warehouse set.

### Inspection

8.1 Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. The following periods from time of sampling shall be allowed for completion of testing:

| | |
|---|---|
| 1 day test | 6 days |
| 3 day test | 8 days |
| 7 day test | 12 days |
| 28 day test | 33 days |

### Rejection

9.1 The cement may be rejected if it fails to meet any of the requirements of this specification.

9.2 Cement remaining in bulk storage at the mill, prior to shipment, for a period greater than six months, or cement in bags in local storage in the hands of a vendor for more than three months, after completion of the tests may be retested and may be rejected if it fails to conform to any of the requirements of this specification.

9.3 Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

9.4 Cement failing to meet the test for soundness in the autoclave may be accepted if it passes a retest, using a new sample, at any time within 28 days thereafter. The provisional acceptance of the cement at the mill shall not deprive the purchaser of the right to reject on a retest of soundness at the time of delivery of the cement to the purchaser.

# TABLE 1

***Chemical Requirements***

**M 85**

| | Type I | Type II | Type III | Type IV[1] | Type V[1] |
|---|---|---|---|---|---|
| Silicon dioxide ($SiO_2$), min, per cent | – | 21.0 | – | – | – |
| Aluminum oxide ($Al_2O_3$), max, per cent | – | 6.0 | – | – | – |
| Ferric oxide ($Fe_2O_3$), max, per cent | – | 6.0 | – | 6.5 | – |
| Magnesium oxide (MgO), max, per cent | 5.0 | 5.0 | 5.0 | 5.0 | 4.0 |
| Sulfur trioxide ($SO_3$), max, per cent: | | | | | |
| When $3CaO.Al_2O_3$ is 8 per cent or less | 3.0 | 3.0 | 3.5 | 2.3 | 2.3 |
| When $3CaO.Al_2O_3$ is more than 8 per cent | 3.5 | – | 4.5 | – | – |
| Loss on ignition, max, per cent | 3.0 | 3.0 | 3.0 | 2.5 | 3.0 |
| Insoluble residue, max, per cent | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 |
| Sodium and potassium oxide ($Na_2O$+0.658 $K_2O$), max, per cent[2] | 0.6 | 0.6 | 0.6 | 0.6 | 0.6 |
| Tricalcium silicate ($3CaO.SiO_2$),[3] max, per cent | – | 55[5] | – | 35 | – |
| Dicalcium silicate ($2CaO.SiO_2$),[3] min, per cent | – | – | – | 40 | – |
| Tricalcium aluminate ($3CaO.Al_2O_3$),[3] max, per cent | 15 | 8 | 15[4] | 7 | 5 |
| Tetracalcium aluminoferrite plus twice the tricalcium aluminate[3] ($4CaO.Al_2O_3$+2($3CaO.Al_2O_3$), or solid solution ($4CaO.Al_2O_3.Fe_2O_3$ + $2CaO.Fe_2O_3$), as applicable, max, per cent | – | – | – | – | 20.0 |

(1) See Note 1, p. 81.

(2) This requirement applies only when the Engineer specifies "low-alkali cement." Such cement should be specified only when alkali-reactive aggregates are to be used in the concrete. The maximum value of 0.6% may be reduced when the experience of the Engineer indicates that such action is desirable.

(3) The expressing of chemical limitations by means of calculated assumed compounds does not necessarily mean that the oxides are actually or entirely present as such compounds.

When the ratio of percentages od aluminum oxide to ferric oxide is 0.64 or more, the percentages of tricalcium silicate, dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite shall be calculated from the chemical analysis as follows:

Tricalcium silicate = (4.071 X per cent CaO) (7.600 X per cent $SiO_2$) – (6.718 X per cent $Al_2O_3$) – (1.430 X per cent $Fe_2O_3$) – (2.852 X per cent $SO_3$)

Dicalcium silicate = (2.867 X per cent $SiO_2$) – (0.7544 X per cent $C_3S$)

Tricalcium aluminate = (2.650 X per cent $Al_2O_3$) – (1.692 X per cent $Fe_2O_3$)

Tetracalcium aluminoferrite = 3.043 X per cent $Fe_2O_3$

When the alumina-ferric oxide ratio is less than 0.64, a calcium aluminoferrite solid solution (expressed as ss ($C_4AF$ + $C_2F$) is formed. Contents of this solid solution and of tricalcium silicate shall be calculated by the following formulas:

ss ($C_4AF$ + $C_2F$) = (2.100 X per cent $Al_2O_3$) + (1.702 X per cent $Fe_2O_3$)

Tricalcium silicate = (4.071 X per cent CaO) – (7.600 X per cent $SiO_2$) – (4.479 X per cent $Al_2O_3$) (2.859 X per cent $Fe_2O_3$) – (2.852 X per cent $SO_3$).

No tricalcium aluminate will be present in cements of this composition. Dicalcium silicate shall be calculated as previously shown.

In the calculation of $C_3A$, the values of $Al_2O_3$ determined to the nearest 0.01 per cent shall be used.

Values for $C_3A$ and for the sum of $C_4AF$ + $2C_3A$ shall be reported to the nearest 0.1 per cent. Values for other compounds shall be reported to the mearest 1 per cent.

(4) When moderate sulfate resistance is required for type III cement, tricalcium aluminate may be limited to 8 per cent. When moderate sulfate resistance is required, the tricalcium aluminate may be limited to 5 per cent.

(5) When moderate heat of hydration is required, a limit of 58 per cent on the total tricalcium silicate and tricalcium aluminate shall apply for type II cement.

