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Construction Inspection Techniques for Base Course Construction

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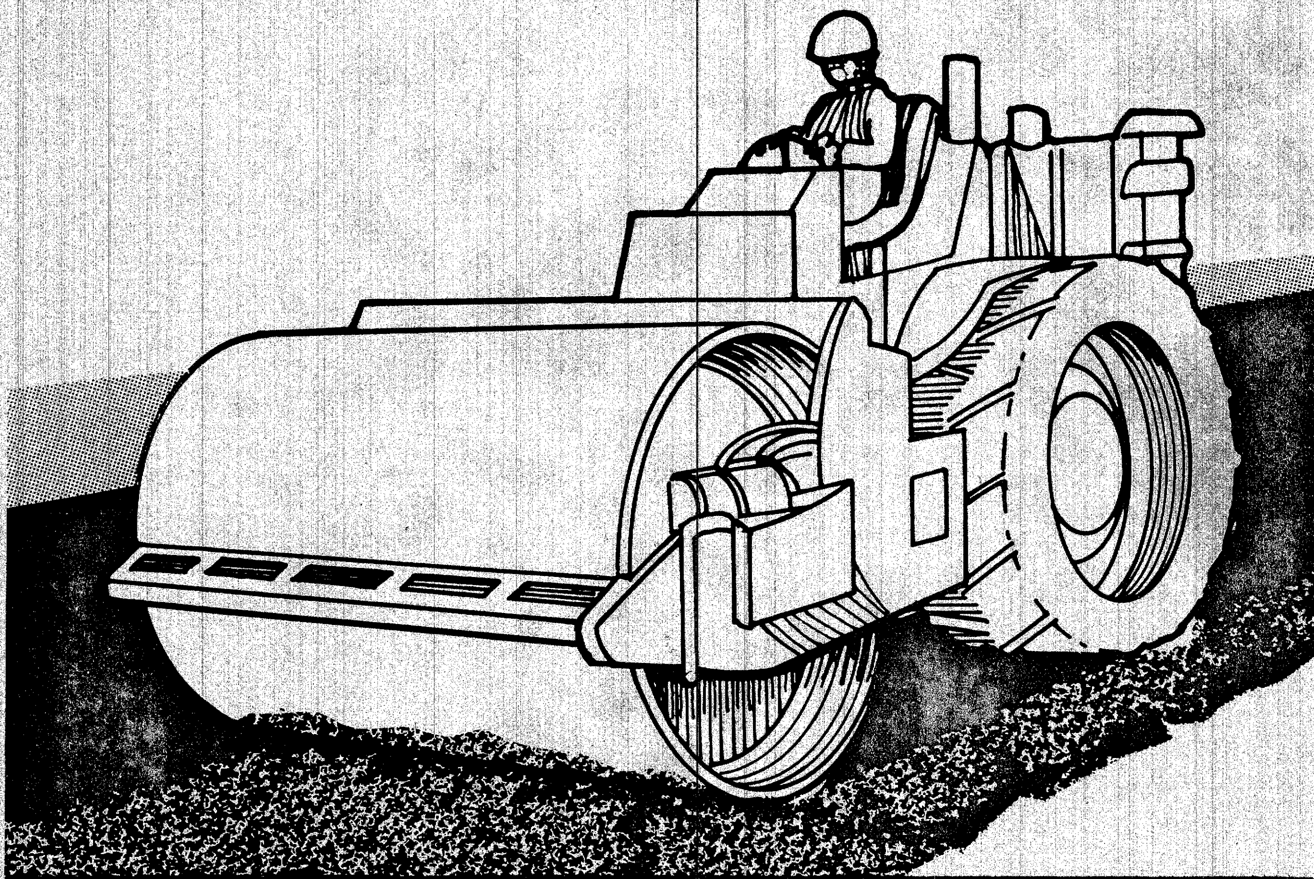

Prepared By
Construction and
Maintenance Division

CONSTRUCTION INSPECTION TECHNIQUES

for

BASE COURSE CONSTRUCTION

Prepared by

Construction and Maintenance Division

May 1986

Federal Highway Administration

U.S. Department of Transportation

BASE COURSE CONSTRUCTION

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1.0 INTRODUCTION

1.1 DEFINITION/FUNCTION

AASHTO, M146, Terms Relating to Subgrade, Soil-Aggregate, and Field Materials defines "Base" as "the layer used in a pavement system to reinforce and protect the subgrade or subbase." It, therefore, is the layer of material which lies immediately below the wearing surface. This definition applies whether the wearing surface is bituminous or concrete.

Because the base course lies close under the pavement surface, it is subject to severe loading. It follows that the materials in a base course must be of high quality and subjected to quality specifications and careful construction.

The properties and characteristics of a pavement base course depend on its function. In turn, function depends on pavement type and environmental condition. The general functions of a base are:

- o structural capacity
- o prevention of pumping
- o protection against frost action
- o drainage
- o prevention of subgrade volume change
- o expedition of construction

Providing structural capacity as part of a roadway's design section is a base course's primary and most important function. With heavy wheel loads of today's traffic, the stresses imposed on a base course are high and these stresses must be resisted partially by the structural capacity of the base material.

Pumping is a phenomenon associated with concrete pavements and involves the creation of a void at slab joints, water infiltration into the roads, and expulsion of water and fines. The process is progressive, causing a larger and larger void to be created under the leave slab. Prevention of water entering the base, ensuring a free-draining base material and proper lateral drainage, or stabilization of the base are ways to assist in preventing pumping.

Frost action is a complex process caused by freezing temperature, accumulation of water, subsequent ice expansion, and the formations of ice lenses. This expansion causes heaving of the roadway and subsequent failure after thawing. The major factor in its occurrence is the frost susceptibility of the subgrade soil and base course largely dictated by permeability or the amount of fine material.

Proper drainage of the roadway section is very important to allow the removal of water, which is damaging to the stability of subgrade material. In the case of a permeable base, good drainage provides for moisture to drain through the base and adjacent shoulder into the roadway ditches or subsurface drainage system. In the case of dense graded impermeable bases, good drainage consists of removal of surface water and provision for removal of infiltrated water through the shoulders and/or longitudinal drains so it is not trapped in the base.

Some subgrade soils are very susceptible to volume change due to moisture infiltration which often permeates downward through the roadway surface.

Dense graded impermeable base material seals or protects such moisture susceptible soil from moisture damage. In addition, base material should not be susceptible to small changes by limiting the material properties.

Although seldom a primary function, a stable, dense, compacted base course provides a firm pavement layer for construction equipment to ride on, thereby preventing excess subgrade damage prior to final paving. It also may function as a temporary protection layer against weather factors when surface construction is delayed or done in stages.

Obviously not all base courses will need or can have all these functions at once. The desirability of importance of each of these individual functions will vary depending on the type of pavement and environmental conditions as will be discussed.

1.2 STRENGTH MEASUREMENT

From time-to-time the terms high quality, structural strength, or resistance to deformation under load will be used in reference to bases. What we are primarily referring to when using these terms is the stability or strength of the base course material. Several tests have been developed over the years to measure stability.

Stability is normally not directly measured in the field to evaluate a base course. It is primarily a laboratory test used to evaluate a material and the results are used in design. However, since base course stability is the most important factor of a functioning base, it is important to understand it and how it relates to field control measurements/tests.

Among highway engineers, a commonly used test and that referred to in this manual is the California Bearing Ratio (CBR) Test (AASHTO T193). The CBR Test is performed in general as follows and as depicted in Figure No. 1-1.

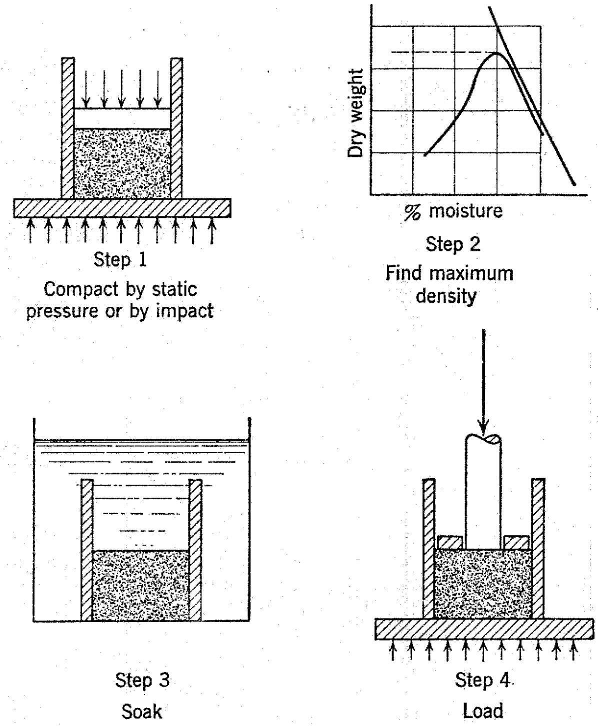


FIGURE NO. 1-1 - Laboratory Procedure For Finding The CBR Of A Soil

STEP 1. Disturbed samples of material at different moisture contents, are compacted by a static load or impact hammer into cylindrical steel molds 6 inches in diameter and 8 inches tall. The resulting specimen depth is about 4 inches.

STEP 2. The moisture-density curve is plotted, and the sample with greatest dry-density is selected.

STEP 3. The specimen, still in the mold, is immersed in water and soaked for 4 days to simulate saturation that may occur in service.

STEP 4. A small cylindrical piston (2" diameter) is forced into the still confined test specimen. Load deformation data is gathered as the specimen is penetrated.

From load deformation data, the CBR ratio (%) reflecting the bearing strength is calculated from the following formula:

$$\text{CBR}(\%) = \frac{\text{LOAD CARRIED BY TEST SPECIMEN @ 0.1 PENETRATION}}{\text{LOAD CARRIED BY STANDARD CRUSHED ROCK BASE @ 0.1 PENETRATION}}$$

CBR load-deformation curves for a variety of soil materials are shown in Figure No. 1-2. Thus, the CBR percent indicates the quality of the materials in terms of that of an excellent crushed-stone base course, which has a CBR of 100.

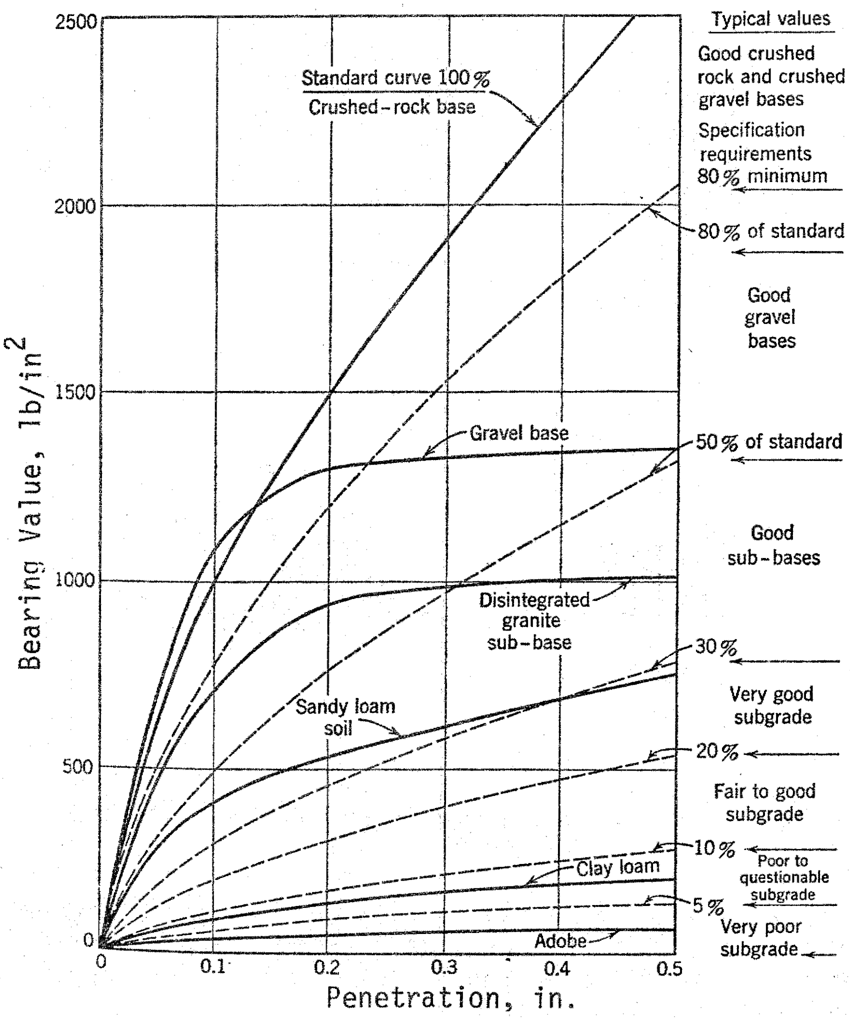


FIGURE NO. 1-2 - Load Penetration Curves For Typical Soils Tested By the CBR Method

1.3 ROADWAY LOAD TRANSFER

The function of any roadway design section and, therefore, the base is to reduce and distribute wheel loads so that stresses at the subgrade level can be withstood by the existing material. Flexible asphalt and concrete rigid pavements transfer or distribute loadings differently and, thus, are designed according to different criteria.

Flexible roadways consist of a relatively thin riding surface built over a base course and subbase on a compacted subgrade. Figure No. 1-3 shows a typical flexible pavement cross-section which depicts the thin riding surface relative to the total pavement section. For structural design purposes, the thickness of the flexible roadway is meant to include all components of the pavement cross-section above the compacted subgrade. Thus, the subbase, base course, and riding surface are the structural design components of the flexible pavement.

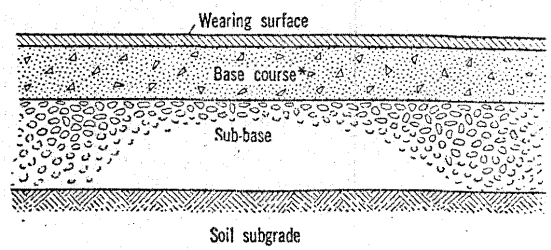


FIGURE NO. 1-3 - Typical Flexible Pavement Section

The strength of a flexible pavement is due to building up of thick layers which distribute the load over the existing subgrade. Because the asphalt wearing surface is relatively thin, its structural contribution is relatively small. It primarily provides a smooth, dustless, impervious cover over the pavement base and protects it from excessive wetting and drying and traffic abrasion. For this reason, the base course in a flexible pavement becomes a main structural element.

In addition, because asphalt pavements are flexible and deform locally under load, a load acts as a point load and stresses in the under-layers are distributed and reduced according to the depth below the surface and radial distance from the assumed point load. For this reason, and the structural weakness of the thin surface course, a base course's strength or stability is of paramount importance in flexible pavements and other factors are generally secondary.

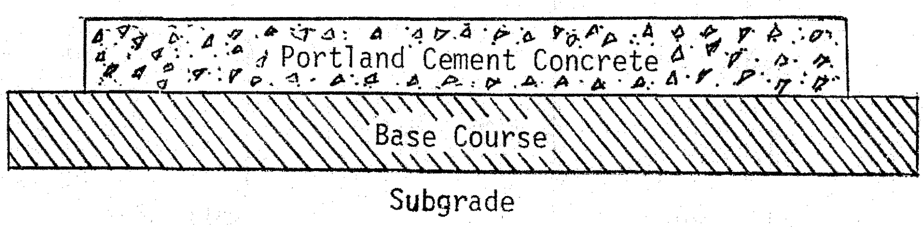


FIGURE NO. 1-4 - Typical Concrete Pavement Section

Rigid concrete pavements are made up of a concrete layer and generally a base course layer on the compacted subgrade. Figure No. 1-4 shows a typical concrete pavement cross-section and depicts the thickness of the concrete riding surface relative to the total pavement cross-section. The concrete slab pavement, because of its rigidity and bending action, distributes the load to the base course over a relatively wide area due to the slab or beam action. Thus, almost all the structural load capacity is supplied by the slab itself.

The major strength factor in concrete pavement design is the flexural strength of the concrete making it act as a larger slab. Because of this wide area of load distribution, the base course material under the slab is subjected to low stresses and base course strength or stability is of less importance than with flexible pavements. For this reason, the other factors such as pumping control, frost action control, shrinkage and swell control, and of construction expediency are often more important in concrete pavement base course design. In concrete roadways, variations in base and subgrade strength have little influence on the roadway structural capacity and the importance of base course strength uniformity is not as great as in a flexible pavement.

1.4. PARTICLE GRADATION NOMENCLATURE

Soil materials are classified by particle size and defined by certain nomenclature. It is important to understand and define such nomenclature according to particle size in order to understand base course material characteristics.

AASHTO Standard Specification M147, Materials for Aggregate and Soil-Aggregate Subbase, Base, and Surface Course and many State specifications designate the 1", 3/8", and No. 4, 10, 40, and 200 sieves when defining base material gradation. Utilizing these sieve sizes, the following material size definitions are given in AASHTO M146, Standard Definitions of Terms Relating to Subgrade, Soil-Aggregates, and Fill Materials:

TABLE NO. 1-1 - Soil-Aggregate Particle Definitions

Definition	Sieve Size
STONE	> No. 10
(Coarse: 1" - 3")	
(Medium: 3/8" - 1")	
(Fine: No. 10 - 3/8")	
SAND	No. 200 - No. 10
(Coarse: No. 40 - No. 10)	
(Fine: No. 200 - No. 40)	
SILT AND CLAY	< No. 200

Throughout the text, reference will be made to the properties and characteristics imparted to a base material by certain components. It is, therefore, important to designate these components and understand how they affect the behavior or properties of a granular base course material. Table No. 1-2 indicates these 3 components and their respective sieve designation.

TABLE NO. 1-2 - Base Course Component Designations

COMPONENT	SIEVE SIZE
COARSE	> No. 200
FINES	< No. 200
BINDER	< No. 40

The coarse component, being greater than the No. 200 sieve, includes gravel, coarse sand, and fine sand. In a well graded granular base course material, this coarser material furnishes bearing strength through point-to-point contact and internal friction. It also furnishes hardness which is dependent on the coarse material properties.

The fines component is that material passing the No. 200 sieve. Such sized material is composed of silt and clay which provides cohesion to a base course material and also imparts other properties.

The binder component is that size material which passes the No. 40 sieve and includes the fines component. Binder material includes fine sand, silt, and clay. It serves to fill the voids in the coarse material contributing somewhat to internal friction strength. It also binds the coarser material together. Certain tests are performed on the binder component which define the plasticity of the material.

While these component designations are generally used in performing tests and in discussing granular material characteristics in theory, variations in the sieve designations are often used in field application and discussions. For example, coarse material may be referenced as that size material greater than the No. 200, 4, 8, or 10 sieve in actual field application.

1.5 SOIL-AGGREGATE CLASSIFICATION SYSTEM

In order to have a common understanding between exploration, design, and construction personnel, it is essential to have a uniform soil designation system. Two of the most widely used systems are the AASHTO and ASTM (Unified) classification systems. The AASHTO system is commonly used by a majority of SHA's for pavement related purposes and will be covered here.

In 1931, the then Bureau of Public Roads adopted a soil classification system known as the original BPR system which attempts to group soils in the order of performance under the direct action of wheel loads. In 1947, this system was adopted by AASHTO and today is contained in AASHTO Specification M145 Classification of Soils and Soil Aggregate Mixtures for Highway Construction Purposes.

It classifies soils according to their strength and suitability for highway subgrade purposes into seven "A" groups (as shown in Table No. 1-3) each with uniform properties as defined by sieve gradation and characteristics of the binder course. It is not intended to get into soil classification and materials properties in great detail except to indicate several important properties which are worth noting. The most important and obvious is that there are seven AASHTO soil-aggregate groups A-1 through A-7.

Table No. 1-3 defines a granular material as one with 35 percent or less passing the No. 200 sieve and designates A-1, A-2, and A-3 materials as granular. Figure No. 1-5 shows the general relationship between the amount of coarse gravel material and density. It indicates that as the percent of gravel increases the density and, therefore, the stability of a material increases. Since granular materials (higher gravel percent) have higher stability, they are more suitable for highway base course as will be discussed later in this course.

The A-1 through A-3 granular materials are further defined and subdivided according to the amount and suitability of the binder material which binds and provides cohesion to the coarse material. Thus, A-1 material is superior to A-2 and A-3 materials as a base coarse because it has a low amount of fine material with suitable plasticity. This point will be discussed later in more detail.

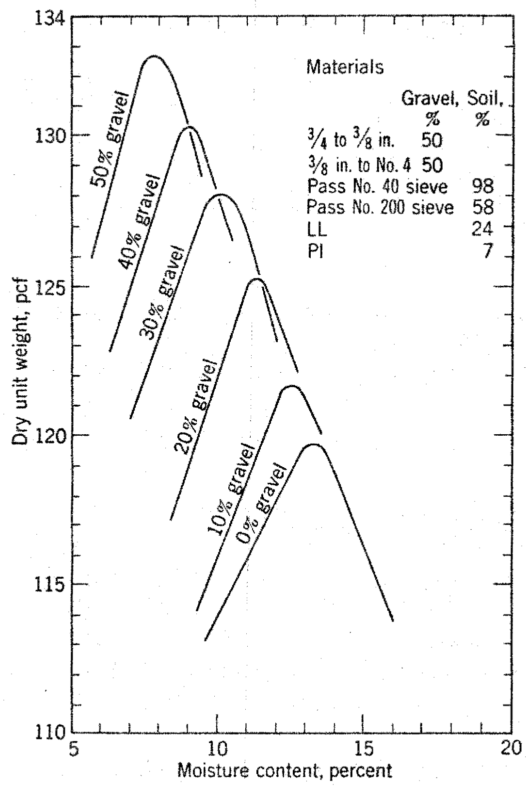


FIGURE NO. 1-5 - Gravel Content Vs. Density And Moisture Content Compaction According To T99

Table No. 1-3 defines silty-clay materials as those with greater than 35 percent passing the No. 200 sieve and designates them as A-4, A-5, A-6 or A-7 materials. A silty-clay material causes the smaller amount of coarse material to float in the finer material which tends to destroy aggregate interlock as well as, in some cases, have very poor plasticity properties. They are, therefore, not suitable for base course or subbase use.

TABLE NO. 1-3 - Classification Of Soils And Soils Aggregate Mixtures

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silty-lay Materials (More than 35% passing 0.075mm)			
	A-1			A-2							
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7 A-7-5 A-7-6
Sieve Analysis, Percent passing:											
2.00mm (No. 10).....................	50 max.	.....	...	.....	.....	.....	.....	.....	.....	.....	.....
0.425mm (no. 40).....................	30 max.	50 max.	51 max.	.....	.....	.....	.....	.....	.....	.....	.....
0.075mm (No. 200)...................	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	35 min.	36 min.	36 min.	36 min.
Characteristics of Fraction passing 0.425mm (No. 40)											
Liquid limit........................	.....	.....	.....	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plastic index.......................	6 max.		N.P.	10 max.	10 max.	11 min.	11 mjn.	10 max.	10 max.	11 min.	11 min.
Usual Types of Significant Constituent Materials.................	Stone Fragments, Gravel & Sand		Fine Sand	Silty or Clayey Gravel or Sand				Silty Soils		Clayey Soils	
General Rating as Sugrade............	Excellent to Good							Fair to Poor			

The important thing to note in the AASHTO classification system is that as the numerical designation increases, generally speaking, the structural capability of the material to function as a highway base course material decreases. Thus, an A-1 material is superior to the others and as the numerical designation increases (A-2 ---- A-7), the material becomes of poorer quality. An A-7 material which is the poorest quality is a highly plastic sticky clay. Figure No. 1-6 indicates the suitability of the various A soil classifications for the roadway elements. Note that A-1-a, A-1-b, a-2-4, A-2-5, and A-3 can be acceptable for base with a small amount of binder material. As will be discussed later, in the stabilized soil section, the lower quality materials should not be used unless soil stabilization is employed.

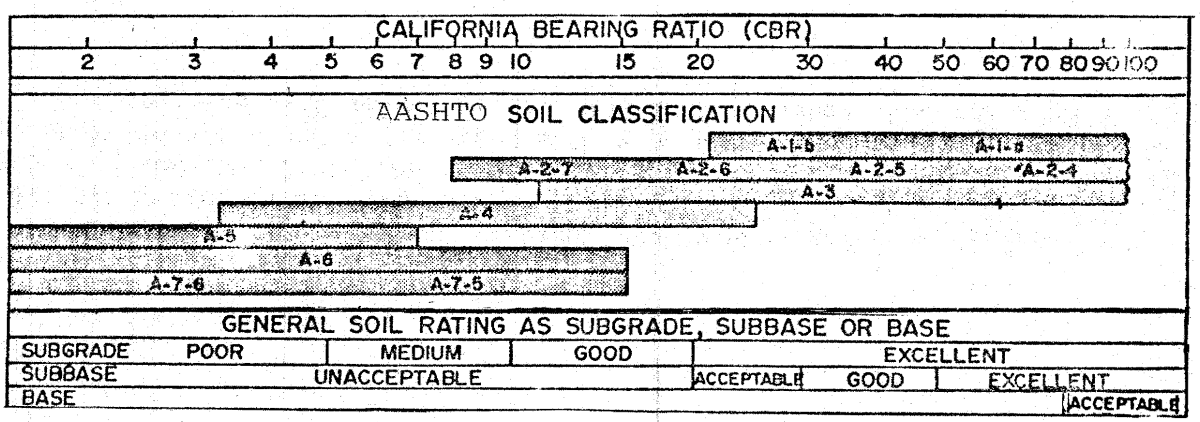


FIGURE NO. 1-6 - Suitability Of Soil For Roadway Elements

1.6 STANDARD GRADATION NOMENCLATURE

In the early 1960's, studies indicated great lack of uniformity among the States in the number and size of sieves used and in the designations of aggregate gradations. As a result, a simple and uniform system for identification of standard aggregate gradations was developed. Known as the Simplified Practice Recommendations (SPR), the system is contained in **AASHTO M43 "Standard Size of Coarse Aggregate for Highway Construction."** This system is not universally used by all SHD's but has gained wide enough use to warrant knowledge of it.

The SPR gradings result in a readily understandable system of individual size and grading designations consisting basically of a single digit number. The single digit number series starts with No. 1 for the standard commercial aggregate having the largest top-size particles and progresses from No. 1 through No. 9 as the individual standard course aggregates decrease in size as shown in Table No. 1-4.

TABLE NO. 1-4 - Simplified Practice Recommendations Numbering System

Basic SPR designations	Combinations of basic designations	Nominal Size		Size Limits	
		Maximum	Minimum	Maximum	Minimun
1		3-1/2 in.	1-1/2 in.	4 in.	3/4 in.
2		2-1/2 in.	1-1/2 in.	3 in.	3/4 in.
3		2 in.	1 in.	2-1/2 in.	1/2 in.
	357				
4		1-1/2 in.	3/4 in.	2 in.	3/8 in.
	467				
5		1 in.	1/2 in.	1-1/2 in.	3/8 in.
	56				
	57				
6		3/8 in.	3/8 in.	1 in.	No. 4
	67				
	68				
7		1/2 in.	No. 4	3/4 in.	No. 8
	78				
8		3/8 in.	No. 8	1/2 in.	No. 16.
9		No. 4	No. 16	3/8 in.	No. 50

Because of the consistent demands for certain longer gradings than the relatively short ones represented by the basic No. 1 No. 9 series, shown in the first column of Table No. 1-4, a secondary grading series was developed by combining the basic gradings. These combinations of the basic grading are identified by corresponding combinations of the single digit numbers.

Thus, standard aggregate No. 357, shown in the second column of the Table No. 1-4 which immediately follows No. 3 is a combination of standard sizes No's. 3, 5, and 7 in such proportion as to conform to the grading band limits. Table No. 1-5 indicates the standard sizes of coarse aggregates in AASHTO M43.

TABLE NO. 1-5 - Standard Sizes Of Coarse Aggregate

Size Number	Nominal Size, Square Openings	Amounts Finer than Each Laboratory Sieve (Square Openings), weight percent														
		4-in. (100-mm)	3½-in. (90-mm)	3-in. (75-mm)	2½-in. (63-mm)	2-in. (50-mm)	1½-in. (37.5-mm)	1-in. (25.0-mm)	¾-in. (19.0-mm)	½-in. (12.5-mm)	⅜-in. (9.5-mm)	No. 4 (4.75-mm)	No. 8 (2.36-mm)	No. 16 (1.18-mm)	No. 50 (300-μm)	No. 100 (150-μm)
1	3½ to 1½-in. (90 to 37.5-mm)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5	...	...	...	...	...	...	...
2	2½ to 1½-in. (63 to 37.5-mm)	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...	...
24	2½ to ¾-in. (63 to 19.0-mm)	...	...	100	90 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...	...	...	...
3	2 to 1-in. (50 to 25.0-mm)	...	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...
357	2-in. to No. 4 (50 to 4.75-mm)	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5	...	...	...	...
4	1½ to ¾-in. (37.5 to 19.0-mm)	...	...	...	...	100	90 to 100	20 to 55	0 to 15	...	0 to 5	...	...	...	...	...
467	1½-in. to No. 4 (37.5 to 4.75-mm)	...	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5	...	...	...	...
5	1 to ½-in. (25.0 to 12.5-mm)	...	...	...	...	...	100	90 to 100	20 to 55	0 to 10	0 to 5	...	...	...	...	...
56	1 to ⅜-in. (25.0 to 9.5-mm)	...	...	...	...	...	100	90 to 100	40 to 85	10 to 40	0 to 15	0 to 5	...	...	...	...
57	1-in. to No. 4 (25.0 to 4.75-mm)	...	...	...	...	...	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...
6	¾ to ⅜-in. (19.0 to 9.5-mm)	...	...	...	...	...	...	100	90 to 100	20 to 55	0 to 15	0 to 5	...	...	...	...
67	¾-in. to No. 4 (19.0 to 4.75-mm)	...	...	...	...	...	...	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...	...	...
68	¾-in. to No. 8 (19.0 to 2.36-mm)	...	...	...	...	...	...	100	90 to 100	...	30 to 65	5 to 25	0 to 10	0 to 5	...	...
7	½-in. to No. 4 (12.5 to 4.75-mm)	...	...	...	...	...	...	...	100	90 to 100	40 to 70	0 to 15	0 to 5	...	...	...
78	½-in. to No. 8 (12.5 to 2.36-mm)	...	...	...	...	...	...	...	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	...	...
8	⅜-in. to No. 8 (9.5 to 2.36-mm)	...	...	...	...	...	...	...	...	100	85 to 100	10 to 30	0 to 10	0 to 5	...	...
89	⅜-in. to No. 16 (9.5 to 1.18-mm)	...	...	...	...	...	...	...	...	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	...
9	No. 4 to No. 16 (4.75 to 1.18-mm)	...	...	...	...	...	...	...	...	...	100	85 to 100	10 to 40	0 to 10	0 to 5	...
10	No. 4 to 0[A] (4.75-mm)	...	...	...	...	...	...	...	...	...	100	85 to 100	...	...	...	10 to 30

[A] Screenings

2.0 BASE COURSE TYPES

It is generally considered that there are three different categories of base courses. These are macadam, granular, and stabilized bases.

2.1 MACADAM BASE COURSES

Macadam was the base type used since the early historic efforts of systematic road design and construction. Today, some States specify macadam base material and the term is used from time to time. Macadam is a layered base system with each layer of generally uniform size stone. Each subsequent layer is of a smaller size stone worked into the larger stone layer below.

The term macadam indicates a base course layer of crushed stone of relatively large size (4-inch max.) to which is added, during construction, small stone particles (1-inch max.) or screenings called "choke" which are then worked down into the mass by rolling. The material is taken directly from the crushing process with screens splitting the crushed material into the appropriate size fraction. Macadam base courses are generally considered pervious to water infiltration. The use of large top size aggregate bound or sealed with choke helps surface stability and reduces damage by paving equipment operations.

Dry bound macadam denotes a stone base course which is compacted through rolling or vibration only. Water bound macadam indicates that water is added after addition of screenings and then the entire layer rolled.

2.2 GRANULAR BASE COURSES

From a design standpoint, there are two types of granular base courses which vary only in gradation design. The two types are open-graded and dense-graded base courses.

2.2.1 Open Graded

Open graded base courses are composed of largely coarse material with the amount of fines being limited. Such a material allows for the free passage of water or moisture through the subgrade, therefore, called free draining. Base courses may fail due to water moisture intrusion downward through the pavement surface or upwards through the subgrade due to capillary action. In either case, the result is a weakening or loss of stability of the base due to washing out of the fine material or increasing of the moisture content.

Where base course moisture is expected to be a problem, an open-graded base material may be utilized instead of a dense graded base course. The disadvantage in their use is that open-graded bases have lower stability which must be considered in the pavement's design, requires positive drainage systems, and are susceptible to infiltration clogging. They are also much more susceptible to damage by construction vehicle traffic and environmental deterioration; therefore, they cannot be left exposed for long periods and often must be stabilized with asphalt as depicted in Figure No. 2-1, open-graded bases should not be placed directly on fine subgrade soils. A firm dry subgrade may become wet under pavement possibly due to reduced evaporation and intrude the open graded under action of traffic. Therefore, a properly designed layer of fine aggregate sand or a synthetic filter blanket should be placed between the subgrade and an open-graded base. It is important, however, that provisions be made for water flowing through the layers to get out as quickly as it gets in.

For highway pavements, the design should required that the base or subbase be extended for the full width of the pavement or that there be a longitudinal drainage system with well designed drainage ditches large enough to carry the discharge rapidly. Open graded bases, along with their associated drainage systems, may be more difficult to construct.

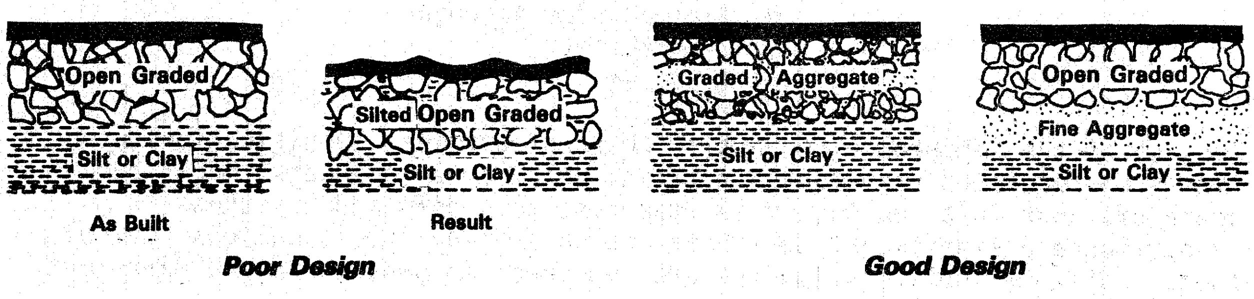


FIGURE NO. 2-1 - Open Graded Base Courses

2.2.2 Dense-Graded

Dense-graded base courses are composed of uniformly graded coarse and fine material containing a sufficient amount of fines to fill the coarse material's voids and bind the material together into a dense cohesive mass. Such a material has high density and stability when compacted and is, therefore, highly impervious to moisture. Its long-term satisfactory function is dependent on maintaining prevention of water intrusion up from the subgrade or down through the pavement layer. Such water intrusions may cause loss of fines resulting in the loss of stability.

2.3 STABILIZED BASE COURSES

In its broadest sense, base course stabilization involves a chemical or mechanical treatment designed to increase or maintain the stability and properties of a material which otherwise might not be usable. Thus, in the broadest sense, a modification of gradation or even density compaction can be considered stabilization. However, for the purpose of this material, stabilization refers only to chemical stabilization and will be treated separately in another section.

3.0 IMPORTANT BASE CHARACTERISTICS

3.1 DENSITY/STABILITY RELATIONSHIP
3.2 GRADATION DISTRIBUTION
3.3 PERCENT FINES
3.4 MOISTURE CONTENT
3.5 MAXIMUM SIZE AGGREGATE
3.6 PLASTICITY INDEX
3.7 AGGREGATE HARDNESS
3.8 PERMEABILITY
3.9 QUALITY FACTOR INSPECTION PRIORITY

The following properties are considered significant in a base course and should be of interest to a field engineer:

- Stability Density
- Gradation
- Percent Fines
- Percent Moisture
- Plasticity
- Maximum Size Aggregate
- Permeability
- Hardness

Because of the importance of most of these properties, specifications place limits on furnished base course material which area engineers should be aware of. For this reason, it is important to understand why these properties are significant to a base course and why specifications control them. This will better allow the area engineer to understand and carry out his duties.

Since a base course lies close to the pavement surface, it must possess high stability in order to withstand the wheel loadings imposed on it. Without adequate stability the base will fail and, thus, the roadway will fail over time. For a given grain size distribution, stability is related to density. The actual relationship between density and stability can vary widely depending on the material gradation, type, and moisture content. However, for a well graded granular material there is a more definite relationship.

3.1 DENSITY/STABILITY RELATIONSHIP

As indicated in Figure No. 3-1, as the density of a well graded granular material increases, the stability increases. The relationship between stability and density is non-linear and the rate of change in stability is much more pronounced above about 130 pounds per cubic foot dry density which is common for most well-graded granular material. This points out the importance of adequate compaction and field density tests especially at the higher densities at which granular road base courses are constructed in assuring proper stability. At the higher densities, small changes in density will cause much larger changes in stability. High stability is what we are trying to achieve in most cases.

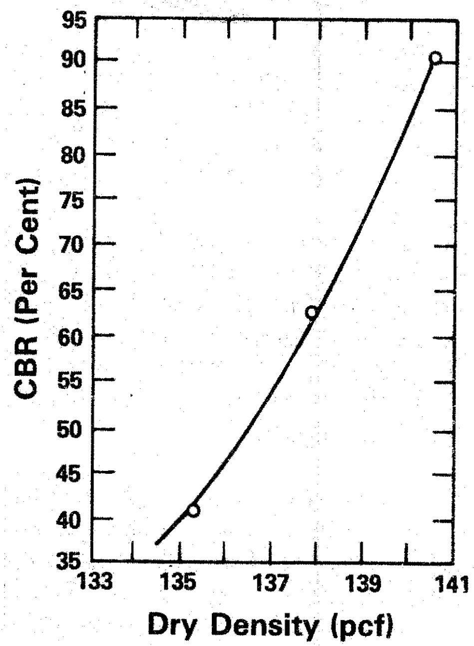


FIGURE NO. 3-1 - Variation of Stability with Density

All base courses will continue to consolidate under continuous traffic loading over time and it is important to reduce the potential for such progressive consolidation. Such in-place consolidation increases with decreased base material placement density as shown in Figure No. 3-2 illustrating the importance of high uniform density.

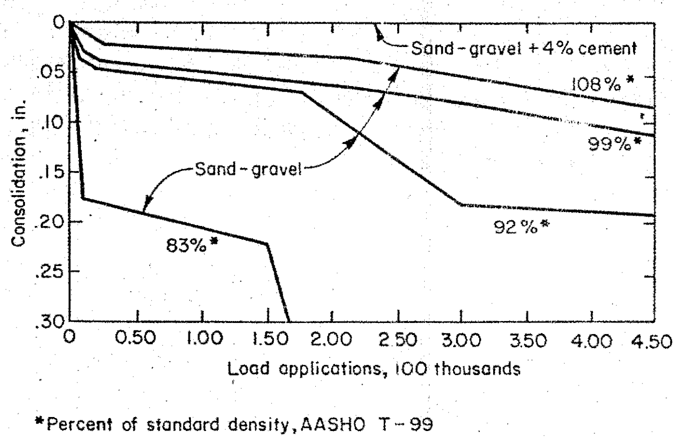


FIGURE NO. 3-2 - Base Consolidation Under Repetitive Load

3.2 GRADATION DISTRIBUTION

The density of a material is greater when it consists of particles of various sizes rather than particles of a more uniform size. This is known as a materials particle size distribution. As an example, Figure No. 3-3 shows a mass composed entirely of spheres of one size which in its most dense state is 74 percent solids and 26 percent voids. If smaller spheres are introduced into the mass as shown in Figure No. 3-4, the amount of solid material is increased but does not change the total volume.