## Methods of Sampling and Testing

10.1 The sampling and testing of portland cement shall be in accordance with the following standard methods of the American Association of State Highway Officials:

Sampling . . . . . . . . . . . T 127
Chemical analysis . . . . . . T 105

Sodium and potassium oxides may be determined by either the flame photometry method outlined in T 105, Part II, or the gravimetric method outlined in T 105, Part I.

Fineness:
Turbidimeter . . . . . . . . . . T 98
Air permeability . . . . . . . . T 153
Soundness . . . . . . . . . . . . T 107
Time of setting:
Gillmore needles . . . . . . . T 154
Vicat . . . . . . . . . . . . . . . T 131
Air content of mortar . . . . . T 137
Normal consistency . . . . . . T 129
Tensile strength . . . . . . . . T 132
Compressive strength . . . . . T 106
False set . . . . . . . . . . . . . T 186

## TABLE 2
### Physical Requirements
### M 85

| | Type I | Type II | Type III | Type IV[1] | Type V[1] |
|---|---|---|---|---|---|
| Fineness, specific surface, sq.cm. per g. (alternate methods):[2] | | | | | |
| Turbidimeter test: | | | | | |
| Average value, min | 1.600 | 1.600 | ... | 1.600 | 1.600 |
| Min. value, any one sample | 1.500 | 1.500 | ... | 1.500 | 1.500 |
| Average value, max | 2.200 | 2.200 | ... | 2.200 | 2.200 |
| Max. value, any one sample | 2.300 | 2.300 | ... | 2.300 | 2.300 |
| Air permeability test: | | | | | |
| Average value, min | 2.800 | 2.800 | ... | 2.800 | 2.800 |
| Min. value, any one sample | 2.600 | 2.600 | .. | 2.600 | 2.600 |
| Average value, max | 4.000 | 4.000 | ... | 4.000 | 4.000 |
| Max. value, any one sample | 4.200 | 4.200 | ... | 4.200 | 4.200 |
| Soundness: | | | | | |
| Autoclave expansion, max., per cent | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Time of setting (alternate methods):[3] | | | | | |
| Gillmore test: | | | | | |
| Initial set, min., not less than | 60 | 60 | 60 | 60 | 60 |
| Final set, hr., not more than | 10 | 10 | 10 | 10 | 10 |
| Vicat test (T131): | | | | | |
| Set, min., not less than | 45 | 45 | 45 | 45 | 45 |
| Air content:[5] | | | | | |
| Air content of mortar prepared and tested in accordance with Method T 137, max., per cent by volume | 12.0 | 12.0 | 12.0 | 12.0 | 12.0 |
| Tensile strength, psi:[4] | | | | | |
| The average tensile strength of not less than three standard mortar briquets, prepared in accordance with Method T 132, shall be equal to or higher than the values specified for the ages indicated below: | | | | | |
| 1 day in moist air | ... | ... | 275 | ... | ... |
| 1 day in moist air, 2 days in water | 150 | 125 | 375 | ... | ... |
| 1 day in moist air, 6 days in water | 275 | 250 | ... | 175 | 250 |
| 1 day in moist air, 27 days in water | 350 | 325 | ... | 300 | 325 |
| Compressive strength, psi.:[4] | | | | | |
| The average compressive strength of not less than three mortar cubes, prepared in accordance with Method T 106, shall be equal to or higher than the values specified for the ages indicated below: | | | | | |
| 1 day in moist air | ... | ... | 1.700 | ... | ... |
| 1 day in moist air, 2 days in water | 1.200 | 1.000 | 3.000 | ... | ... |
| 1 day in moist air, 6 days in water | 2.100 | 1.800 | ... | 800 | 1.500 |
| 1 day in moist air, 27 days in water | 3.500 | 3.500 | ... | 2.000 | 3.000 |
| False set, final penetration, min. per cent[6] | 50 | 50 | 50 | 50 | 50 |

1 See Note 1, p. 81

2 Either of the two alternate fineness methods may be used at the option of the test laboratory. However in case of dispute or when the sample fails to meet the requirements of the Blaine meter, the Wagner turbidimeter shall be used, and the requirements in Table 2 for this method shall govern.

3 The purchase should specify the type of setting time test required. In case he does not so specify, or in case of dispute, the requirement of the Vicat test only shall govern.

4 The purchaser shall specify the type of strength test desired. In case he does not so specify the requirements of the compressive strength test only shall govern. Unless otherwise specified, the strength tests for Types I and II cement will be made only at 3 and 7 days. The strength at any age shall be higher than the strength at the next preceding age.

5 Cement producing an air content of mortar between 12 and 16 percent may be accepted at the discretion of the purchaser when it is to be used in air entraining concrete and the air content of this entraining concrete is controlled at the mixer.

6 This requirement applies only when specifically requested.

## SPECIFICATIONS FOR LIME

### Standard Specification for Lime for Soil Stabilization

### AASHO DESIGNATION: M 216-68

### Scope

1.1 This specification covers two types, including 3 grades each, of lime to be used for soil stabilization as follows:

1.1.1 Type I. High calcium hydrated lime containing a maximum magnesium content, calculated as magnesium oxide, of 4 percent by weight. Compliance with chemical composition requirements shall be determined by use of AASHO T 219.

1.1.2 Type II. Magnesium or dolomitic lime containing magnesium, calculated as magnesium oxide, greater than 4 but no more than 36 percent by weight. Compliance with chemical composition requirements shall be determined by use of ASTM C25-58 (Chemical Analysis of Limestone, Quicklime and Hydrated Lime).*

Note. No attempt is made to present requirements for waste lime, commercial lime slurry, etc. Specification requirements for these materials could be better determined on a local basis.

*Except that Section 2, "Samples for Analysis," is excluded.

### Basis of Purchase

2.1 The purchaser and contractor should agree upon the grade desired before construction begins. When no grade is agreed upon the requirements of Grade A shall govern. When Grades B and C are used, plan quantities for Grade A shall be increased, without additional cost to the purchaser, as follows:

| Type | Grade B | Grade C |
|---|---|---|
| I | 6% | 20% |
| II | 2% | 5% |

### Chemical Limits

3.1 Type I lime when sampled and tested by procedures prescribed herein shall conform to the following requirements:

| | Grade A | Grade B | Grade C |
|---|---|---|---|
| Min. Hydrate alkalinity, percent by weight $Ca(OH)_2$ ........................ | 90 | 85 | 75 |
| Max. Unhydrated lime content, percent by weight CaO....... .............. | 7 | 8 | 9 |
| Max. "Free Water" content, percent by weight $H_2O$.................... | 3 | [illegible] | 2 |

3.2 Type II lime when tested in accordance with ASTM Designation C 25 shall conform to the following requirements:

| | Grade A | Grade B | Grade C |
|---|---|---|---|
| Calcium and magnesium oxide content of ignition residue, minimum percent | 98 | 96 | 94 |
| Carbon dioxide (as received basis) maximum percent | 3 | 4 | 8 |
| Unhydrated calcium oxide (as received basis) maximum percent | 7 | 8 | 9 |
| "Free Water" content, maximum percent | 3 | 3 | 2 |

Ignition to constant weight shall be performed utilizing an electric muffle furnace operating at 1000-1100°C (1800-2000 F.).

### Physical Requirements

4.1 Type I and Type II lime shall conform to the following particle size requirements when tested according to AASHO T 192 (modified):

| | Grade A | Grade B | Grade C |
|---|---|---|---|
| Max. residue retained on a No. 30 (0.600 mm) sieve, percent by weight | 2 | 3 | 4 |
| Max. residue retained on a No. 200 (0.075 mm) sieve, percent by weight | 12 | 14 | 18 |

### Packaging and Marking

5.1 When the lime is delivered in bags, the name and brand of the manufacturer and type shall be plainly identified thereon. A bag shall contain a nominal weight of 50 pounds (25 kg) and all bags shall be in good condition at time of inspection. When lime is delivered in bulk, information regarding type and manufacturer shall be contained in the invoice accompanying the shipment.

### Inspection

6.1 Every facility shall be provided the purchaser for careful sampling and inspection of the lime at either the plant or at the site of the work as may be specified by the purchaser.

### Rejection

7.1 The lime shall be rejected if it fails to meet any of the requirements of these specifications.

7.2 Lime remaining in storage that becomes difficult to handle or that fails to conform to the requirements of this specification when resampled and tested, or both, may be rejected.

7.3 In the case of bag lime, bags varying more than 5 percent from the specified weight may be rejected. If the average weight of bags in any shipment as shown by weighing 50 bags taken at random is less than that specified, the entire shipment may be rejected.

### Methods of Sampling and Testing

8.1 The sampling and testing of hydrated lime shall be in accordance with the following standard methods of AASHO or ASTM:

Sampling–AASHO T 218
Chemical Analysis for Type I lime–AASHO T 219
Chemical Analysis for Type II lime–ASTM C 25*
Particle Size Analysis–AASHO T 219

## TABLE 6.9

## SPECIFICATIONS FOR STABILIZATION

### ASPHALT STABILIZATION

| Sieve Size | % Passing |
|---|---|
| 1" | 100 |
| 4 | 50 – 100 |
| 30 | 25 – 100 |
| 100 | 10 – 65 |
| 200 | 5 – 25 |
| by weight of total mix | 3 – 15% asphalt |

The above represents the normal gradation limits of materials recommended for stabilization with asphalt. However, by proper selection of types and grades of asphalt materials soils outside of these limits can be stabilized.

### CEMENT STABILIZATION

| AASHO Soil Group | Range of Cement Percent by Weight |
|---|---|
| A-1-a | 3 – 5 |
| A-1-b | 5 – 8 |
| A-2-4 | 5 – 8 |
| A-2-5 | 5 – 8 |
| A-2-6 | 5 – 9 |
| A-2-7 | 5 – 9 |
| A-3 | 7 – 11 |
| A-4 | 7 – 12 |
| A-5 | 8 – 13 |
| A-6 | 9 – 15 |
| A-7 | 10 – 16 |

The plasticity of the material governs the ability of mixing the cement into the soil. It is generally recognized that the mixing operation is usually the most important phase of a cement stabilization project. It is recommended that materials displaying plastic index above 10 be pre-treated with lime prior to mixing the cement into the material. Cement has been known to stabilize heavy clays when used in combination with a pre-treatment of lime.

### LIME STABILIZATION

Lime stabilization is suitable for a broad range of soil types. Lime can be used in combination with a pozzolan for stabilization of sandy soils. Lime does not generally react with silts. The best criteria to use in establishing the suitability of lime as a stabilizing agent is the changes which occur due to the introduction of lime into the soil. The principal changes occurring during lime stabilization are as follows:

1. Reduction in PI and volume change.
2. Flocculation of clay particles, making the soil more friable.
3. Increase in optimum moisture content, thereby permitting compaction under wetter conditions; soils dry out more rapidly.
4. Increase in strength and stability through cementing action.
5. Resistence to water absorption and capillary rise.