The density is increased, the voids decreased and therefore stability increased. The greatest possible density will be obtained by continuing to put smaller spheres or particles in the remaining void spaces until particles of very small size have been included and all voids are filled. Such a material is said to be well or dense graded. It is for this reason specifications place limits on the various sieve sizes to try to attain as close to a uniform gradation as possible.

A coarse granular mixture which has little or no fines and is well graded, gains its stability from grain-to-grain, point-to-point contact of the coarse material as shown in Figure No. 3-5. While such a material may have better drainage properties, it will have low stability due to the lack of complete surface contact binding the coarser material together.

At the other extreme, a material may contain a large amount of binder and reduce a large amount of the coarse grained point-to-point contact, thus,greatly reducing stability as shown in Figure No. 3-6. What little stability the material has is a result of the surface binding of the fine particles only. In such a material, the coarse aggregates merely float in the fine material which serves as a lubricant. Such a material is low in permeability, easy to compact, easily affected by moisture changes, and generally has very low stability.

The base material for an individual project should have a reasonably constant gradation to allow compaction equipment to produce the uniform density and stability support essential for good pavement performance. Abrupt changes in base gradation can cause abrupt changes in support stability. AASHTO M155 establishes minimum material requirements to control pumping but does not provide acceptable gradation control. AASHTO M147 divides various base materials into separate gradations as a guide. Each SHA will specify its own gradations.

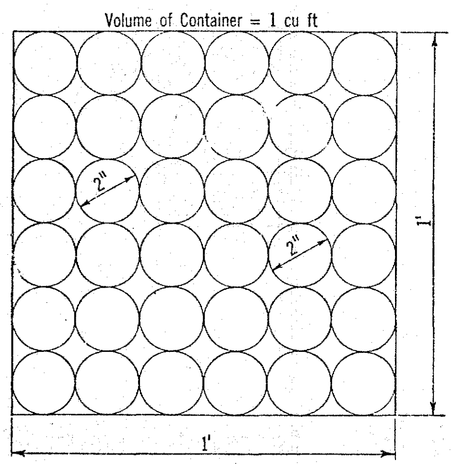


FIGURE NO. 3-3 - Single-Sized Particles

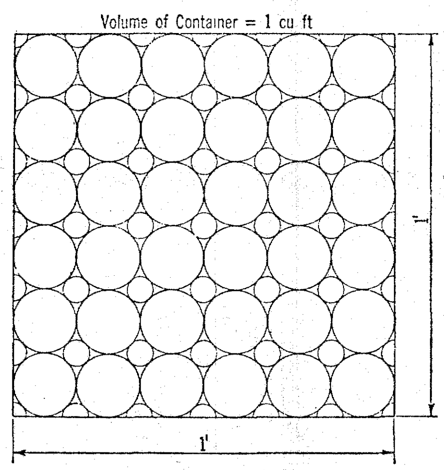


FIGURE NO. 3-4 - Variable-Sized Particles

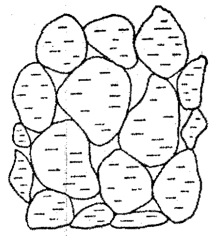


FIGURE NO. 3-5 - Coarse Aggregate With No Fines

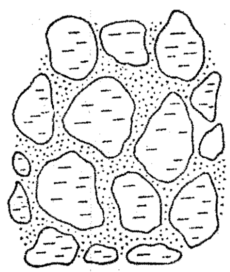


FIGURE NO. 3-6 - Coarse Aggregate With Excess Fines

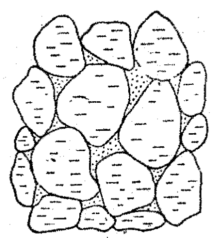


FIGURE NO. 3-7 - Coarse Aggregate With Sufficient Fines
For Maximum Density

3.3 PERCENT FINES

An aggregate soil mixture containing sufficient fines to just fill the voids between the courser material gains its stability from coarse aggregate point-to-point contact and also from the complete surface binding contact between all particles provided by the fines as shown in Figure No. 3-7. Thus, its density is high and its permeability is low. Such a material is moderately difficult to compact but is ideal from the standpoint of stability.

The question arises as to what a sufficient amount of fines is. As previously stated, when we refer to the term "fines," we are referring to the material passing the No. 200 sieve. For a given compaction effort on a well-graded granular material, there is an optimum percent of fines for maximum stability as shown by Figure No. 3-8. This slight difference is due to the non-linear relationship between density and stability. This also shows that it is preferable to be below the desirable percent fines for maximum stability. To control concrete pavement pumping AASHTO M155 recommends a maximum 15 percent passing the No. 200 sieve.

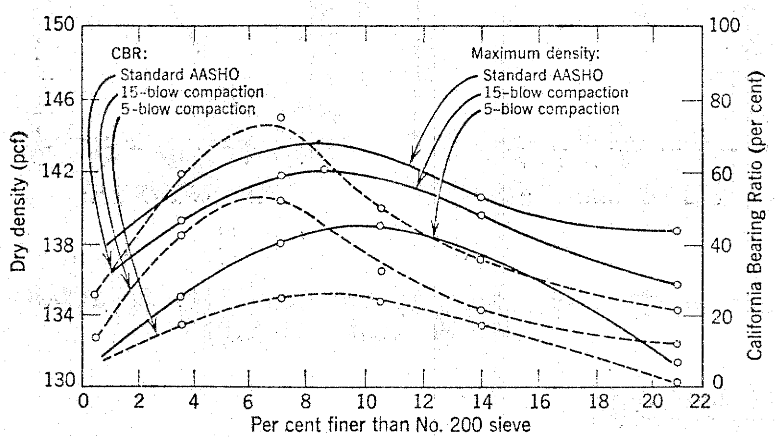


FIGURE NO. 3-8 - Density and Stability Relationship To Percent Fine Material

3.4 MOISTURE CONTENT

Moisture content has a pronounced effect on the density and, therefore, the stability of a material subject to a given compactive effort. As moisture content increases, density will increase for a given compactive effort to some maximum value corresponding to the optimum moisture content for the material. Thus, as depicted in Figure No. 3-9 for each base material there is a maximum density and stability which corresponds to optimum moisture. Below the optimum moisture, additional water tends to fill the voids and increase the lubricating effect of the coarser material, thereby increasing the density for a given compactive effort.

Above the optimum moisture, water added displaces solids rather than voids thereby decreasing the density and thus stability. The in-place effect of moisture changes is illustrated in Figure No. 3-2 which depicts in-place consolidation under load as moisture content changes. In figure No. 3-2 the first 150,000 load repetitions are made at optimum base moisture. Water was added for the next 150,000 load applications and for the next 150,000 load repetitions the base was saturated. The consolidation increased under loading as moisture content increased.

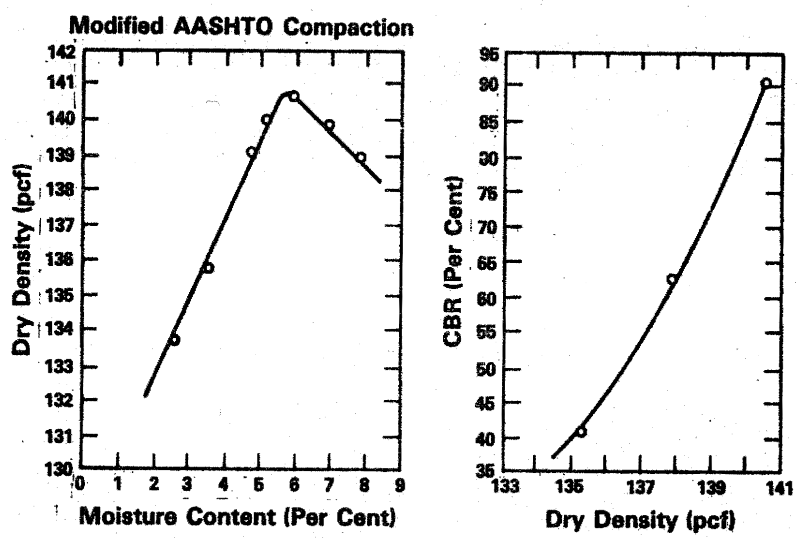


FIGURE NO. 3-9 - Density and Stability Relation
To Moisture Content

3.5 MAXIMUM SIZE AGGREGATE

In a well graded granular material, the dry density increases for a given compactive effort. However, the larger the maximum aggregate, the more difficult it is to work and compact a given material. Large aggregates will pull out of the surface during grading and compaction and compaction equipment tends to fracture or degrade large aggregate sizes.

Also, many believe that a more uniform distribution of pressure is obtained with the use of smaller maximum size aggregates. The larger the maximum aggregate size, the more susceptible the mix is to segregation. Thus, generally a granular base material uses a top size of 1-2 inches although larger aggregates can be used.

3.6 PLASTICITY INDEX

The physical properties of the binder material have a great effect on the stability especially when grain to grain contact is destroyed. The properties of the binder material, which is the material passing the No. 40

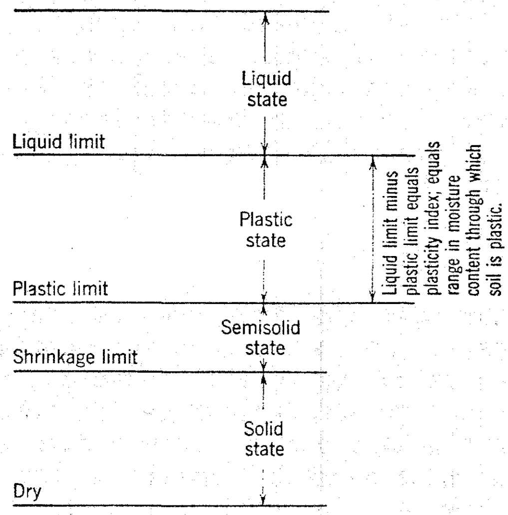


FIGURE NO. 3-10 - Binder Material Properties

sieve, are determined by tests for the percent moisture representing the liquid limit, and plastic limit, and the plasticity index which are shown in Figure No. 3-10. The plasticity index is the difference between the liquid and plastic limits (Atterberg limits) and represents the range of moisture content within which the binder material exhibits the properties of a plastic solid.

A plastic solid is one which can be kneaded without cracking. It also is a measure of the amount of clay material in the binder, the cohesive properties of the soil, and indicates the bonding properties of the fine clay fraction. The plasticity index is an indicator of the amount and suitability of the binder fraction's clay material.

A base course material with a high plasticity index will be very susceptible to moisture and tend to soften when wet. Excessive amounts of clay in a base course where moisture evaporation cannot take place will cause swelling and softening of the base course. Generally, material with a plasticity index less than 10 is considered nonexpansive.

Accordingly, limits are placed on the amount of binder material and the plasticity index of the binder material. Generally, maximum stability will occur when there is 14-15 percent binder material, the plasticity index is 6 percent, and when the liquid limit is not greater than 25 percent. AASHTO M155 recommends that if the percent passing the No. 200 sieve exceeds 15 percent, or the plasticity index exceeds 6 percent or the liquid limit exceeds 25 percent that stabilization be used to reduce concrete pavement pumping.

In order to ensure a uniform gradation of the binder fraction and to ensure that the binder fraction is not excessively fat with clays, a maximum limit is generally placed on the percent passing the No. 200 sieve in relation to that passing the No. 40 sieve. This ratio is generally 2/3 and is called the dust ratio.

3.7 AGGREGATE HARDNESS

Hardness of base course material is generally thought of as the ability of the aggregate to withstand degradation due to abrasion and/or crushing. This is important since degradation can increase the amount of fine material which can decrease density and stability if excess amounts are present. Such material degradation can occur during construction under the action of compaction equipment or after construction due to traffic loading. For this reason, aggregate abrasion loss should be limited to less than 40 percent under the Los Angeles abrasion test.

3.8 PERMEABILITY

Inadequate drainage of the pavement structure has been identified as a primary cause of pavement distress. The thinking on the permeability of base courses is changing. Rather than using impermeable dense-graded materials, several States are using open-graded or permeable bases to allow infiltrated moisture to rapidly drain through the base and out from underneath the pavement structure. The effect of more permeable material under flexible pavements is still being evaluated. However, for rigid pavements, it appears that the use of free draining base material is considered beneficial.

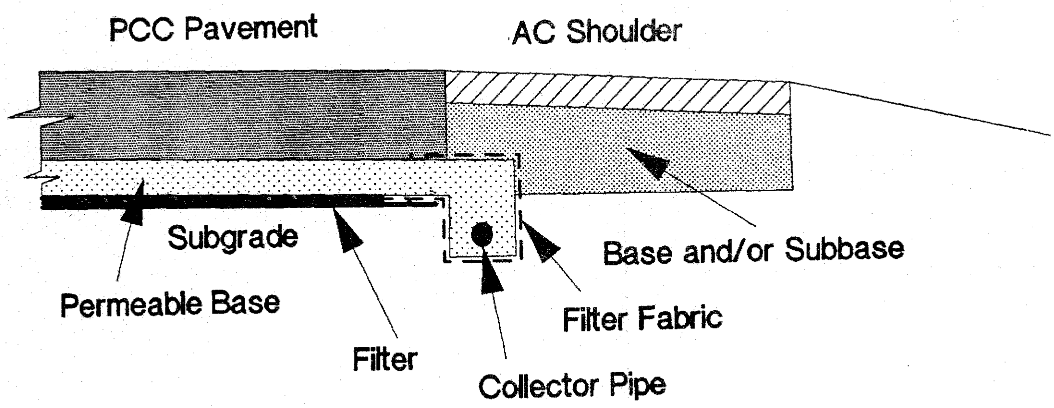
For permeable bases we are recommending a coefficient of permeability in excess of 1000 feet/day. This value of permeability is significantly greater than that of most pavement bases constructed in the U.S. which were normally less than 2 feet/day.

A permeable base is characterized by an open-graded crushed angular aggregate with essentially no fines. A longitudinal edgedrain system should be used to rapidly drain the moisture that collects in the permeable base. Daylighting of permeable base is not recommended because it is subject to clogging from vegetation and/or roadside slope debris. Typical permeable base pavement sections are shown in Figure No. 3-11.

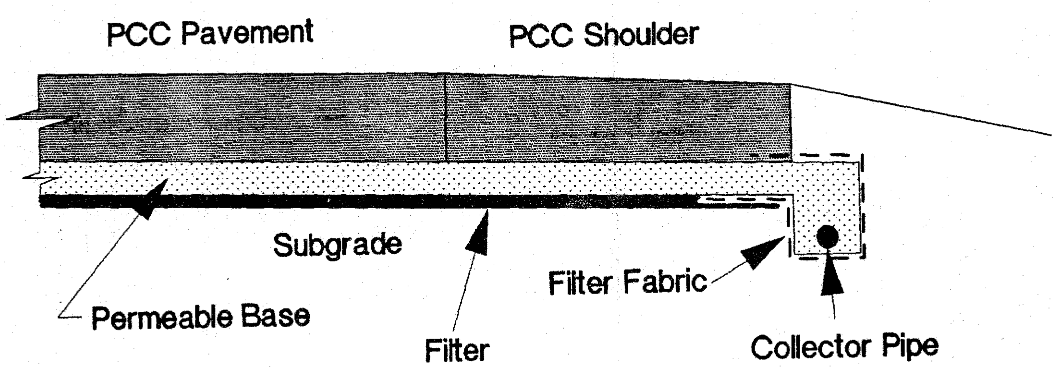
Permeable bases can be designed and constructed without significant changes to conventional practices. However, construction of permeable base pavements does require more care than dense-graded base pavements. This is because of the absence of fines and the corresponding reduced stability. To prevent the intrusion of fines into the permeable base and longitudinal edgedrain collector system, a filter layer meeting the filter criteria is used. To increase stability of highly permeable gradations, asphalt cement (at approximately 2 percent) or Portland cement (at 2-4 bags per cubic yard) have been used. The treated permeable bases have sufficient stability for construction traffic, however, extra care is needed to prevent contamination of the layer. Untreated permeable bases, although sufficiently stable to pave on, are more easily displaced than dense-graded base. Additional care is required by equipment operators and truck drivers when placing and finishing the pavement. Quite often, a roller is used to "dress up" untreated permeable material immediately in front of the paver.

Figure 3-11

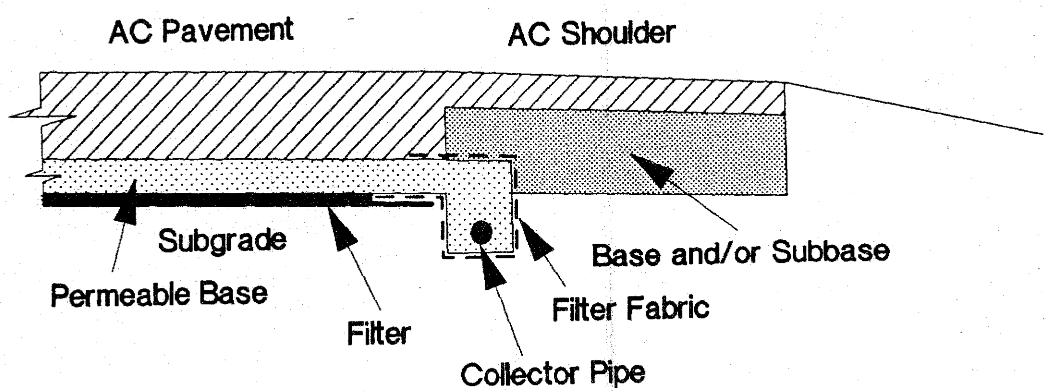
Typical Permeable Base Pavement Sections



PCC Pavement/AC Shoulder Section



PCC Pavement/PCC Shoulder Section



AC Pavement/AC Shoulder Section

Most States restrict construction equipment other than the paving and finishing equipment from traversing the permeable base. Also, most States now specify tracked pavers on untreated permeable bases which better distribute the load. Another concern with untreated permeable aggregate material is the possible segregation of the material during placement and degradation of the aggregate under construction traffic (if allowed). Several States specify that untreated permeable aggregate be placed at a certain percent moisture to reduce segregation.

The grade of treated permeable bases is more difficult to modify once it has been placed and compacted. Also, keeping the highly permeable base material clean and free from contamination is a concern.

Other modifications in the case of PCC pavement include the use of wider tires on the reinforcing mesh cart, and the use of longer pins to hold dowel baskets in place.

Additional guidance on the use of free draining bases is expected to be issued during 1989.

3.9 QUALITY FACTOR INSPECTION PRIORITY

In 1979, the FHWA Technical Advisory T 5080.1 indicated and grouped quality factors or tests in order of importance or significance. Given limited personnel resources and available time on a project, the chart gives an indication to an FHWA area engineer which quality factors should be reviewed and in what priority. Eliminating those quality factors related to stabilized or treated bases, the quality factor groupings for untreated/unstabilized bases reduces to those quality factors shown in Table No. 3-2.

TABLE NO. 3-2 - Base Course Key Quality Factors

Group I	Group II	Group III
Density (compaction)	Moisture Content	Surface Smoothness
Gradation	Uniformity of Materials	Placement Inspection
	Drainage	
	Thickness	
	Aggregate Quality	

Aggregate quality is largely pre-determined as a material source approval factor and not a day-to-day construction activity. Because density is related to moisture content we shall combine the two factors into one. Also uniformity of materials is in a sense related to gradation and placement inspection. Good control of gradation will help to ensure uniformity of materials and placement inspections also is intended to insure uniformity of materials. Taking these factors into consideration, the Table No. 3-2 quality factor can be simplified as shown in Table No. 3-3.