## SUGGESTED SPECIFICATIONS FOR MULTI-LAYER SURFACE TREATMENTS IN TROPICAL CLIMATES USING LIQUID ASPHALTS (AFTER PULLEN AND PREVOST 1969)

1. *Prime coat with a single seal coat*
   Prime M. C. 30 Spray at 0.12 to 0.20 glns/sq.yd.
   *Seal Coat:* 80/100 Pen. or M.C.3000 } Spray at 0.18 to 0.22 glns/sq.yd.
   Cover aggregate $\frac{1}{2}$ inch nominal size, spread 90 sq.yds/ton.

   The Prime coat must be left to dry before applying the seal coat and the rate of application will depend on the condition of the road surface and the speed of drying to a uniform film.

2. *Prime coat with two coat surface treatment*
   Prime M. C. 30 Spray at 0.12 to 0.20 glns/sq.yd.
   1st Seal: 80/100 Pen. or M. C. 3000 } Spray at 0.18 to 0.22 glns/sq.yd.
   Cover aggregate 3/4 inch nominal size at 70 sq.yd/ton
   2nd Seal: 80/100 Pen. or M.C.3000 } Spray at 0.22 to 0.26 glns/sq.yd.
   Cover aggregate 3/8 inch nominal size at 110 sq.yd/ton

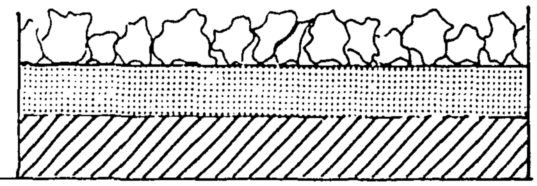
3. *Prime coat and two coat surface treatments* with second coat applied after road has been open to traffic for several months.
   Prime: M. C. 30 Spray at 0.12 to 0.20 glns/sq.yd.
   1st Seal: 80/100 Pen. or M. C. 3000 } Spray at 0.15 to 0.20 glns/sq.yd.
   Cover aggregate 3/8 inch nominal size at 110 sq.yd/ton.
   2nd Seal (after several months)
   80/100 Pen. or M. C. 3000 } Spray at 0.20 to 0.24 glns/sq.yd.
   Cover aggregate 1/2 inch nominal size at 90 sq.yds/ton

In East Africa the following specifications have been noted.

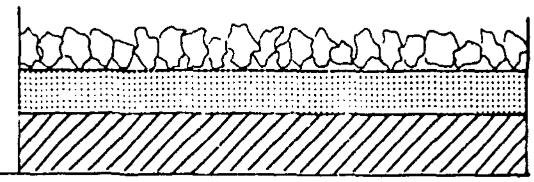
4. *No Prime Coat*
   1st Seal: 180/200 Pen or R.C. 2 } With 3/8 inch aggregate 0.20 to 0.22 glns/sq.yd.
   With 1/2 inch aggregate 0.22 to 0.25 glns/sq.yd.
   2nd Seal: 180/200 Pen or 80/100 Pen } 0.33 glns/sq.yd. with 3/4 inch aggregate.
   Cover aggregate application:
   3/8 inch nominal size: 90 to 140 sq.yd/cubic yd.
   1/2 inch nominal size: 65 to 95 sq.yd/cubic yd.
   3/4 inch nominal size: 40 to 65 sq.yd/cubic yd.

5. *Prime and Triple Seal*
   Prime coat: M. C. 2 0.10 to 0.12 glns/sq. yd (left for seven days)
   1st Seal Coat: 80/100 Pen: 0.22 glns/sq.yd with 3/4 inch aggregate at 55 sq.yds/cu yd.
   2nd Seal Coat: 80/100 Pen: 0.28 glns/sq.yd with 1/2 inch aggregate at 85 sq.yds/cu yd.
   3rd Seal Coat: 80/100 Pen: 0.10 glns/sq. yd. with 1/4 inch to dust at 300 sq.yds/cu yds.

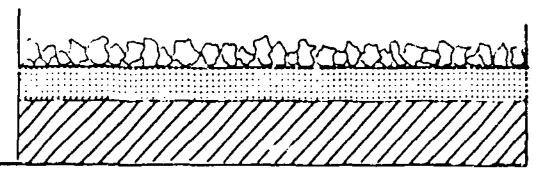
AGGREGATE – 12.5/18 mm 14 L/m$^2$
EMULSION 60 % 2.100 kg/m$^2$
OR
EMULSION 70 % 1.800 kg/m$^2$



AGGREGATE – 8/12.5 mm 9 L/m$^2$
EMULSION 60 % 1.500 kg/m$^2$
OR
EMULSION 70 % 1.300 kg m$^2$



AGGREGATE – 5/8 mm 6 L/m$^2$
EMULSION 60 % 1.200 kg/m$^2$
OR
EMULSION 70 % 1.050 kg/m$^2$



AGGREGATE 2.5/5 mm 4 L/m$^2$
EMULSION 60 % 0.900 kg/m$^2$
OR
EMULSION 70 % 0.800 kg/m$^2$

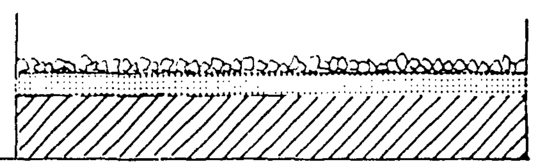


FIGURE 6.5 – SINGLE PASS SURFACE TREATMENT (After Pullen and Prevost 1969)

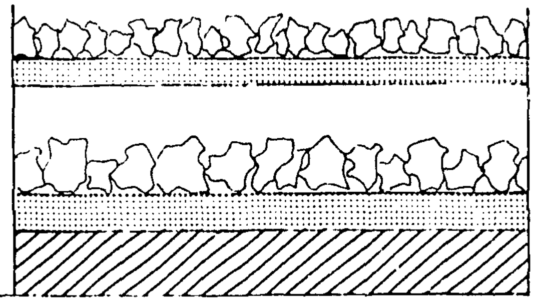
## TABLE 6.10

**Conversions for Figure 6.5**

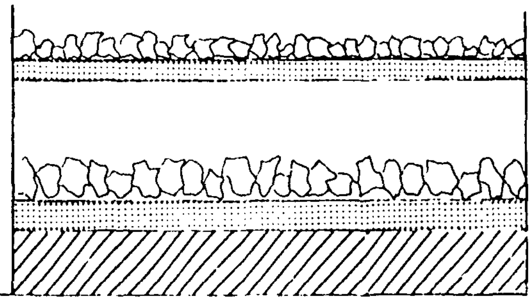
| | |
|---|---|
| 12.5/18 mm | # 1/2 / 0.709 in. |
| 14 l/m$^2$ | # 2.576 Imp. Gal. (3.08 US gal.) per sq. yd. |
| 14 kg/m$^2$ | # 2.576 Imp. Gal. (3.08 US gal.) per sq. yd. |
| 1.8 kg/m$^2$ | # 3.317 lbs/sq. yd. |
| 8/12.5 mm | # 0.315/ 1/2 in. |
| 9 l/m$^2$ | # 1.656 Imp. Gal. (2 US gal.) per sq. yd. |
| 1.5 kg/m$^2$ | # 2.765 lbs/sq. yd. |
| 1.3 kg/m$^2$ | # 2.400 lbs/sq. yd. |
| 5/8 mm | # 0.200/0.315 in. |
| 6 l/m$^2$ | # 1.104 Imp. Gal. (1.320 US gal.) per sq. yd. |
| 1.2 kg/m$^2$ | # 2.210 lbs/sq. yd. |
| 1.05 kg/m$^2$ | # 1.935 lbs/sq. yd. |
| 2.5/5 mm | # 0.100/0.200 in. |
| 4 l/m$^2$ | # 0.736 Imp. Gal. (0.880 US gal.) per sq. yd. |
| 0.9 kg/m$^2$ | # 1.660 lbs/sq. yd. |
| 0.8 kg/m$^2$ | # 1.475 lbs/sq. yd. |

US Gallon x 4.951 x 10$^{-3}$ = Cubic Yards

| | | | |
|---|---|---|---|
| AGGREGATE – | | 8/12.5 mm | 9 L/m² |
| | EMULSION 60 % | | 1.400 kg/m² |
| | OR | | |
| | EMULSION 70 % | | 1.200 kg/m² |
| AGGREGATE – | | 12.5/18 mm | 14 L/m² |
| | EMULSION 60 % | | 2.100 kg/m² |
| | OR | | |
| | EMULSION 70% | | 1.800 kg/m² |



| | | | |
|---|---|---|---|
| AGGREGATE – | | 5/8 mm | 6 L/m² |
| | EMULSION 60 % | | 1.100 kg/m² |
| | OR | | |
| | EMULSION 70 % | | 0.950 kg/m² |
| AGGREGATE – | | 8/12.5 mm | 9 L/m² |
| | EMULSION 60 % | | 1.400 kg/m² |
| | OR | | |
| | EMULSION | | 1.200 kg/m² |



| | | | |
|---|---|---|---|
| AGGREGATE – | | 2.5/5 mm | 4 L/m² |
| | EMULSION 60 % | | 0.800 kg/m² |
| | OR | | |
| | EMULSION 70 % | | 0.700 kg/m² |
| AGGREGATE – | | 5/8 mm | 6 L/m² |
| | EMULSION 60 % | | 1.100 kg/m² |
| | OR | | |
| | EMULSION 70 % | | 0.950 kg/m² |

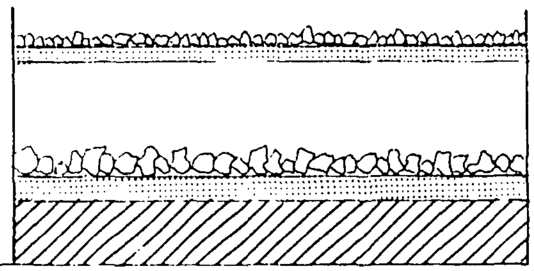


FIGURE 6.6 – DOUBLE SURFACE TREATMENT (After Pullen and Prevost 1969)