TABLE NO. 3-3 - Base Course Key Quality Factors (Simplified)

Group I	Group II	Group III
Density (compaction, Moisture content)	Thickness	Surface Smoothness
Gradation (uniformity of materials)	Construction Drainage	Placement Inspection (uniformity of materials - stockpiling)

Thus, the number of factors and groupings became much simplified. In actual point of fact, all six factors are important and, if possible checks on all should be made by an FHWA area engineer.

4.0 GRANULAR BASE COURSE CONSTRUCTION

4.1 MATERIAL MIXTURE TYPES

There are a large variety of "granular materials" used in pavement bases throughout the country. The actual use and selection of the materials and mixtures to be used varies due to available materials, cost, roadway type and traffic. There are generally three materials combinations as follows:

- soil aggregate
- graded aggregate
- graded crushed stone aggregate

Soil aggregate is a combination of granular materials and soil particles which occur together naturally and which may be used with little or no processing other than crushing. Generally, such material occurs in natural deposits or pits and are poorly or variable graded. For this reason they are not very desirable as base materials on high type roadways. However, due to their local availability and low cost they are at times used on low volume roads not subject to high stresses.

The actual gradation obtained is a result of the inherent production of the natural pit material and the crushing operation, and results in reduced gradation control. The material is sometimes referred to as pit run or crusher run material. It generally has lower stability, due to its inherent less than optimum gradation and poorer material properties as compared to processed and combined materials.

4.2 SUBGRADE PREPARATION

The subgrade soil is the surface on which the base and other layers will rest. Subgrade analysis and preparation are vital to obtain good performance during the life of the road. It must be firm, as uniform in character as possible, and shaped to assure proper drainage.

Often, roadway construction is done in stages resulting in the subgrade being subjected to weather condition, and construction equipment traffic for extended periods prior to actual base course construction. The SHA inspector should be observant of heavy construction equipment on the subgrade even in tested areas considered good to detect weak areas.

Graded aggregate is crushed or uncrushed coarse aggregate from one source combined with an optimum amount of fine mineral soil from other sources. Such material from different locations is processed and controlled by careful mixing to obtain the desired gradation and to ensure it is free from organic material and low in silt and clay.

Because of the mineral soil consistency, graded aggregate may be a plastic material and subject to control of the plasticity index. They are more costly than soil aggregate mixtures but generally possess higher strength. Thus, they are widely used throughout the country subject to design and field control.

Graded crushed stone aggregate is a processed and graded material which generally is composed entirely of the products of one or more stone crushers. Because of the material's angularity and because the fine material is stone, it possesses little adhesive properties and is friable in dry weather and can abrade under traffic. For this reason, graded crushed stone should not be left exposed for long periods as weather and construction traffic may abrade it. It is, however, a highly dense granular material which gives the highest degree of stability under load and is nonplastic. Because of this, it is a highly desirable material for high type roadways subject to availability and cost consideration.

The pavement design analysis, cost analysis, and materials availability in the base material selection process will determine the most appropriate material. However, regardless of whether a graded aggregate or graded crushed stone base is utilized the field construction process, inspection controls and testing are similar as will be covered in this section.

An adequately compacted, firm, and unyielding, dry subgrade is important in order to secure proper compaction of the base material. A properly prepared and compacted subgrade may develop localized soft, yielding areas caused by such things as moisture, raveling, and rutting brought about by environmental conditions and construction traffic. Such conditions must be repaired prior to base construction. It is, therefore, important that the subgrade be inspected for such conditions, reprocessed, compacted and bought to proper grade.

Such local deficiencies should be corrected either by removal and replacement of the deficient material or reworking and regrading it to produce the desired strength. While in some cases removed material may be replaced with base material, caution should be taken to ensure material is of similar drainage properties. This will prevent depressions in the impervious subgrade from trapping water when the base is a pervious material.

Prior to base course construction, the project engineer should make a careful, visual inspection to detect problem areas especially if the subgrade has been left exposed and under traffic. As area engineer, you should also be alert for such conditions and make sure the project engineer has conducted a sufficient review and taken appropriate corrective action.

4.3 QUARRY/CRUSHING OPERATIONS

The construction methods and procedures related to granular base course material begins at the quarry or pit where rock material is obtained, crushed, screened and sized so that upon processing and/or mixing it will eventually meet specifications.

Crushing operations may be permanent large type plants working out of large established quarries supplying a variety of industrial needs. They may also be very small, temporary, crushing operations brought in to crush material for isolated projects in remote areas.

The types of materials used for base courses such as limestone, granite, natural pit run, etc., will produce different gradations of aggregates after processing by crushing. A single statewide specification or base course gradation will require different crushing operations and extra screens depending on such factors as the material hardness and production of fines during crushing. Therefore, the actual operations are beyond the scope of this course as gradation will vary with type of equipment, equipment settings, type of material, rate of feed, and material moisture. In addition, while it may be desirable, it is not really essential that an FHWA area engineer have detailed knowledge of the operations and variables involved in the proper operation of rock crushing plants.

When a quarry or pit is first opened up, whether on a permanent continuing basis, or only for an individual project, the SHA should conduct qualification tests on the stone to ensure it meets quality specifications. Thereafter, tests are run periodically according to a prescribed schedule to assure continued material acceptability. Some tests normally run are:

- Los Angeles abrasion test
- soundness tests
- production of plastic fines in aggregates
- specific gravity
- absorption

Should the area of the quarry/pit being worked change, additional tests should be conducted. Although not an essential day to day check, an area engineer should make a point to check that the proper tests have been run recently with acceptable results at the initial start up of operations.

In the production of graded aggregate, or graded crushed stone aggregate, materials are normally stockpiled in various sized fractions in advance of project use or demand. This helps to ensure proper gradation control and formulation later for various gradations which the plant may be asked to produce.

In addition, it helps to prevent segregation since the size fractions are of more uniform size which helps prevent segregation due to handling, equipment, and weathering. Best gradation control can be maintained when stockpiled in at least two sizes and desirably three. Normally, the three sizes used are as follows with the majority of the material falling within the designated sieve range;

STOCKPILE PILE	SIEVE RANGE
Coarse aggregate	> No. 4
Fine aggregate	< No. 4 - No. 200
Fine material (binder)	< No. 40

Should the SHA specify an open graded granular material, the amount of fine material may be restricted to provide for high permeability.

Generally, if the SHA specifies a graded aggregate, then the coarse material will be crushed stone, the fine aggregate will be sand, and the binder material will be soil. For a graded crushed stone aggregate, all the three size fractions are made of crushed stone material with addition of minor amounts of silt and clay for cohesion.

4.4 STOCKPILING MATERIALS

Consistent gradation is an important factor in ensuring uniform stability to avoid differential settlement under future traffic loadings. When a falling stream, or small mass of granular material, is brought to rest, mutual collisions between small and large particles exert forces which on average, tend to distribute the large particles around the lower, outer portion of the resulting cone. This is more commonly known as segregation which contributes to nonuniformity of gradation.

The larger the difference between the material's large particles and fine particles the more pronounced the segregation can be. Such segregation may occur as a result of material handling at various points in the construction process; one of which is material stockpiling.

Procedures for handling and stockpiling base course material will vary from job to job since all plant and contractor operations vary and since specifications may not specify handling and stockpiling procedures. An agency usually requires a contractor to meet aggregate gradation specifications. It is important for an area engineer to be knowledgeable of good and bad stockpiling practices which affect aggregation, gradation and base stability. Figure No. 4-1 graphically presents good and bad stockpile practices.

Important stockpile and handling considerations are:

- stockpile in sized fractions
- preparation of stockpile area
- separation of different stockpiles
- layer stockpiles (not conical)
- reduce free fall distance
- proper material removal procedures
- tracked vehicles
- excess handling
- stockpile age

Although stockpiling in various size fractions helps prevent segregation by reducing differential particle size, good material handling practices should apply whether stockpiled in various fractions prior to blending, or in the final gradation blend of the material to meet specifications.

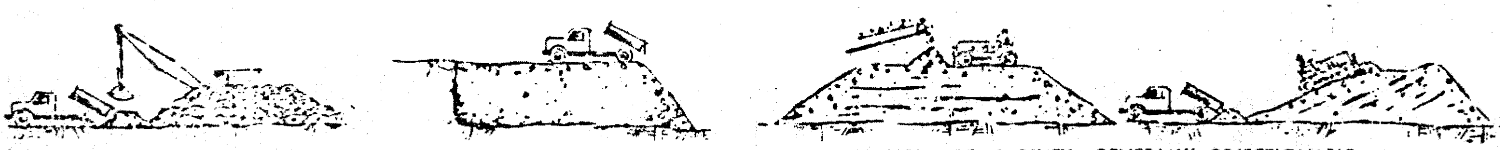
Before stockpiling granular base course material begins, the area should be suitably prepared. Generally, this consists of preparing a firm clean area with good drainage and of adequate size. In addition, adequate provisions should be made to keep stockpiles separate by widely spacing stockpiles or using bulkheads between stockpiles if necessary.

The shape of the prepared stockpile should always be controlled in a proper manner. When material containing both coarse and fine is heaped into a stockpile with sloping sides, the coarser particles tend to roll down the slope and accumulate, segregating the material. Therefore, material should be stockpiled in layers and not in tall conical piles.

Truck delivered material should be placed close together over the surface of the stockpile. If a crane bucket is used the material should be placed, not cast, in adjacent piles. The thickness of each layer will be dictated by truck size but generally should be 5-6 feet. It should be emphasized that stockpiled material should not be allowed to slide or roll to its ultimate location. Thus, material should not be dumped over the stockpile edge, cast, or pushed downhill.

Material with fines should not be allowed to free fall excess distances to avoid wind blowing and separating the very fine material from the course material. To prevent this, a chimney or enclosed shoot is desirable to prevent exposure to wind.

INCORRECT METHODS OF STOCKPILING AGGREGATES CAUSE SEGREGATION AND BREAKAGE



PREFERABLE

A crane or other equipment should stockpile material in separate batches, each no larger than a truckload, so that it remains where placed and does not run down slopes.

OBJECTIONABLE

Do not use methods that permit the aggregate to roll down the slope as it is added to the pile, or permit hauling equipment to operate over the same level repeatedly.

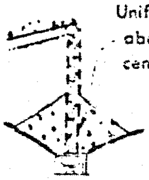
LIMITED ACCEPTABILITY—GENERALLY OBJECTIONABLE

Generally, a pile should not be built radially in horizontal layers by a bulldozer working with materials as dropped from a conveyor belt. A rock ladder may be needed in this setup.

A bulldozer stocking progressive layers on slope not flatter than 3:1 is also objectionable, unless materials strongly resist breakage.

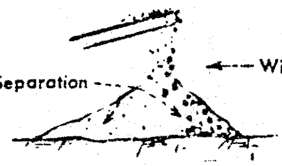
STOCKPILING OF COARSE AGGREGATE WHEN PERMITTED

(Stockpiled aggregate should be finish-screened at batch plant; when this is done, no restrictions on stockpiling are required.)



CORRECT

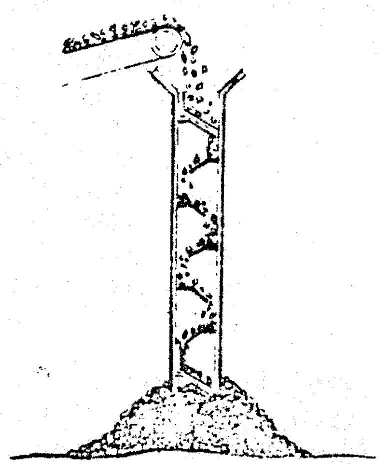
Chimney should surround material falling from end of conveyor, to prevent wind from separating fine and coarse materials. Openings should be provided as required to discharge materials at various elevations on the pile.



INCORRECT

Do not allow free fall of material from high end of conveyor, which would permit wind to separate fine from coarse material.

UNFINISHED OR FINE AGGREGATE STORAGE (DRY MATERIALS)



When stockpiling large-sized aggregates from elevated conveyors, minimize breakage by use of a rock ladder.

FINISHED AGGREGATE STORAGE

FIGURE NO. 4-1 - Correct and Incorrect Handling of Aggregates

At large plants, one often sees belt-fed and dropped material placed in large, very high conical stockpiles. These would, on the surface, seem contrary to good stockpile procedures. However, such stockpiles are made of uniform-sized material to be blended later and thus are much less susceptible to segregation.

There are no set rules for removing materials from a stockpile but some guidelines are generally applicable. It is preferred to remove the material with a front end loader from the near vertical face of the stockpile. In doing so, the bucket should be kept 6-12 inches above the ground to avoid picking up foreign material. Having a bulldozer or other tracked vehicle working on top of the stockpile is also objectionable since it increases the possibility of crushing and degradation of the material.

The extent of handling and moving of the material will also reflect segregation. The material should not, therefore, receive excess handlings; i.e., loaded and unloaded or moved from point to point. Moved material should be checked for segregation by running gradation tests prior to incorporation in the work.

The age of stockpile has to also be considered since factors such as freezing and thawing, dust collection, etc., could have an effect on gradation. For this reason, a stockpile left standing for long periods should receive gradation tests prior to starting work.

4.5 STOCKPILE BLENDING CALCULATIONS

As was previously indicated, graded aggregate fractions are normally stockpiled and must be recombined to meet the desired specification range. This combining process takes place in a pugmill which is a piece of equipment into which the fractions are belt fed, adjusted for moisture, and mixed.

All pugmill operations, large or small, must initially be calibrated to determine the proper settings which will produce the desired specification gradation. In addition, during construction, material changes or problems may be encountered necessitating a redetermination of the stockpile mixing percentage proportions.

While proportion determinations and plant calibration is primarily the contractor/plant responsibility, most SHA's require inspectors to check the calibration. An area engineer should be aware of this and check that proper calibration has taken place. It is, therefore, important that he understand mathematical methods of determining the initial and needed changes in proportions.

A widely used method is to combine the different material fractions by weight (gradation) so that the resultant grading is within the specification range and then test a sample of the mixture to see if the L.L. and P.I. are satisfactory. With the proportions of each fraction as a starting point, the pug mill belt settings are made. Then on a trial and error basis fine adjustments are made to produce the most desirable gradation.

Determining the proportions of two or more aggregates to blend for a gradation within specification can be done mathematically, graphically, or by trial and error. Sophisticated mathematic procedures to determine combination proportions are involved and time consuming even with a calculator.

Graphical methods have been devised to determine blend proportions. As the number of aggregates to be combined is increased, graphical methods get very complicated. However, for two and sometimes three aggregate materials, graphical solutions are fairly easy to use, particularly in the field. Sometimes, plant personnel may be able to arrive at a satisfactory mix without any preliminary calculations. However, it is good to conduct both steps to avoid unnecessary delays and production of nonspecification material initially. Two illustrative graphic examples will be shown demonstrating blending percentage determinations from two and three stockpiles.

EXAMPLE - TWO STOCKPILE BLENDING CALCULATIONS

Suppose we have two stockpiles with aggregate gradation A and B as shown in Table No. 4-1.

TABLE NO. 4-1 - Gradations for Example Blending Two Stock Piles

	PERCENT PASSING				
SIEVE	A	B	C	76/24% BLEND	73/27% BLEND
3/4	98	100	98-100	98.5	98.5
3/8	65	98	65-85	72.9	73.9
No. 10	35	79	40-50	45.6	46.9
No. 40	30	15	18-28	26.4	25.9
No. 200	10	1	6-10	7.8	7.6

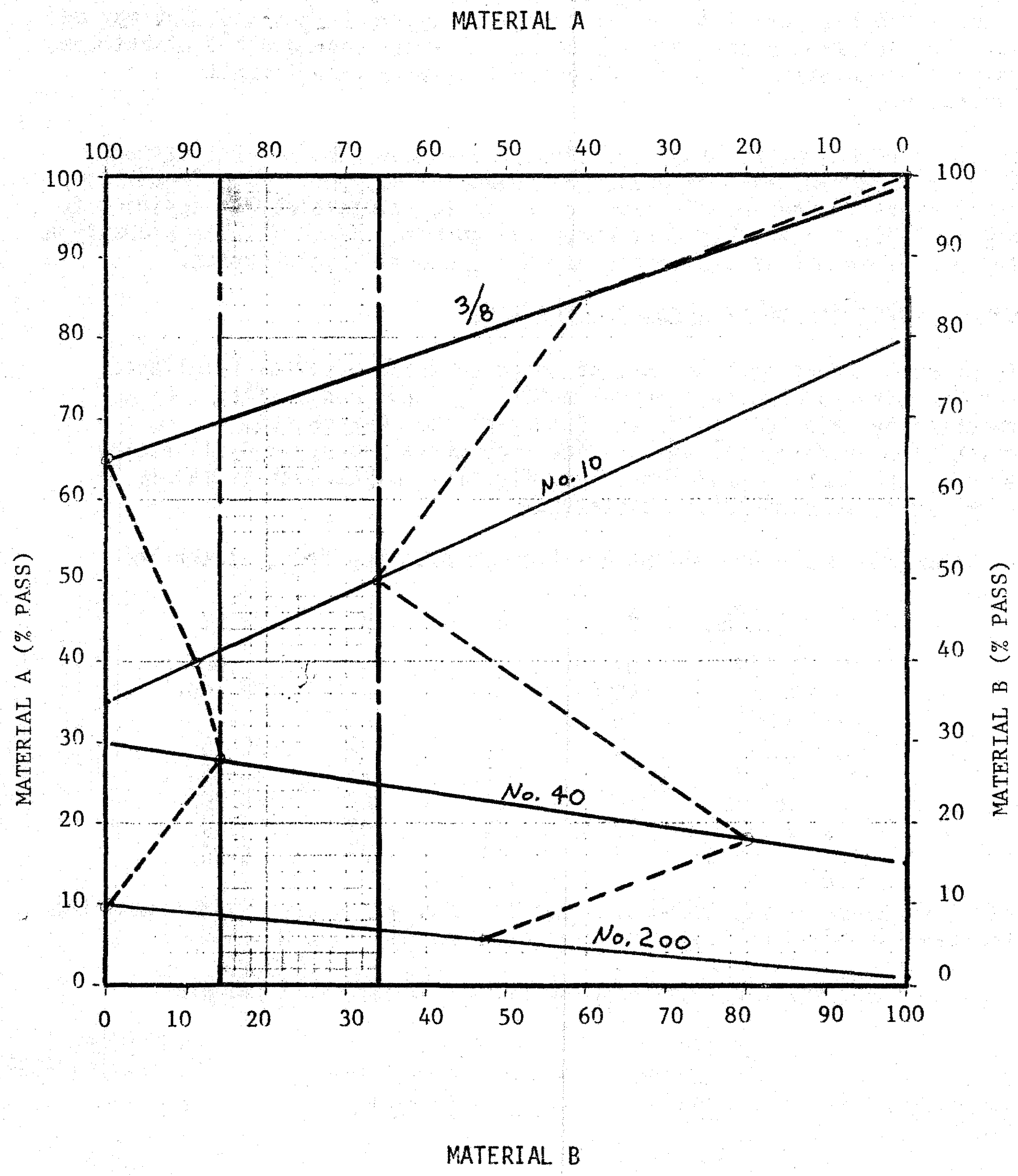


FIGURE NO. 4-2 - Nomegraph For Combining Two Materials

Neither material A nor B will meet the required material C specification range by itself. The Figure No. 4-2 nomograph illustrates how to determine the two materials proportions so that their combination would meet the specification shown.

The percent passing each sieve for both aggregate stockpiles A and B are plotted on the left-hand and right-hand ordinate, respectively. These points are then connected by straight lines which are labeled for the sieve sizes the lines represent. The specification range percent passing is then plotted on each sieve line and connected by the dashed lines. This defines for each sieve size the limited percentage of aggregate A and B that may be blended to meet specification gradation C. In this case a blend of between 14-34 percent material B, and 66-86 percent material A will meet specification.