## TABLE 6.11

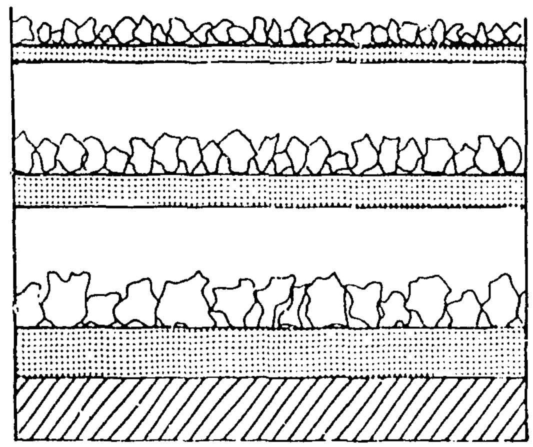
**Conversions for Figure 6.6**

| | |
|---|---|
| 8/12.5 mm | # 0.315 / 1/2 in. |
| 9 l/m² | # 1.656 Imp. Gal. (2 US gal.) per sq. yd. |
| 1.4 kg/m² | # 2.580 lbs/sq. yd. |
| 1.2 kg/m² | # 2.210 lbs/sq. yd. |
| 12.5/18 mm | # 1/2 / 0.709 in. |
| 14 l/m² | # 2.576 Imp. Gal. (3.08 US gal.) per sq. yd. |
| 2.1 kg/m² | # 3.870 lbs/sq. yd. |
| 1.8 kg/m² | # 3.317 lbs/sq. yd. |
| 5/8 mm | # 0.200/0.315 in. |
| 6 l/m² | # 1.104 Imp. Gal. (1.320 US gal.) per sq. yd. |
| 1.1 kg/m² | # 2.030 lbs/sq. yd. |
| 0.950 kg/m² | # 1.750 lb /sq. yd. |

| | |
|---|---|
| 8/12.5 mm | # 0.315 / 1/2 in. |
| 9 l/m² | # 1.656 Imp. Gal. (2 US gal.) per sq. yd. |
| 1.4 kg/m² | # 2.580 lbs/sq. yd. |
| 1.2 kg/m² | # 2.212 lbs/sq. yd. |
| 2.5/5 mm | # 0.100/0.200 in. |
| 4 l/m² | # 0.736 Imp. Gal. (0.880 US gal.) per sq. yd. |
| 0.8 kg/m² | # 1.475 lbs/sq. yd. |
| 0.7 kg/m² | # 1.290 lbs/sq. yd. |
| 5/8 mm | # 0.200/0.315 in. |
| 6 l/m² | # 1.104 Imp. Gal. (1.320 US gal.) per sq. yd. |
| 1.1 kg/m² | # 2.030 lbs/sq. yd. |
| 0.950 kg/m² | # 1.750 lbs/sq. yd. |

US Gallon x $4.951 \times 10^{-3}$ = Cubic Yards

| | | |
|---|---|---|
| AGGREGATE - | 5/8 mm | 6 L/m$^2$ |
| EMULSION 60 % | | 0.800 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 0.700 kg/m$^2$ |
| AGGREGATE | 8/12.5 mm | 9 L/m$^2$ |
| EMULSIONS 60% | | 1.300 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 1.150 kg/m$^2$ |
| AGGREGATE - | 12.5/18 mm | 14 L/m$^2$ |
| EMULSION 60 % | | 2. 000 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 1.700 kg/m$^2$ |



ALTERNATE

| | | |
|---|---|---|
| AGGREGATE - | 5/8 mm | 6 L/m$^2$ |
| EMULSION 60 % | | 1.600 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 1.400 kg/m$^2$ |
| AGGREGATE - | 8/12.5 mm | 9 L/m$^2$ |
| EMULSION 60 % | | 1.400 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 1.200 kg/m$^2$ |
| AGGREGATE | 12.5/18 mm | 14 L/m$^2$ |
| EMULSION 60 % | | 1.100 kg/m$^2$ |
| OR | | |
| EMULSION 70 % | | 0.950 kg/m$^2$ |

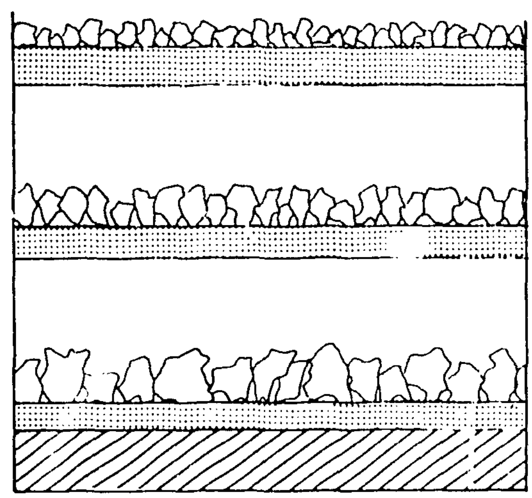


FIGURE 6.7 – TRIPLE SURFACE TREATMENT (After Pullen and Prevost 1969)

## TABLE 6.12

**Conversions for Figure 6.7**

| | | | |
|---|---|---|---|
| 5/8 mm | # 0.200/0.315 in. | 6 l/m$^2$ | # 1.104 Imp. Gal. (1.320 US gal.) per sq. yd. |
| 6 l/m$^2$ | # 1.104 Imp. Gal. (1.320 US gal.) per sq. yd. | 1.6 kg/m$^2$ | # 2.950 lbs/sq. yd. |
| 0.8 kg/m$^2$ | # 1.475 lbs/sq. yd. | 1.4 kg/m$^2$ | # 2.580 lbs/sq. yd. |
| 0.7 kg/m$^2$ | # 1.290 lbs/sq. yd. | | |
| 8/12.5 mm | # 0.315 / 1/2 in. | 8/12.5 mm | # 0.315 / 1/2 in. |
| 9 l/m$^2$ | # 1.656 Imp. Gal. (2 US gal.) per sq. yd. | 9 l/m$^2$ | # 1.656 Imp. Gal. (2 US gal.) per sq. yd. |
| 1.3 kg/m$^2$ | # 2.400 lbs/sq. yd. | 1.4 kg/m$^2$ | # 2.580 lbs/sq. yd. |
| 1.150 kg/m$^2$ | # 2.120 lbs/sq. yd. | 1.2 kg/m$^2$ | # 2.210 lbs/sq. yd. |
| 12.5/18 mm | # 1/2 / 0.709 in. | 12.5/18 mm | # 1/2 /0.709 in. |
| 14 l/m$^2$ | # 2.576 Imp. Gal. (3.08 US Gal.) per sq. yd. | 14 l/m$^2$ | # 2.576 Imp. Gal. (3.08 US gal.) per sq. yd. |
| 2 Kg/m$^2$ | # 3.685 lbs/sq. yd. | 1.1 kg/m$^2$ | # 2.030 lbs/sq. yd. |
| 1.7 kg/m$^2$ | # 3.135 lbs/sq. yd. | 0.950 kg/m$^2$ | # 1.750 lbs/sq. yd. |
| 5/8 mm | # 0.200/0.315 in. | | |

US Gallon x $4.951 \times 10^{-3}$ = Cubic Yards

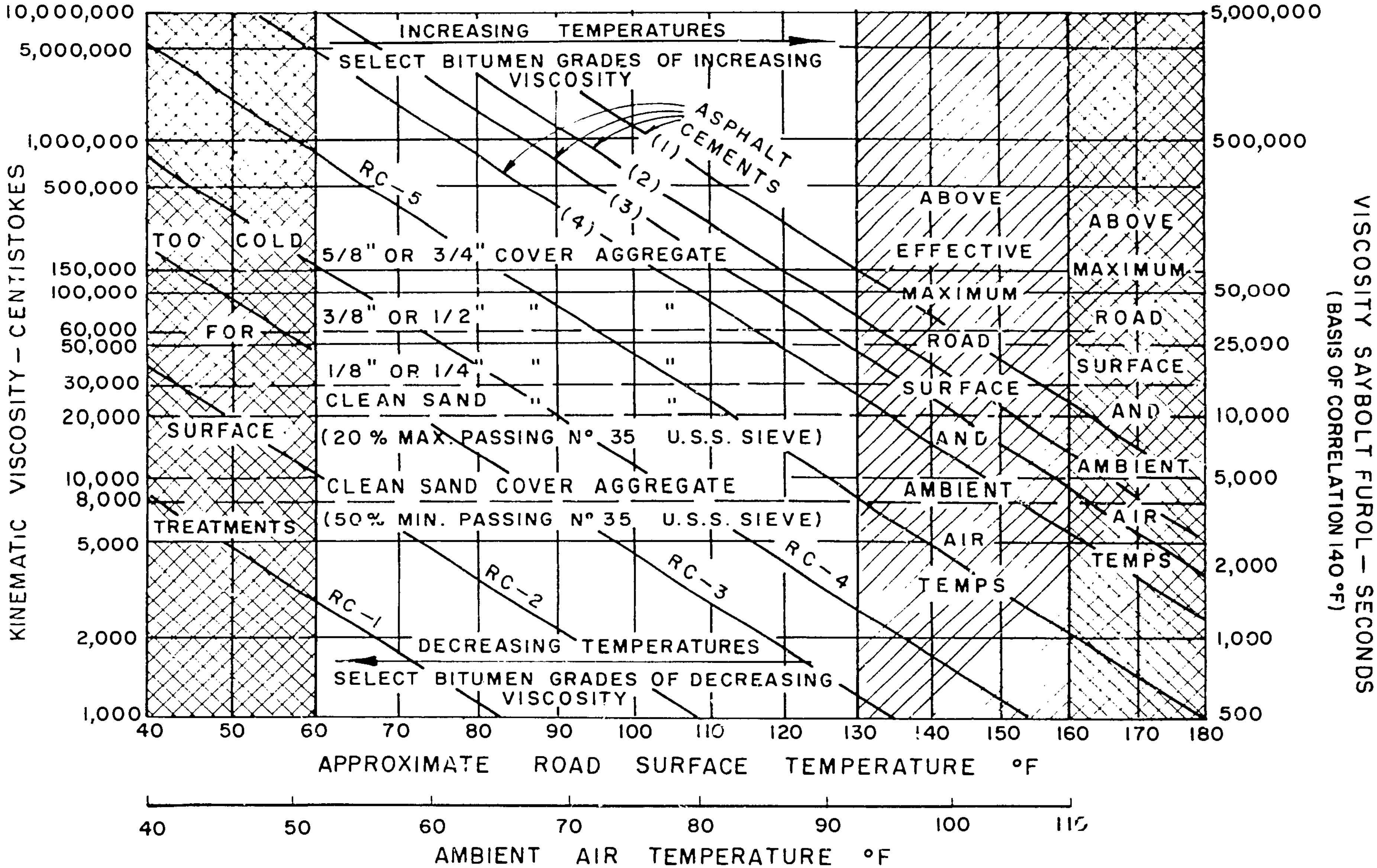



FIGURE 6.8a — SELECTION OF BITUMINOUS BINDER FOR SURFACE TREATMENTS OR SEAL COATS IN TROPICAL CLIMATES DETERMINED BY SURFACE TEMPERATURES AND SIZE OF COVER AGGREGATE (After Pullen an Prevost 1969)