Selecting a blend in the middle of these ranges would be a 76/24 percent mixture which would give the gradation shown in Table No. 4-1. Since this blend is close to the No. 40 sieve upper limit, it would appear desirous to change to 73/27 percent blend as shown in Table No. 4-1. This is probably a better blend percent as a starting point for pug mill calibration.

EXAMPLE - THREE STOCKPILE BLEND CALCULATIONS

It is sometimes necessary to combine three separate material fractions. Suppose we wish to combine a gravel material, sand, and a silty clay as represented by materials A, B, and C in Table No. 4-2 to meet a specification gradation as shown. The term gravel, sand, and silty clay refer only to size of particles. Actually, they might just as easily be crushed rocks, screenings and stone dust.

TABLE NO. 4-2 - Gradations For Example Blending Three Stockpiles

SIEVE	A	B	C	SPEC.
1 1/2	100	-	-	100
1	98	-	-	75-100
3/4	76	-	-	-
1/2	48	-	-	40-75
No. 4	27	100	-	30-60
No. 10	18	64	100	20-45
No. 40	10	28	97	15-30
No. 200	4	11	85	5-20

First, the material gradations and specification range are divided into the percentages of their constituent parts as shown in Table No. 4-3.

TABLE NO. 4-3 - Constituent Parts - Three Blend Example

Material	COURSE (% Retained on #4)	FINE (% Passing #4, less % Passing #200)	SILT/CLAY (% Passing #200)
A	73	23	4
B	0	89	11
C	0	15	85
SPEC	70-40	25-40	5-20

The next step is to plot Table No. 4-3 material A, B, C, and specification range points on the Figure No. 4-3. These are shown on triangular nomograph represented by points A, B, C and the parallelogram. Draw lines connecting points A, B and C. Draw a line from A through the parallelogram Center D intersecting line BC at point E. Draw a line from B through the parallelogram center D intersecting line AC at point F. Draw a line from C through the parallelogram center D intersecting line AB at point G.

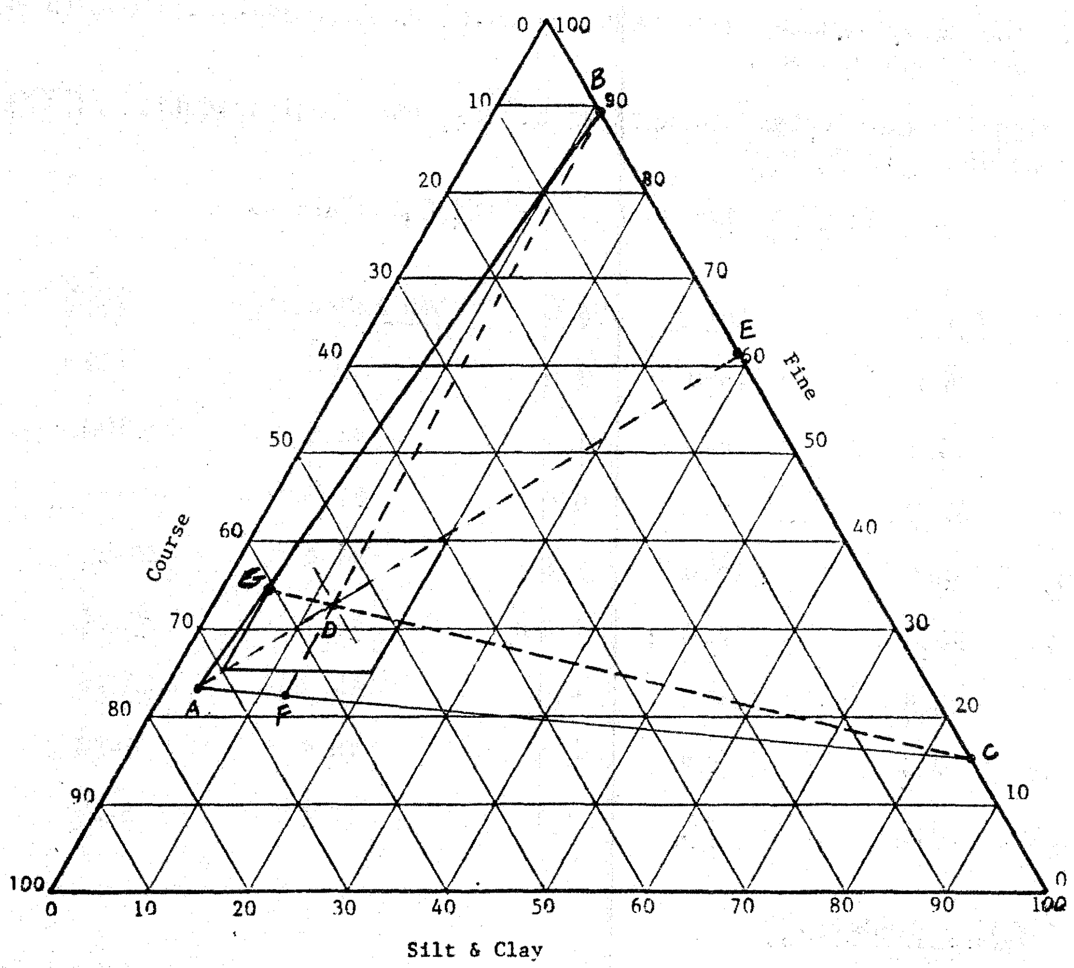


FIGURE NO. 4-3 - Nomegraph For Combining Three Materials

The percentage of material from each stockpile needed to meet the desired specification can then be calculated by measuring the length of the Figure No. 4-3 nomograph lines using any convenient scale and using the following formulas:

$$\% \text{ Material A} = \frac{DE}{AE} \times 100 = \frac{5.0}{6.7} \times 100 = 75\%$$

$$\% \text{ Material B} = \frac{DF}{BF} \times 100 = \frac{1.1}{6.9} \times 100 = 16\%$$

$$\% \text{ Material C} = \frac{DG}{CG} \times 100 = \frac{0.7}{7.6} \times 100 = \underline{9\%}$$

$$100\%$$

NOTE: The above lengths were measured using an Engineer's scale with 30 divisions per inch.

Utilizing the calculated percentages blends, the final gradation will be as shown in Table No. 4-4.

TABLE NO. 4-4 - Final Blend Calculations

SIEVE	75% A	16% B	9% C	COMBINED BLEND	SPEC
1 1/2	75.0	16.0	9.0	100	100
1	73.5	16.0	9.0	98.5	75-100
3/4	57.0	16.0	9.0	82.0	------
1/2	36.0	16.0	9.0	61.0	40-75
No. 4	20.3	16.0	9.0	45.3	30-60
No. 10	13.5	10.2	9.0	32.7	20-45
No. 40	7.5	4.4	8.7	20.6	15-30
No. 200	3.0	1.8	7.7	12.5	5-20

4.6 PUGMILL OPERATIONS

The secret to the production of an acceptable continuously uniform material is to require that the different component material fractions be mixed in a mechanical mixer or pugmill.

4.6.1 Pugmill Feed Control

Pugmill operations are almost always continuous v.s. the batch type and thus must be supplied material by a continuous feed operation depositing the different material fractions into the pugmill in the correct proportions. This is done by a multiple bin feed system. Each individual bin's feed consists of a gate, and precision belt feeder and or electric vibratory feeder control. Figure No. 4-4 depicts a schematic of the possible feed operation and control.

(a) Precision Belt Feeder Control (b) Electric Vibrator Feeder Control

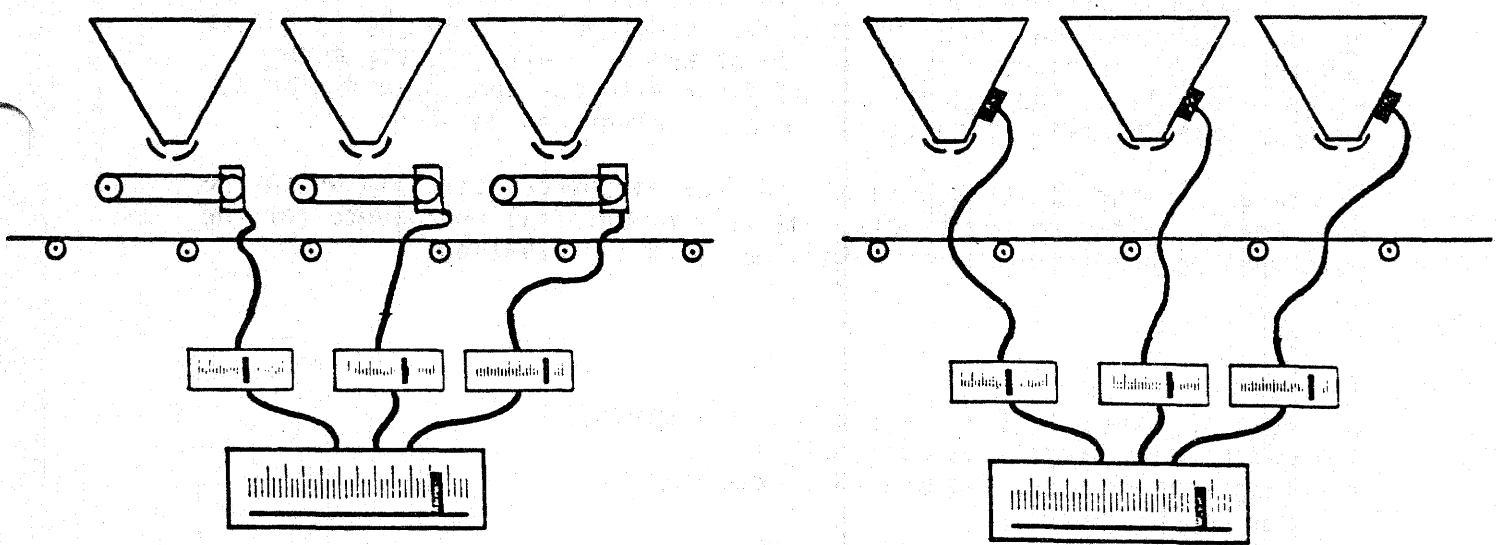


FIGURE NO. 4-4 - Material Bin Feed Control

The exact amount of each bin's material going into the mixture is controlled by setting the gate opening, varying the precision belt speed, and/or varying the frequency of the bin vibrator. The aggregate feeder gate system must be calibrated, set and secured to ensure uniform flow of aggregate. While this calibration is the responsibility of the contractor or plant operator, it is often checked by SHA inspectors. It is, therefore, important for the area engineer to be aware of what is involved in this process.

The gates should be calibrated for each type and size of aggregate used. Manufactures often furnish approximate calibrations for the gate feed systems with the equipment. However, the only accurate way to set the gates is to prepare unique calibration charts based on the local aggregates to be used. Permanent large type plants with pugmill operations generally use only certain size material in a bin. Consequently, they have a great deal of experience and know very closely what the settings should be.

There are two methods for calibrating the aggregate feeders: (1) adjustable gate openings with fixed speed belt feeders, and (2) semi-fixed gate openings with variable speed belt feeders.

Adjustable Gate Openings with Fixed Speed Belt Feeders

In this method, calibration is done by first opening one gate to 25 percent or less of maximum and then starting the feeder belt. When the feeder is running at approximately the same rate at which it will operate during actual production, the aggregate that flows from the open gate during a certain time interval is collected in a container and weighed.

If the gate being calibrated is a type that discharges directly on to the main feeder system conveyor belt, the flow of material per minute for the gate opening being checked is determined from the equation:

$$q = \frac{WR}{r}(1-m)$$

q = rate of flow of dry aggregate (lb. per minute)
W = weight of aggregate measured (lb.)
r = length of belt section where material was removed (in feet)
R = belt speed in feet per minute: and
m = moisture content of aggregate

On continuous belt feeder systems, where the gates discharge on to a small belt instead of a large, main belt, the flow of material can be calculated using the number of revolutions of the small belt. In doing so, the equation above is used, however,

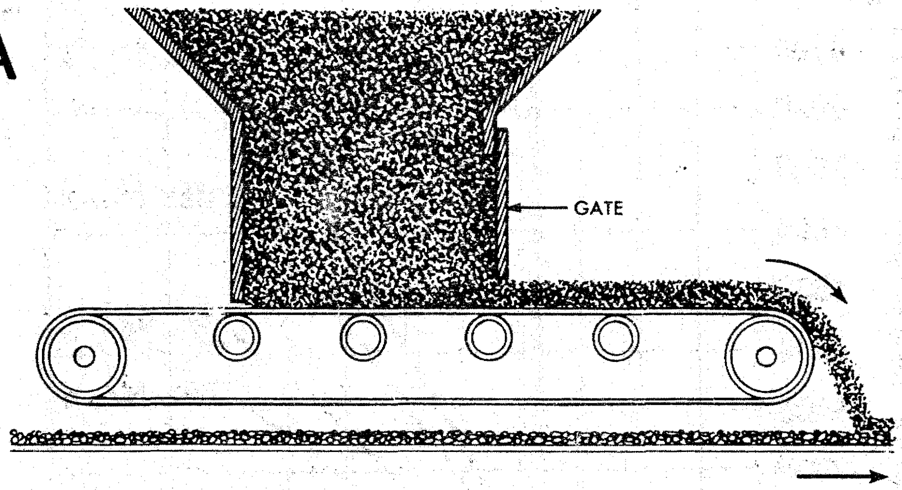
r = the number of revolutions of the small belt during aggregate collection, and
R = belt revolutions per minute (rate of revolution)

Gates located at the bottom of the bins feed controlled amounts of the different sized aggregates onto a conveyer belt which carries the aggregates to the pugmill. There are several different types of gate feeder systems. Among the two most common are the continuous belt type and the vibratory type as shown in Figure No. 4-5.

(a) Continuous Belt Type

(b) Vibratory Type

A



B

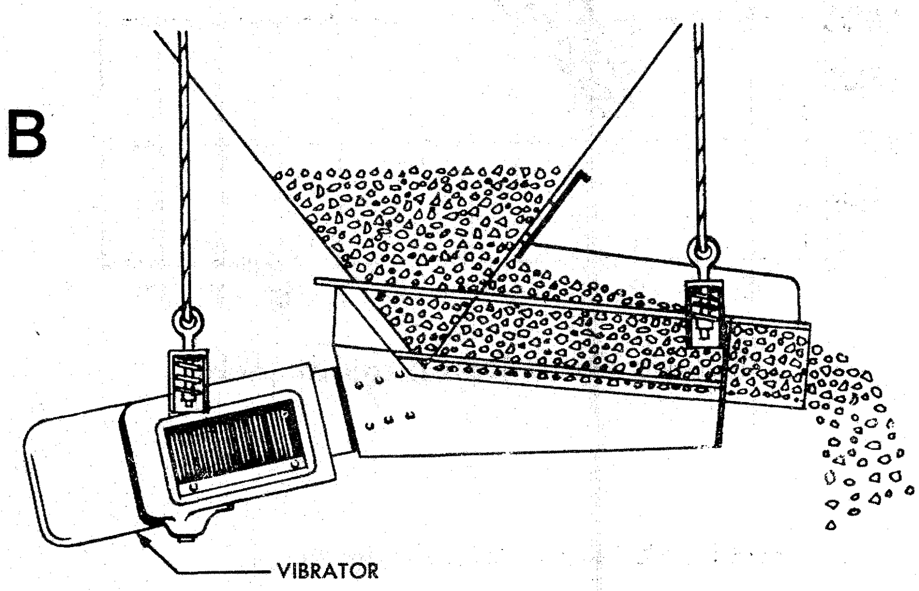


FIGURE NO. 4-5 - Gate Feeder Systems

The operation is repeated for three or more different openings of each gate. When multiple calculations have been done for each gate that will be used during production, a calibration chart is prepared. On the chart, gate openings in inches are plotted on the horizontal scale, and the dry weight of aggregate in pounds per-minute is plotted on the vertical scale. A typical calibration chart is shown in Figure No. 4-6.

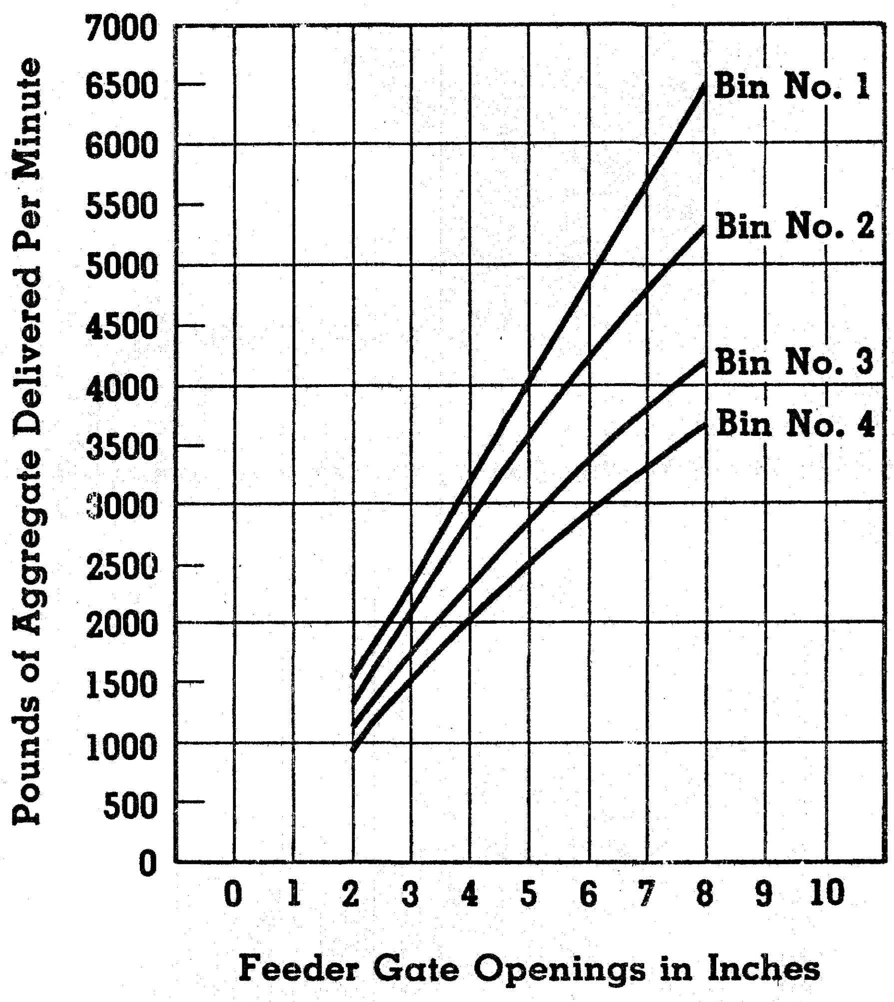


FIGURE NO. 4-6 - Calibration Chart

From the calibration chart the proper gate openings for each cold bin can be determined. In making this determination, the discharge rate of each gate must be balanced with the discharge rates of the other gates to ensure proper gradation of the combined mix.

Gate openings are dependent upon the project plant production in tons per hour. Gate openings that will produce that rate are calculated by using the equation:

$$Q = \frac{TP}{3}$$

where

Q = required rate of flow (lb. per minute)
T = plant production (tons per hour)
P = percent by weight of total mix.

Semi-Fixed Gate Openings with Variable Speed Belt Feeders

In many modern plants the feed gates are not adjusted for every range, but are controlled by variable speed belt feeders and vibrating feeders (measured in revolutions per minute, (RPM)).

To increase or decrease the amount of feed from a given bin, the RPM of the belts is increased or decreased according to the desired production rate.

To bring about this calibration, all cold feeds are filled with their respective sizes of aggregates. The plant is then started and the first feeder is set to run a given RPM. Once the plant is running uniformly, the amount of material discharged during a particular time period, say 30 minutes, is collected and weighed.

This procedure for the same bin or feeder is repeated for at least three calibrations (20, 50, and 70 RPM, for example). The production rate for the first feeder at each of these settings is calculated and plotted on a chart similar to the one shown in Figure No. 4-7. The procedure is repeated for each of the remaining feeders.