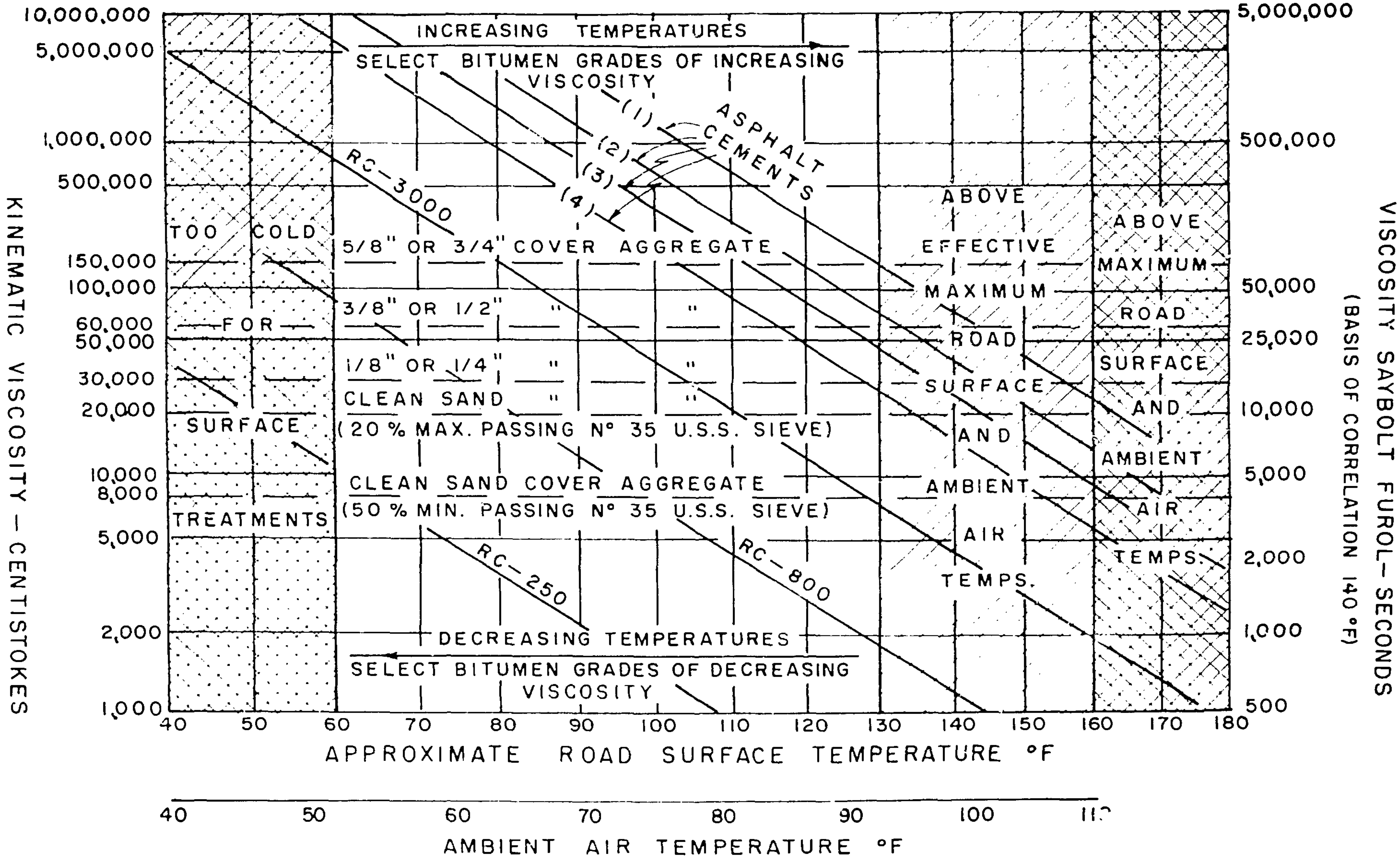



FIGURE 6.8b – SELECTION OF BITUMINOUS BINDER FOR SURFACE TREATMENTS OR SEAL COATS IN TROPICAL CLIMATES DETERMINED BY SURFACE TEMPERATURES AND SIZE OF COVER AGGREGATE (After Pullen and Prevost 1969)

## TABLE 6.13

**Spreading Temperatures for Cutback Asphalt (After Pullen and Prevost 1969)**

| U.S. Grade Cutbacks RC–MC | Kinematic Viscosity Centistokes at 140°C | Recommended Spraying Temperature Ranges °F and °C Low Viscosity / High Viscosity |
|---|---|---|
| 30 | 30 – 60 | 83–125°F/118–158°F<br>28–52°C/48 – 70°C |
| 70 | 70 – 140 | 120–165°F/145–190°F<br>49–74°/63 – 88°C |
| 250 | 250 – 500 | 162–208°F/183–234°F<br>72–98°C/84–112°C |
| 800 | 800 – 1,600 | 197–247°F/218–268°F<br>92–119°C/103–131°C |
| 3,000 | 3,000 – 6,000 | 233–283°F/249–300°F<br>112–139°C/120–149°C |

## TABLE 6.14

**Spreading Temperatures for Emulsified Asphalts Recommended by the Asphalt Institute**

| | | |
|---|---|---|
| Anionic Emulsions | | |
| RS-1 | 75 – 130°F | 24 – 55°C |
| RS-2 | 110 – 160°F | 43 – 71°C |
| MS-2 | 100 – 160°F | 38 – 71°C |
| SS-1 | 75 – 130°F | 24 – 55°C |
| SS-1 h | 75 – 130°F | 24 – 55°C |
| Cationic Emulsions | | |
| RS-2K | 75 – 130°F | 24 – 55°C |
| RS-3K | 110 – 160°F | 43 – 71°C |
| CM-K | 100 – 160°F | 38 – 71°C |
| SM-K | 100 – 160°F | 38 – 71°C |
| SS-K | 75 – 130°F | 24 – 55°C |
| SS-K h | 75 – 130°F | 24 – 55°c |