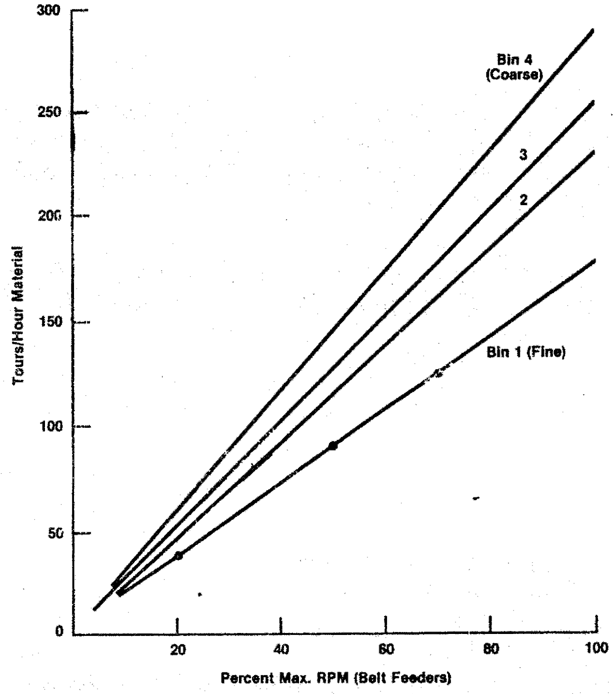


FIGURE NO. 4-7 - Calibration Feed Chart

If the gate feed is controlled by vibrators then the charts are developed using the vibrator motor amperage.

Bins may be fed by surge piles over an under ground tunnel containing the bin and feed system. They may also be fed by a front end loader removing material from stockpiles and filling the bins in a continuous operation. Loader operators should not remove stockpiled material at ground level to avoid contaminated or segregated material. Either method is acceptable. However, surge pile feed systems reduce handling and the opportunity for contamination and segregation. Surge pile systems are, however, associated with permanent type plants.

4.6.2 Pugmill Mixing

The predetermined quantities of material fractions are fed continuously from the main feed belt into the pugmill where they are blended by screw type paddle shaft blenders (twin shaft pugmill) which mix and move the blended material out the end of the pugmill box where it is deposited in a surge silo for transfer to haul trucks. The pugmill paddle tips are adjustable and easily replaced. Paddle should be checked periodically to ensure they are not excessively worn, damaged or missing which can cause poor mixing of the material.

To ensure proper mixing, clearance between the paddle tips and the linear should be less than one-half the maximum aggregate size. Also, poor mixing can occur if the chamber is being overfilled or underfilled. If overfilled, the uppermost material tends to float above the paddles and may not be completely mixed. If underfilled, the paddles tend to rake through the material without complete mixing. For best mixing the paddles should be barely visible above the mixing material in the pugmill chamber.

4.6.3 Moisture Content Control

It is essential that the approximate optimum moisture content be uniformly incorporated in the pugmill blending operation. This is essential both from the standpoint of proper compaction and minimization of segregation. Most specifications require that the moisture content be within perhaps + 25 percent of optimum moisture. Thus, it is essential that periodic checks of stockpile moisture be conducted particularly after rain or if wet areas of the stockpile are encountered. However, since material fraction stockpiles are generally free draining, moisture content should stay fairly constant resulting in a blended mix below optimum. This generally necessitates that water be added at the pugmill.

In very hot, windy weather and with long haul distance, evaporation can be anticipated to take place during hauling and placing making it preferable to be slightly above optimum.

Water is added continuously in the pugmill through a spray bar at a certain rate so that it is blended during the mixing process. The rate of water feed is dependent on the rate of material feed and moisture content of the material introduced into the pugmill and is controlled by use of water meter or other device. Moisture content should meet specifications and remain uniform.

As long as this is the case, we need not be overly concerned how the water is introduced. However, if problems occur, the accuracy of the water meter or other control device should be checked. Also, all spray bar nozzles should be operational to ensure water is introduced uniformly in the bin and not all at one point.

Mixed material may be stockpiled for future use. However, blended stockpiled material increases handling causing segregation and degradation, will likely require moisture modification at time of use, and will require additional testing. It is, therefore, not a desirable situation. It is much preferred to mix the material at the proper moisture and haul it directly to the job site for placement.

4.7 TRANSPORTING AND SPREADING OPERATIONS

There are good and bad practices and items to be aware of and checked which may influence the quality of the final base course of which an Area Engineer should be aware. These areas include:

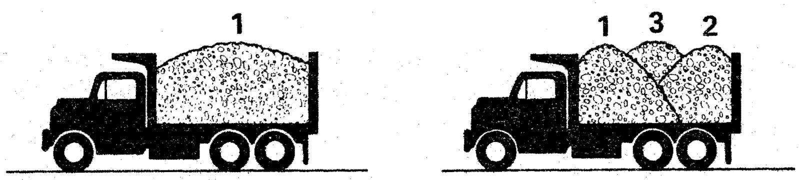
- o lift thickness
- o operations conducive to segregation

4.7.1 Loading/Hauling Materials

Depending upon the placing operation, an end dump truck or bottom dump may be used. In the case of stockpiled material, trucks may be loaded using a front-end loader. Again, from the standpoint of excess handling and segregation this is not a desirable loading method. Due to rapid loading of trucks, drivers tend to pull under the storage silo without moving the truck in the proper loading method.

A truck loaded without moving is shown in Figure No. 4-8(a). When loaded in this manner the truck bed fills in a single cone and spreads out filling the bed. The course material segregates around the perimeter of the cone and is deposited around the trucks bed walls. As the truck is dumped, the segregated material will remain segregated. Desirably, a truck loading procedure shown in Figure No. 4-8(b) should be used. Such loading causes less segregation and has a tendency to remix minor segregation upon discharge.

(a) (b)



Poor Desirable

FIGURE NO. 4-8 - Truck Bed Filling Procedures

Wind passing over the filled truck bed during transportation causes moisture evaporation especially in very hot, dry weather. It is, therefore, important that trucks be equipped with covers and used if haul distances are excessive to maintain proper moisture content.

In some cases material is dumped directly on the subgrade and then spread. In such cases material should be dumped in one spot. Truck forward motion while placing ("tailgating") is sometimes done to aid in spreading the material. Such truck spreading action is very conducive to segregation and should not be allowed.

Often a contractor will use the compacted base as his haul road. In such cases the material can become abraded or contaminated at the surface by mud or clay from truck tires. Care should be taken to ensure that both the stockpile area and haul road to the project are out of the mud. Using the base material as a haul road has the advantage of proof rolling the base which can detect soft spots. All such contaminated areas or soft spots should be repaired prior to project completion or paving.

4.7.2 Spreading Operations

Often, base course design depth is such to require placement in more than one layer. Most SHA specifications require more than one lift if the required base course compacted depth is greater than 6 inches. It is good practice to place the lower layers of the base course wider than the upper layers to ensure there is not excessive drop-off which may cause sloughing and poor compaction along the edge. Generally, the lower base layer should be wider by twice the thickness of the top layer.

It is also good practice to ensure proper layer edge compaction to construct at least a 3-foot width of shoulder as a header and compact them simultaneously with each layer of base material. However, care must be taken not to mix shoulder and base materials when they are not alike.

Depending on the size and nature of the highway project many methods and equipment types may be used to spread the material. Common methods are:

- o motor grader
- o spreader box
- o automatic grader/spreader

Although poor practice, often on small projects or areas, truck-end dumping and spreading with a motor grader is the only feasible method. This method is very conducive to segregation and drying of the material and should be avoided if possible. In addition, the dumping of material and spreading with a motor grader causes nests or hard areas where the material was dumped and run over by the motor grader. These hard areas and adjacent soft areas are not conducive to uniform compaction under subsequent rolling. Careful measurement of loads required and load placement is essential to minimize the distance the material piles must be moved and to ensure proper lift thickness with minimum effort.

The most efficient method for high production work is to spread the material with a mechanical spreader. Mechanical spreaders are preferable because of:

- o high spread productivity
- o uniform depth placement
- o uniform width placement
- o minimum segregation
- o conducive to uniform compaction
- o minimize material movement
- o reduce moisture loss

The Jersey mechanical spreader is simply a box on wheels with a strike-off blade to control the thickness and is pushed by a dozer. Trucks end dump on the subgrade into the spreader box. The dozer then pushes the box forward with the strike-off blade spreading to the proper depth. Spread width is about 10 feet with 1-2 foot extension available on either side. The depth of spread is simply controlled by raising the height of the box and strike-off blade relative to the subgrade. It is important to keep the spreader box at least half full between trucks; otherwise, segregation may result and control over alignment and thickness is reduced.

A mechanical automatic spreader is a large piece of equipment which has the advantage of handling very large quantities of material, spreading over a large roadway width, and being fully automatic as far as control of vertical and horizontal alignment. It, therefore, is advantageous to use on very large projects with considerable base material placement. When utilizing an automatic spreader, a large supply of material delivered to the job is necessary in order to take advantage of its high production.

Pugmill blending and hauling operations must be adequate to supply necessary material quantities. Likewise, adequate compaction equipment is required to keep up with placement and minimize material drying. Material is spread evenly over the full width of the machine by screw-type blades which also cut or spread the material at the required depth. Ensuring adequate width and depth of material is subject to adequate supply, placement, and feed of material into the machine.

The automatic precision guidance system for elevation and alignment of the machine is provided by the electric sensor system. This system consists of a nylon or wire string adjacent to the roadway accurately set as to alignment and grade relative to that of the base material. An electric sensor device attached to the machine tracks on this wire and controls and varies the alignment and elevation of the machine.

This system can be extremely accurate but is subject to the accurate surveying and setting of the string control line. It is imperative that the setting of this control wire be set as close as possible before actual spreading operations. Subsequent to compaction of the base course, automatic spreaders have the additional advantage of being able to trim the base very accurately to final grade.

5.0 GRANULAR BASE COURSE COMPACTION

The importance of compaction to the load-bearing capacity of a granular base cannot be overemphasized. A 10 percent increase in a materials percent solids from 75 to 85 percent can easily result in a five fold increase in CBR stability or support value from 50 to 250 percent. Base material must therefore be compacted on the roadway to the degree defined by SHA specification so that appreciable additional densification will not take place under load.

5.1 FIELD COMPACTION AND MOISTURE SPECIFICATION

There are three generally used methods for specifying minimum requirements for compaction. These are:

- soil density control
- compaction effort control
- combination of soil density and compaction effort control

Each method can be made to produce satisfactory compaction if it controls soil moisture along with other material mix properties.

Soil Density Control

Control of soil density is probably the most desirable method and that specified by the majority of SHAs for high type roadways. This is done by specifying and controlling the field-in-place dry weight per cubic foot and also specifying a maximum and minimum limiting value of moisture content as percent of specified target value.

In most cases, it is specified that maximum density and optimum moisture (target density and moisture) be determined for the base material according to AASHTO T99 or T180. Thus, an SHA specification will specify minimum field compaction of say 95 percent of AASHTO maximum test density (T99 or T180) and a moisture content of 90 to 110 percent of optimum moisture content. In place base course material density tests are required at a frequency rate specified in the SHA's material testing manual. Base course lifts not meeting density specifications should be recompacted and/or reworked until they meet specification requirements.

Compaction Effort Control

Only specifying or controlling compactive effort is also used to control compaction. This method is often referred to as the "ordinary compaction" method and is generally used on low volume roadways. Of the three methods it is the least desirable because of the lack of control exercised and the lack of knowledge of actual field density obtained.

In this method, equipment which must be used is normally specified and the base layers are required to be compacted until there is no further visual evidence of consolidation. Additional water and mixing of the material may take place based on visual observations to the satisfaction of the SHA engineer. It is basically an eye-ball method with little actual testing. When used, there should be good documentation of the compaction equipment used, that the equipment was inspected and found to meet specification, and the extent of lift-rolling.

Soil Density and Compaction Effort Control Combination

The third method is a combination of density control and compaction effort control and known as the control strip technique. This method is especially suited to granular base materials which are uniform in composition as opposed to subgrade material which is variable.

This method utilizes a field test method to establish satisfactory construction procedures and acceptance criteria. It does not depend on the questionable correlation of the results of laboratory impact test methods with compaction by field equipment. It is especially suited to granular base course material where impact tests are most limited.

In this method a control strip from the source of material to be used and the same thickness as the layer to be constructed is continuously compacted until no appreciable increase in consolidation takes places as evidenced by nuclear gauge readings.

Moisture content is also controlled within percentages of optimum. Control strip density tests are taken and an average target density determined based on the average of several control strip nuclear gauge readings. It is then required that each section (lot) of a lift be compacted within a percentage of this control strip density based on several nuclear gauge readings.

For best results and control either the soil density control method or control strip method should be used on high type roadways. The ordinary compaction method is not normally recommended for Federal-aid work and should only be used on very low volume roadways.

An area engineer should know which compaction method is specified. If the soil density control or control strip method are used, the area engineer should know target density and moisture. Active base course construction and testing procedures should be observed if possible. Density tests records and test reports should be reviewed for frequency of tests obtained and compliance with specifications.

5.2 LIFT DEPTH

In addition to material specifications and density control specifications, most SHA's also specify the maximum lift depth for each base course layer. Control of lift thickness is very important because it is an important factor governing the degree of compaction that can be obtained in the lower portion of a thick lift. This is due to the compaction equipment pressure decreasing with depth below the lift surface. Also accurate measurement of density obtained is not as easily determined on thick lifts.

Many of the difficulties of obtaining the desired compaction can be traced to lift thickness in excess of that which can be handled by the rolling equipment used. For this reason specifications generally state that lifts should not exceed 6 to 8 inches compacted depth (10-12 inches uncompacted) depending on the material and type equipment used. Such a specification should be considered general with some rolling equipment requiring thinner lifts and some allowing for thicker lifts. It does not mean that in all cases the lift thickness should be the maximum.

The most suitable lift thickness should be determined in the field based on the material and equipment. If smooth steel wheel rollers are used, lift depth might be on the lower side; if a vibratory roller is used, lift depth may be in the higher range of 8-10 inches. An area engineer should be aware of roller compaction abilities and ensure during project inspection that lifts are being placed to the proper depth.

5.3 COMPACTION EQUIPMENT

The success, economy, and ease of obtaining specification density depends in large measure to the proper selection, type, and weight of equipment used for rolling granular base material. There are four types of rolling equipment in general highway construction use:

- sheepsfoot roller
- rubber tired roller
- steel wheel rollers
- vibratory rollers

Sheepsfoot Roller

Sheepsfoot rollers are generally not recommended for compacting granular base course materials because the penetration and pressure exerted by the feet tend to degrade the course stone material in the base course. Also, the action of the feet pulling out of the granular material tend to catch and pull out the course material, thereby tearing it up rather than compacting the base course. Sheepsfoot rollers are conducive to compacting non-granular silty clay materials found in subgrades or stabilized non-granular materials used in base courses.

Rubber Tired Rollers

Rubber tired rollers are quite common and recommended for granular base course construction. They are excellent for initial breakdown and subsequent base course layer rolling because of the compaction and kneading action which takes place knitting the granular particle surfaces inside the lift into a dense and uniform mass.

Rubber tired rollers can vary in weight from 3 to 35 tons by adding ballast in the form of sand or water. Weights in the 7 to 15 ton range are common for highway work. The number of wheels may vary from 7 to 9, with the steering axle having the fewest wheels and the drive axle having the most.

Rubber tired roller compaction is dependent on the contact pressure between the tires and layer surface which approximately equals the tire inflation pressure. Tire inflation pressure should generally be kept between 60-90 psi for granular base material. In order to assure that roller compaction is uniform, it is important that all the roller's tires be inflated to the same level. Changing the roller weight does not change tire (contact) pressure and thus does not modify the ability to compact. Changing roller weight only changes the tire contact area and thus influences the depth of compaction or effective lift depth.

Thus, for a given tire pressure and number of passes, increasing the load will not increase density but will increase depth of compaction. Thus, one would expect a rubber tired 8-ton roller to be used for thinner lifts of say 6-8 inches and a 12-ton rubber tired roller to be used in conjunction with lifts of say 10 inches.

Steel Wheel Rollers

Steel wheel rollers are more suited for compacting silty material; however, they are reasonably effective in compacting granular base course when the maximum particle size is less than 1 to 1-1/2 inches and the base is uniformly spread or graded. Smooth steel wheel rollers tend to bridge over large aggregates causing non-uniform compaction. They also have a tendency to bridge and result in non-uniform compaction if there are ruts. Thus, they should only be used if the surface is spread smoothly and unrutted.

Because of this bridging action and since they compact from the top down, generally lifts should be kept on the low side of the lift range or say 6-8 inches. Smooth steel wheel rollers are especially good for finishing rolling to imbed the course particles and obtain a smooth flat surface. Such rollers can be either two-or three-wheeled and are generally in the 6-15 ton range with pressures of 200-400 pounds per linear inch of roller.

Steel wheel rollers weight is changed by filling the drum with ballast such as oil or water and are designed to operate either empty or full. Thus, an 8-12 ton roller may be either 8 or 12 tons and should not normally be filled partially to obtain a weight in between. If 10-ton capacity is desired then one with a 10-ton empty or full capacity should be obtained.

Vibratory Rollers

Vibratory rollers work on the principle of particle rearrangement to decrease the voids between particles causing an increase in density. They are, therefore, excellent and efficient rollers for granular material. They have the distinct advantage in that specifications may allow thicker lift placement. Granular base course lifts of 15 inches are possible.

The compaction and efficiency of vibratory rollers are influenced by three factors:

- o vibration unit
  - amplitude
  - frequency
- o speed
- o static drum weight

The vibratory unit consists of a shaft with off-center weights, called eccentrics, spinning inside the roller drum.

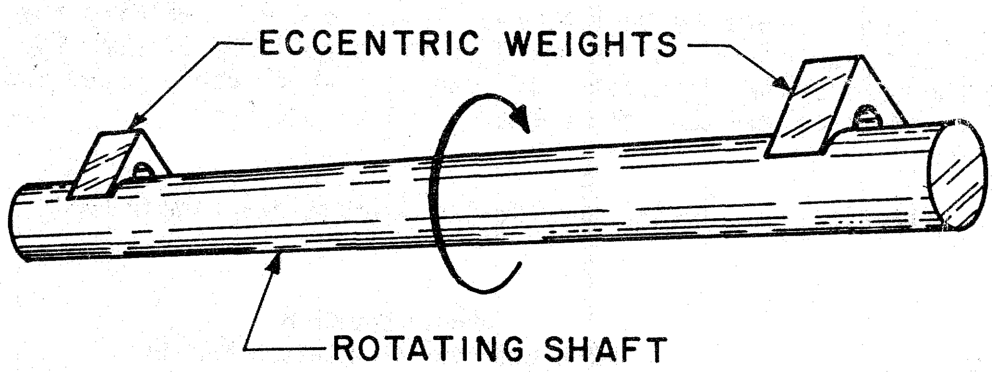


FIGURE NO. 5-1 - Schematic of Vibratory Shaft

The amplitude of the vibrating drum is represented by the drum lift or total vertical motion and is measured in fractions of an inch. Realizing that the actual force hitting the ground is a function of falling height and drum static weight, one can visualize the importance of amplitude. Amplitude primarily effects the depth of lift which may be compacted. Thus, if the granular material lift thickness is on the low side (6 inches) then low amplitude should be used. If the lift depth is high (15 inches) a high amplitude should be used.

Vibratory rollers tend to compact non-cohesive materials like granular base course from the bottom up. If high amplitude is used with thin lifts, the compaction may actually be taking place in the subgrade rather than in the base material. This may overcompact the subgrade forcing excess moisture and fines into the base. This may alter base gradation, permeability or other important characteristics. The subgrade may actually be lower in density due to over compaction. A vibratory roller should never be used on base material placed on a stabilized subgrade. The impact force of the vibratory roller will have a damaging effect on the stabilized subgrade by inducing cracking.

Generally, amplitude is set at low, medium or high with low amplitude one half the high amplitude. Amplitude is varied by increasing or decreasing the distance of the eccentric weights from the shaft. If the weights are close to the shaft as it rotates, the vertical distance the roller moves is small. By moving the weights farther away from the shaft, the force of the vibrations increase and the greater the vertical movement of the roller.

Frequency is the rate at which the drum vibrates and is a function of the speed at which the shaft spins. Each time the shaft makes a complete revolution, it causes a vibration. The speed of vibrations is called frequency, and is measured in vibrations per minute. Vibratory rollers are available from 800 to 4000 cycles per minutes frequency, with 1000 to 2000 common for highway work. Frequency is controlled independently from both amplitude and roller forward motion.

The impact spacing influences the number of passes required and the degree of compaction. Impact spacing is a function of roller frequency and speed as shown in Table No. 5-1.

Generally, speeds should be kept between 1-1/2 to 3 mph (walking speed) resulting in impacts per foot in the 7 to 16 range. The greater the impact spacing the more passes required. If frequency is reduced, speed must be reduced in order to maintain the number of blows per foot.

TABLE NO. 5-1 - Vibratory Roller Impacts Per Linear Foot

Frequencies Cycles per Minute	Roller Speed in Miles per Hour and Feet per Minute				
	1.5 (132)	**2.0 (176)**	**2.5 (220)**	**3.0 (264)**	**3.5 (308)**
1000	7.6	5.7	4.5	3.8	3.2
1200	9.1	6.8	5.5	4.5	3.9
1400	10.6	8.0	6.4	5.3	4.5
1600	12.1	9.1	7.3	6.1	5.2
1800	13.6	10.2	8.2	6.8	5.8
2000	15.2	11.4	9.1	7.6	6.5
2200	16.7	12.5	10.0	8.3	7.1
2400	18.2	13.6	10.9	9.1	7.8
2600	19.7	14.8	11.8	9.8	8.4
2800	21.2	15.9	12.7	10.6	9.1
3000	22.7	17.0	13.6	11.4	9.7

Frequency + speed in ft. per min. = Impacts per linear foot

An area engineer needs to be aware of the ability of rollers to compact and their limitations in order to address field compaction adequacy and problems. SHA project personnel should inspect the compaction equipment and check roller weight, tire pressure, and compaction procedures. The FHWA area engineer should check that SHA personnel have conducted such an inspection initially.

5.4 COMPACTION PROCEDURES

5.4.1 Rolling Pattern

There is no criteria on a desirable rolling pattern for granular base course material similar to that for asphalt compaction. From a practical standpoint rolling takes place longitudinally behind the spreader operation. Rolling should preferably work from free edges toward the center, and each pass should overlap by 1/3 to 1/2 the roller width. On superelevated curves it is preferable to work from the low edge toward the high. Self-propelled reversible type rollers are preferable over the pulled type. Pulled rollers are difficult to control and have a tendency to cause hard spots at the turn around points causing non-uniform compaction.

With proper equipment, specification density is attainable in 3-5 passes. An excessive number of passes is uneconomical and usually indicates a problem and corrective action should be sought. In addition, over-rolling usually requires extended surface watering to prevent surface drying. This over-rolling in combination with watering should be avoided as it can flush a "later" of fines to the surface and fail to increase density. an excess of surface fines at the interface between the base and surface can be conducive to premature pavement problems.

5.4.2 Moisture

As previously indicated, moisture content should desirably be at optimum. Most specifications allow it to be within a certain percentage above or below optimum. Since drying occurs during hauling and surface drying occurs during rolling due to wind and hot weather, it is sometimes better to be slightly above optimum moisture. Generally, a water distributor sprinkles the base just ahead of the roller. This is only effective in restoring moisture loss from surface evaporation since water will only penetrate a short depth.

Because of the uniformity of blended granular base material, excess or insufficient moisture is generally not a significant problem. However, when soil moisture exceeds that needed to obtain required density, the moisture contents must be reduced. Drying great quantities of material is at best slow and costly. Most drying is done by air drying which relies on aeration and exposure to the sun to remove excess moisture. Manipulation and exposure speeds up the drying process and can be done by cultivators or rotary mixtures with their top hood section raised. Also, drying can be expedited by spreading and blending in dry but similar material during the mixing and aeration process. In a similar fashion moisture may be added by adding water to the grade by a water distributor and mixing it with a disc or rotary mixer.

5.4.3 Blading

Although pneumatic tire and steel wheel rollers are conducive to a smooth surface, some surface irregularities exist during rolling especially after initial passes. Consequently, the surface is periodically lightly bladed with a motor grader during rolling to maintain a smooth surface, free from waves and ruts to enhance uniform roller compaction and maintain the roadbed template. This provides for proper run off and prevents water contamination.

5.4.4 Scarifying

Often during the rolling process, especially if the surface is overrolled, surface fines become present. Such fines should be mixed back in with a light scarifying and recompacted prior to application of a prime coat or subsequent layers. In addition, if more than one base course layer is required, the contact surface should be lightly scarified prior to spreading the next course to key the layers together.

5.4.5 Trimming

Finish rolling should be accompanied by light trimming to assure close adherance to specified tolerances with respect to grade and crown. During this operation SHA personnel should make frequent stringline measurement checks of the grade elevation. On large projects, automated fine graders with electronic control are highly recommended for fine trimming.

6.0 JOB CONTROL SAMPLING AND TESTING

6.1 TARGET DENSITY AND MOISTURE TESTS
6.2 GRADATION
6.3 FIELD DENSITY
6.3.1 SAND-CONE METHOD
6.3.2 WATER BALLOON METHOD
6.3.3 NUCLEAR GAGES
6.4 MOISTURE TESTS
6.5 SAMPLING

A project inspector must check all material incorporated into the project. To do this, he must be knowledgeable of the following:

- material to be tested
- required tests
- test frequency
- sample methods
- test methods
- sample location
- reporting requirements

In exercising materials control, it is essential that it be applied uniformly by all inspectors from project to project so that all contractors are treated alike. Therefore, SHA's establish minimum guidelines for inspectors to follow which provide them with the needed information on the above items. As an example, Figure No. 6-1 indicates the AASHTO Construction Manual for Highway Construction schedule for job control sampling and testing of aggregate base material.

Each SHA will have its own FHWA approved materials testing schedule which an FHWA area engineer should be familiar with to ensure the prescribed testing is done. He should also have knowledge of the test and sampling procedures and reporting requirements. A basic knowledge of these items will assist the area engineer in his review of base course material project control and enable him to discuss and have credibility with SHA personnel when conducting his reviews.

Type of Construction	Material	Test	Job Control Samples & Tests	Central Laboratory Samples	Sample Size	Procedures		Remarks
						Sampling	Field Testing	
Section 300 – Bases (Cont.)								
304 Aggregate Base*	Aggregate	Gradation	1/1,000 Tons	1/Source	30#	T-2	T-27	
		Moisture-Density	1/20,000 Tons	1/Source	30#	T-2	T-99 or T-180	If used for control
		Density	1/Layer/¼ Mi			S	T-191 or T-238	
		Moisture	1/Layer/¼ Mi		1#	S	S	
	Binder Soil	Gradation	2/Source	1/Source	10#	T-86	T-27	

FIGURE NO. 6-1 - AASHTO Construction Manual Schedule For Job Control Sampling and Testing

The field control tests normally run on granular base material are:

- o target density/moisture
- o gradation
- o moisture
- o density

In the following subsections, we shall briefly cover each of these along with recommended sampling and typical reporting formats.

6.1 TARGET DENSITY AND MOISTURE TEST

To make sure adequate base compaction has in fact been achieved, it is necessary to measure the in-place dry weight density and to compare this measured value against a specified standard or "target" density established by a standard test. The most commonly used standard test was originally developed by R.R. Proctor in 1933. With some changes, his test has been adopted by AASHTO in two forms known as:

- o AASHTO T99 (standard proctor test)
- o AASHTO T180 (modified proctor test)

Both tests are essentially the same type of dynamic compaction test except the modified tests provides more compactive effort and thus results in dry densities 3-6 PCF greater than the standard T99 test. In addition, both tests provide for four separate subprocedures.

A - 4-inch mold, soil < No. 4 sieve
B - 6-inch mold, soil < No. 4 sieve
C - 4-inch mold, soil < 3/4" sieve
D - 6-inch mold, soil < 3/4" sieve

Since granular base course material generally has a maximum size greater than 3/4", subprocedure C or D is generally used for base material. Material greater than 3/4" in the mixture sample should be replaced by an equal amount of material between the 3/4 inch and No. 4 sieve size. Figure No. 6-2 shows the compaction equipment for the standard (T99) and modified (T180) test and Table No. 6-1 indicates the characteristics of the two test methods.

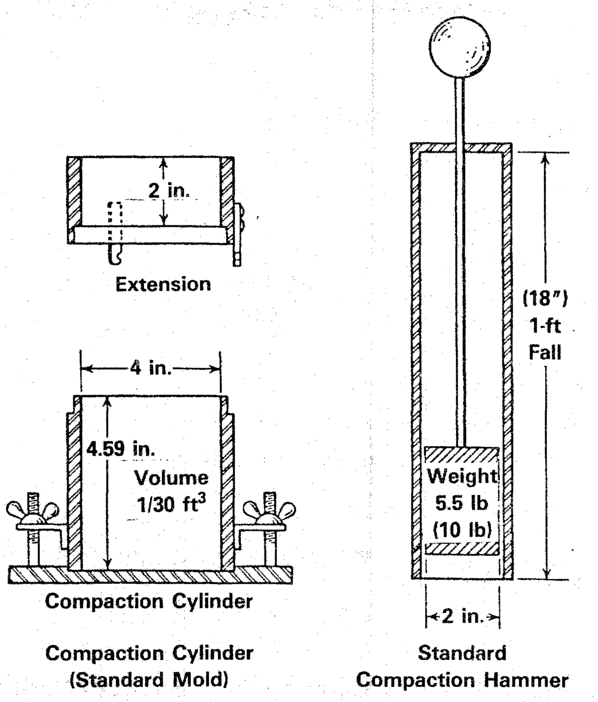


FIGURE NO. 6-2 - Compaction Equipment For AASHTO T99 and (T180) Density Test

TABLE NO. 6-1 - Characteristics of T99 and T180 Density Test

TEST	MOLD SIZE	HAMMER HT./WT.	NO. LAYERS	BLOWS/ LAYERS	COMPACTION FT-LB EFFORT FT^3
T99	4 X 4 1/2	12"/5.5#	3	25	12,400
T180	4 X 4 1/2	18"/10#	5	25	56,300

A single sample of the blended base material meeting the specifications, is blended with incremental moisture contents above and below optimum and tested according to T99 or T180 (whichever is required by the specifications.) This results in a dry density unit weight for each moisture content which are plotted into a moisture content which are plotted into a moisture density curve similar to Figure No. 6-3. The point on the curve marked A corresponds to the maximum dry density and optimum moisture content and becomes the target density/moisture content for the material. The drop hammer can either be the manual type depicted in Figure No. 6-2 or a mechanical type. In either case, it is imperative that the mold be supported in a dense, rigid stable base such as a 200-pound block of concrete, a concrete slab, or abutment etc.

An adequate sample size to run the test (about 30 pounds) should be obtained. The sample should be obtained initially once uniform acceptable material meeting specification has been obtained in the production process and periodically thereafter as indicated in the materials testing scheduled or whenever there is a significant change in material.

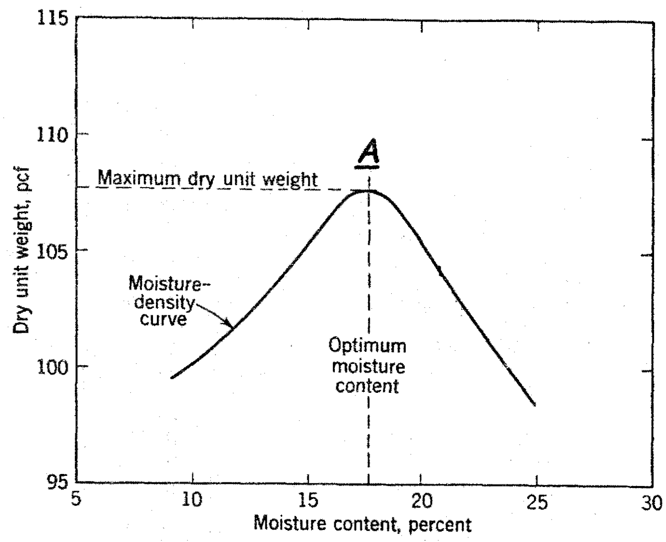


FIGURE NO. 6-3 - Typical Moisture Density Curve

6.2 GRADATION

As discussed in Section 3.2, gradation has an appreciable effect on dry weight densities of soil materials. Laboratory tests have shown dry weight density to vary as much as 10-15 pounds per cubic foot as fine aggregate content was varied from 25-50 percent. Hence, if the gradation of the in-place spread material is different from the gradation of the material used to establish the standard density, the specification density could be unattainable or else be too easily attained, permitting acceptance of inadequate compaction and lower stability.

It is for this reason that so much emphasis is placed on a consistent gradation of base material being hauled to the grade, and/or careful handling to minimize segregation during spreading, and that SHA specifications require gradation of the material. Base course particle size distribution is typically determined by conducting a field gradation analysis as described by AASHTO T27.

Some SHA specifications require the contractor to submit for approval a mix design gradation which conforms or falls within the specification grading band. Once the job mix gradation is established and approved, specifications apply production tolerances to the percent of material passing sieves. This job mix tolerance establishes the master range or job control grading band within which acceptable material tests results must fall.

Should a test result fall out of the master range, it is grounds for concern, possible rejection, and corrective action should be sought. Gradation test results should never be allowed to continuously fall outside the master job control range. Figure No. 6-4 shows a typical gradation chart with the specification grading, job mix, and job control band shown.

Gradation tests samples should preferably be taken at the jobsite according to the SHA's job control testing manual frequency. The area engineer should attempt to witness sample taking and testing on each project whenever possible.

In addition, test records should be reviewed for material acceptance and test frequency.

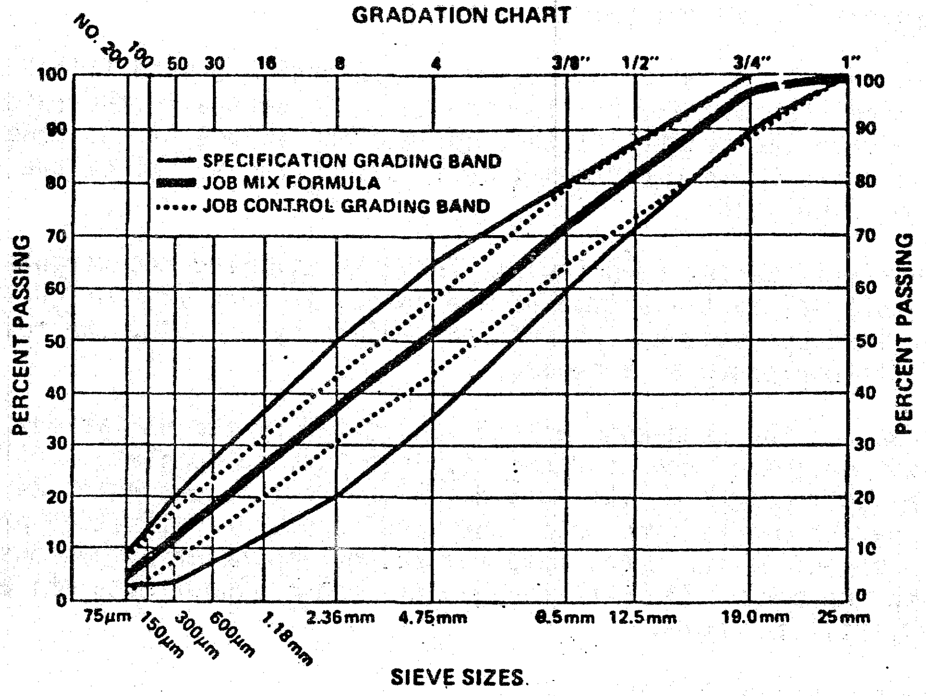


FIGURE NO. 6-4 - Typical Gradation Chart Showing Specification And Control Grading Bands

6.3 FIELD DENSITY

As previously covered in Section 5.1 the degree of field equipment compaction is determined by comparing field dry density against the target density. There are three generally accepted methods for determining field density:

- o sand-cone method
- o water balloon method
- o nuclear method

All three tests are used in the field; however, the nuclear method, because of its rapidity and accuracy is by far the most popular method.

6.3.1 Sand-Cone Method

Under the test method described in AASHTO T191 a volume of in-place compacted material is removed and its removed volume determined using calibrated sand. The test consists of five basic steps:

1. Selecting and leveling a test site on the compacted fill at least 30 feet away from any operating equipment.

2. The base plate is placed on the site and a hole is dug about 6 inches deep and slightly smaller in diameter than the hole in the plate. Care must be taken to collect all material taken from the hole in an adequate

container that must be sealed. Sides of the hole should be vertical and relatively smooth.

3. The material dug from the hole is weighed.

4. The volume of the hole is measured by inverting a sand filled jar and cone over the hole and filling the hole and cone with the sand in the jar. Since the density of the sand and the volume of the cone and hole in plate are known (previously calibrated), the volume of the hole dug can be calculated.

5. The wet unit weight (density) is found by dividing the weight of the damp material by the volume of the hole. The dry unit weight is determined by dividing the wet unit weight by one plus the moisture content (expressed as a decimal).

The accuracy of the sand-cone method is limited by the variations possible in the unit weight of the sand and its inability to completely fill the test hole. The unit weight of the sand deposited in the test hole can be affected by the height from which the sand is poured, vibration present, moisture content of the sand, temperature and amount of extraneous soil mixed with the sand. The sand's angle of repose also limits its ability to completely fill the hole.

6.3.2 Water Balloon Method

The principle of the water balloon test method is similar to the sand cone method. The test procedure is described in AASHTO T205. The main difference between the two methods is that one uses calibrated sand to determine the hole volume while the other uses water confined in a water balloon. Figure No. 6-5 shows a schematic of the test equipment and principle.

The test consists of the following steps:

1, 2, 3. The first three steps are similar to those described for the sand cone test.

4. The volume of the hole is measured using a balloon densometer which is placed over the hole, with the balloon. A valve is opened forcing the balloon filled with water into the hole. The change in fluid volume is read indicating the volume of the hole.

5. Step 5 is the same as that with the sand cone method.

The accuracy of the method depends on the ability of the balloon to fill the hole. Thus, the hole's sides must be smooth and free of excess indentations. Because of the difficulty in providing a smooth surface with large particle material, its accuracy is less with large-size aggregate.

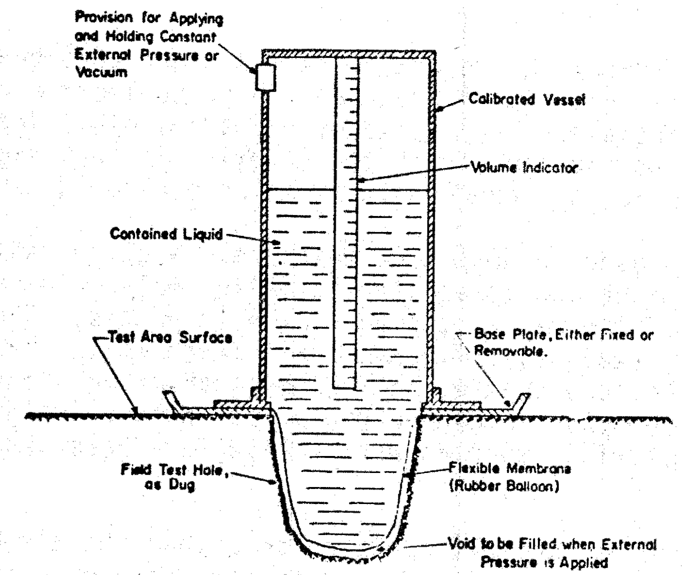


FIGURE NO. 6-5 - Schematic of Water Balloon Test Method

Approximately the same amount of time is required to perform either the sand cone or the balloon test. The major disadvantage of these two methods is the length of time required to perform the test. A single field density determination may take 30 to 60 minutes. This severely limits the number of tests that can be performed without delays to construction. Often, it is not possible to determine if material satisfies density requirements until additional lifts of material satisfies density requirements until additional lifts of material have been placed over material that has been tested. For this reason, nuclear density field determination of density has gained widespread acceptance among SHA's.

6.3.3 Nuclear Gages

The most significant breakthrough in the field of rapid field density and moisture testing has been the use of the nuclear gage method. They are not only far more rapid than conventional methods, but they offer as well a great degree of freedom from human error and require less judgment on the part of the inspector/operator. The major disadvantages are:

- o skilled technicians required
- o high equipment costs
- o time and effort to check and calibrate equipment
- o repair cost and complexity

Still, as SHA's gain experience with them, they have became widely used. Because of their widespread use, it is important for the area engineer to understand their basic operation and test steps.

Theory

Nuclear gages work on the principle of unstable atoms emitting radiation in the form of gamma rays and neutron radiation. This radiation passes through the soil medium which reduces it so that readings are indicative of the soils density and moisture.

Gamma rays have electrical charge but no mass and are absorbed through electrical interaction principles known as the photoelectric effect, compton effect, and pair principle. The degree of gamma ray electrical absorption is directly related to the density of the material in that higher density soils absorb more gamma rays and result in lower nuclear gage reading count.

Neutron radiation is used to determine moisture. Neutron radiation has mass but no electrical charge and loses kinetic energy only by direct collision with nuclei (mass) of the medium through which it passes. The smaller the chemical mass number (weight) of the medium through which the neutron radiation passes, the less neutron radiation is absorbed through collisions and loss of kinetic energy. Hydrogen has a mass number (weight) of one. These hydrogenous materials such as water allow for increased passage of neutron radiation.

Portable nuclear gage devices are operated in either the direct transmission mode or the backscatter mode. The direct transmission mode shown in Figure No. 6-6(a) requires that the probe containing the gamma ray source energy be placed in a tube inserted into small hole in the soil layer. It measures density for predetermined probe depth settings up to about 12 inches. When the probe is retracted even with the gage bottom as shown in Figure NO. 6-6(b), it operates in the backscatter mode measuring density in the top 3-4 inches.

Another separate radiation source is located in the bottom of the gage and emits neutron radiation. This neutron radiation always operates in the backscatter mode measuring the moisture content in the top 3-5 inches of soil aggregate material.

Gage Components

Nuclear gages basically consist of four major components:

- o emitter (probe)
- o detector (scaler)
- o power supply
- o standardization blocks

The emitter or probe is the primary unit of the gage which emits incident radiation. Two different emitters radiation sources are contained in a gage. The source which emits gamma radiation is used to measure density and is located in the end of a height adjustable probe. The source which emits neutron radiation used for measuring moisture is located in the underside of the gage. The emitter for gamma radiation is Cesium-137 which is shielded or encapsulated in lead when not in use. The emitter for neutron radiation is Americum-Beryluim which is encapsulated or shielded by wax when not in use.

The scaler serves as a means of counting the number of varying events occurring the detector tubes as a result of impinging incident and/or reflective radiation. Present day gage scalers almost all read out digitally.

(a) Direct Transmission Mode (b) Backscatter Mode

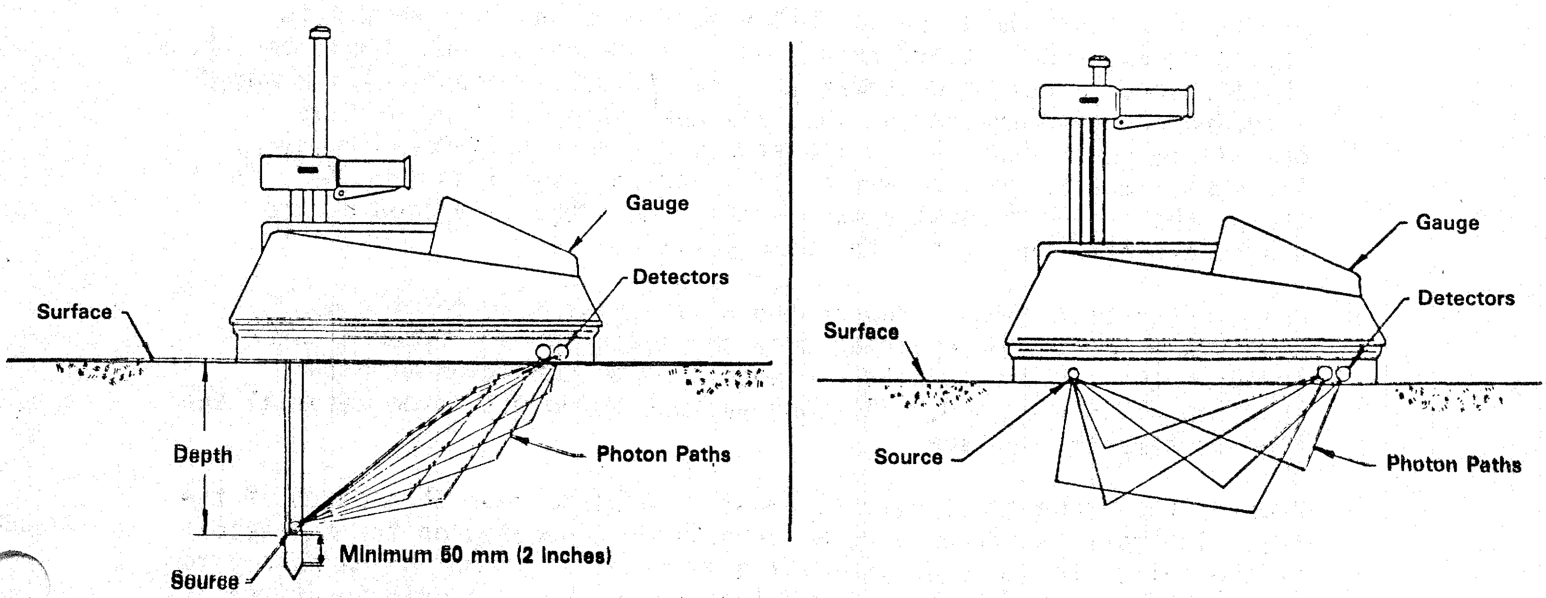


FIGURE NO. 6-6 - Density Gage Geometry

Most power supplies of nuclear gages today may be of three types. These are: an external 12-volt automobile battery, internal batteries capable of operating continuously for 16 hours, and 110-115 volt AC-60 cycle current.

A standardization block is furnished with the gage and is used for determining the standard count in calibrating the gage. This block provides a means of checking the electric circuitry of the gage. Marked deviation of standard count readings are indicative of gage malfunction. Also, standard count readings are used to determine the count ratio which adjust for slight changes in gage electrical circuitry.

Manufacturers supply calibration curves established by determining the nuclear count rate of each of several materials at different known densities plotting the count rate (count ratio) versus each known density and placing a curve through the resulting points. The density of the material used to establish the calibration curve must vary over a range that includes the density of materials that will be tested. Normally, calibration curves should be checked once a year to ensure the curve is representative of the gage. This is necessary because of radiation source aging and gage component age.

Field Procedures

Normal field procedures for using the nuclear gage will involve the following steps. Figure No. 6-7 is a sample field test report data form.

1. The gage should be turned on and allowed to warm up for at least 10-15 minutes before undertaking any tests. Being premature in this may result in erratic counts.

2. Determine the standard count. Since gage readings will vary with location, and local interference, etc., a standard count should be taken at least once a day or whenever its area of use is drastically changed. Standard count determination involves the taking of four or five one-minute count readings on the standard blocks and averaging them. All such readings must be within ± 2 standard deviations ($2\sqrt{n}$). In taking the standard count always ensure the probe is placed on the standard blocks in exactly the same position.

3. Prepare the test site. Preparation of the test site for the gage consists primarily of clearing away the loose surface material and scraping the surface to a fairly smooth plane condition with the leveling plate provided. Fill any surface voids with fine material and lightly tamp the surface.

4. Prepare the probe (rod) hole at least 2 inches deeper than depth of the rod. The hole is either punched or drilled depending on the character of the soil. In rocky or granular type material it may be necessary to drill the 13/16 inch hole. Regardless of whether the hole is driven or drilled, the hole guide on the leveling plate must always be employed.

5. Place the gage on the prepared test site. The most important point to consider is that the gage is evenly placed on the leveled area and that the probe when lowered into the hole is snug up against the hole.

6. Turn the detector on and take a one-minute count for the density and the moisture.

7. Divide the density count by the standard count.

8. Enter the calibration curves and determine the wet density and moisture in pounds per cubic foot.

9. Determine the dry density and percent moisture.

It should be indicated the new modern gages do not require all the above steps as they are almost completely automatic. These modern gages have a "burned chip" with the calibration curve in it so that just pressing the appropriate button will result in a digital readout of the dry density and moisture content.

Report No. 1
Date 5-1-74 Route No. 29 Project No. 0029-039-101, C501
F.H.W.A. No. None County Greene
Section No. 1 Station 958 + 00 to Station 984 + 40
Type Material Subbase Width 26'
Remarks

STANDARD COUNT

	DENSITY		MOISTURE
	265		1418
	263		1425
	261		1427
	261		1413
Total	1050	Average 263	Total 5683 / Average 1421

TEST NO.	STATION	LANE	DENSITY, Count	MOISTURE, Count
1	958 + 50	NBL	322	816
	C/R		1.224	.574
2	964 + 50	NBL	324	816
	C/R		1.234	.574
3	970 + 50	NBL	322	881
	C/R		1.224	.620
4	976 + 50	NBL	330	830
	C/R		1.255	.584
5	982 + 50	NBL	329	835
	C/R		1.252	.588

TEST NO.	Wet Density, lbs./cu. ft. –	Moisture, lbs./cu. ft. =	Dry Density, lbs./cu. ft.	Moisture, %	Optimum Moisture Req'd., %	Max. Density Required, lbs./cu. ft.	Passing	Failing
1	145.0	11.8	133.2	8.9	8.0	127.3		
2	144.0	11.8	132.8	9.0	8.0	127.3		
3	145.0	13.2	132.3	10.0	8.0	127.3		
4	142.0	12.2	129.8	9.7	8.0	127.3		
5	142.3	12.3	129.1	9.6	8.0	127.3		
AVERAGE			131.3		8.0	131.3		

FIGURE NO. 6-7 - Field Test Report Data Form

6.4 MOISTURE TESTS

As previously covered in Section 5.1, the degree of field compaction is accepted based on field density and controlled by field moisture tests. Field moisture content determinations usually accompany field density measurements as density samples must be dried to determine dry density. AASHTO tests indicate that the sample be dried in a convection oven at 110°C for 12-15 hours or to a constant weight to determine moisture content. Because of the time delay involved with the standard oven method, it is generally not acceptable in the field. Consequently, in many instances other more rapid methods of determining moisture content are employed.

One such method is the "speed" moisture tester for determining moisture content. This test can be done in about 3 minutes on most materials, a fraction of the time it takes for the standard convection oven. Because the AASHTO T217 speedy moisture 5 gm and 26 gm sample test is limited to a maximum 3/4" size aggregate, the 200 gm test is sometimes used for base course material.

In this test the soil-aggregate sample and calcium carbide are placed in the test container and sealed. The container is then shaken and the moisture in the sampler reacts with the carbide to form a gas. The pressure generated by the gas is converted to read percent moisture in the sample on a calibrated gage. Procedures for the 200 gm speedy moisture test are contained in FHWA Notice N.5080.19, Rapid Test Procedure RT-15. There are disadvantages to this test such as:

- o test equipment cost
- o test equipment size and weight
- o questionable readings below 60°F
- o questionable readings on some material composition

Anytime a speedy moisture test is employed, test results should be periodically checked against standard convection-oven results.

The use of microwave ovens to dry aggregate soil samples has been found to be a practical and efficient method of determining moisture content. Test procedures are contained in FHWA Notice N 5080.13, Rapid Test Procedure RT-12. It has been found the microwave oven procedures result in approximately the same level of accuracy as the standard convection oven with the length of drying time cut to 4-20 minutes. The actual drying time will vary with the rate of heating of the material which varies with chemical and mineralogic composition, density, specific heat and voids etc. On a trial basis project personnel will learn the time required to dry a material to a constant weight.

Microwaves can either be reflected, transmitted or absorbed by a material. Generally, metal will reflect the energy, and glass, porcelain, paper and plastics are transparent to the energy. Emitted microwave energy is absorbed by the polar molecules and causes them to oscillate at very high speed producing intermolecular friction which produces heat. In the case of water moisture, the generated heat causes it to evaporate.

Some aggregate and soils, such as those containing iron oxide and carbon, may react to microwave energy and in some cases explode. Operators should, therefore, never leave the oven unattended. It is good practice to heat the material in time increments observing its performance with each heating increment until the time required to dry the constant weight is generally established. Once this minimum time is established, a similar sample of same material could then be dried for this length of time without continuous observation of the drying process. Generally heating on a lower cycle for 2 to 3 minute intervals is best until constant weight is obtained.

If problems such as aggregate overheating (turning red-orange with heat), ignition of organic materials, or arcing are observed while drying as the point of zero moisture is approached, the operator should immediately turn off the oven.

Another method is to heat the sample in a pan over an open flame or burner until a constant weight is obtained. When employing this method, the heating should be done gradually to avoid overheating. Also, often the operator stirs the mixture, so that an accidental spillage is always a danger. In both the microwave and open burner method calculated dry density varies slightly. This is one reason why results may differ from dry weights obtained using the conventional convection oven at 110^{o}C.

Nuclear gages are also commonly used today to determine moisture content as covered in Section 6.3.

The key to base course material compaction acceptance is density and moisture checks for control purposes to ensure moisture is in the proper range to allow attainment of specification density. Thus, while the above methods may not be as accurate as the AASHTO prescribed method, they are generally satisfactory for control purposes. However, if moisture results continuously run on the upper or lower tolerance range, corrections should be made to bring the moisture content closer to optimum also, field testing results should be checked against laboratory methods.

6.5 SAMPLING

Proper sampling techniques are as important as the testing and the sampler should use every precaution to obtain samples that represent or depict the process, nature and condition of the material which they represent. Since production, transportation and material placement may cause temporary or permanent segregation as well as moisture loss it is imperative that an Area Engineer recognize good sampling techniques.

Since material meeting specifications at the plant may be hauled and handled carelessly so as to cause it not to meet specifications after placement, the most logical place to take acceptance samples for gradation and moisture is on the grade. However, sometimes this is not practical due to the number of tests, and laboratory and project locations. Therefore, the material production is sometimes tested and controlled at the plant. However, in such instances periodic samples should be obtained after spreading to ensure the product as delivered meets specifications.

There are various points from which to obtain samples and associated good methods for obtaining them. These locations are:

- belt feeds
- stockpiles
- bins
- trucks
- grade

AASHTO T2 gives various guidance regarding the best sampling methods. In taking samples at these points, care must be exercised that the sample is as representative as the sampler can secure with reasonable effort. Since proper sampling is just as important as proper testing it is wise for the Area Engineer to be aware of these sampling techniques.

Sampling from stockpiles should be avoided whenever possible. This is due to the segregation which usually occurs when material is stockpiled with the coarser particles rolling to the outside base of the pile. Good sampling from a stockpile can only be done if every portion of the pile can be reached inside of the pile as well as on the outside. If this cannot be done then the gradation sample is doubtful. However, samples can be taken from the "face" of a stockpile as material is used from it. When it is necessary to sample a stockpile, power equipment is essential to expose the interior to the material at various locations and levels.

Belt conveyors furnish a handy point of sampling. It is necessary to stop the belt and remove a 2-foot long sample of material. The area is defined by inserting a template conforming to the possessive belt's shape and removing all the material from the belt being careful to collect all the fines. At least three such increments from the unit being sampled should be removed and combined to form the sample.

Assuming that the bin has been filled by discharge from chute or screen over the middle of the bin, the sample is finer when the gate is first open due to the coarser particles rolling to the sides, leaving the finer particles in the center. Consequently, securing a representative sample is impossible without completely unloading the bin. However, if it becomes necessary to sample from a bin, a relatively large volume from the full gate discharge cross-section should be released, and "quartered" down to the desired size.

In loading trucks, the coarse particles roll to the side and bottom of a cone-shaped pile and the finer material tends to remain near the center. Since trucks are self propelled and carry relatively small quantities of material there will seldom be an absolute need to sample from them. Therefore, it is more advantageous to dump the pile and treat it like a small stockpile in obtaining a representative sample. Usually one side of the pile will be finer and the other side (away from the position of the trucks) will be coarser.

Sampling from the spread base course may be done by taking at least three approximately equal increments and combining them to form a field sample of appropriate size. Each increment should be taken from the full depth of material taking care to exclude underlying material.

7.0 STABILIZED BASE COURSES

7.1 TYPES

A major concern in recent years has been the shortage of aggregates in both those areas that normally have adequate supplies and in those areas that have never had good quality aggregates available. One way to solve this problem is to use otherwise marginal or unsuitable aggregate soils which have been modified or stabilized to obtain desired qualities. Care must be taken to ensure that proper engineering materials design is performed when using marginal aggregates.

In its broadest sense, soil stabilization may include any mechanical or chemical treatment which makes it more stable and suitable for use. Such actions as blending (gradation change), compaction, and the addition of chemical modifiers are considered stabilization. However, soil stabilization has generally come to refer to the process of stabilization through the addition of chemicals.

This section will deal with the three most widely used chemical stabilization elements:

- cement
- lime
- asphalt

Calcium and sodium chloride are also used as chemical stabilizers to cause aggregate soils to retain moisture, thus reducing dusting and fines loss on unsurfaced roads. However, since they are not generally used for base courses they will not be included in this section.

The various types of chemical stabilizers have been classified according to the property which it imparts to the soil. Classifications applicable to a base material are:

Classification	Function	Stabilizer
Cementing Agent	Increasing strength by chemical action	Cement Lime Lime Fly ash Bitumen
Modifiers	Improve plasticity (may or may not improve strength)	Cement Lime
Water Proofers	Little or no increase in strength	Bitumen

The main function of cementing agents is to form a cementitious compound that more or less permanently bonds together individual large and small size particles of aggregate and soil. Cementing agents thus increase the material's stability or CBR greatly. When a cement agent is applied the material becomes much like concrete except its strength is much less and voids are not filled with cement paste. Instead, the particles are cemented largely at the contact points.

Many times the use of a cementing material is restricted because of cost and, therefore, low quantities of the material admixture may be added to the aggregate soil to merely modify it. Soil modification generally involves borderline base material such as pit run that contains excess fines resulting in high PI and volume change susceptibility. Cement and lime will change the water file on the soil particles, modify the clay minerals to some extent, decrease the soil PI, and reduce volume change susceptibility.

Small amounts of bituminous materials are used in low-grade aggregates to coat the fine particles and retard moisture absorption of the clay fraction. The asphalt thus functions as a waterproofing stabilizer.

7.2 SELECTION AND PROPERTIES

Ideally, field tests should be performed to determine the type or types and characteristics of in-place material as well as those of available borrow material. Laboratory tests should be performed to determine the engineering properties of mechanically stabilized and chemically stabilized soils and borrow. Cost and energy comparison analysis associated with selected material use and stabilization will indicate permanent material selection, design, and stabilization. Except for very large projects this desired engineering approach is rarely done. Therefore, simplified guidelines have been established to indicate those chemical stabilizers which appear most suitable for the particular situation.

One method modified after original works done by the Bureau of Public Roads is shown in Figure No. 7-1. This method utilizes the PI and percent passing the No. 200 seive together with the AASHTO Soil Classification System for the purpose of stabilizer selection.

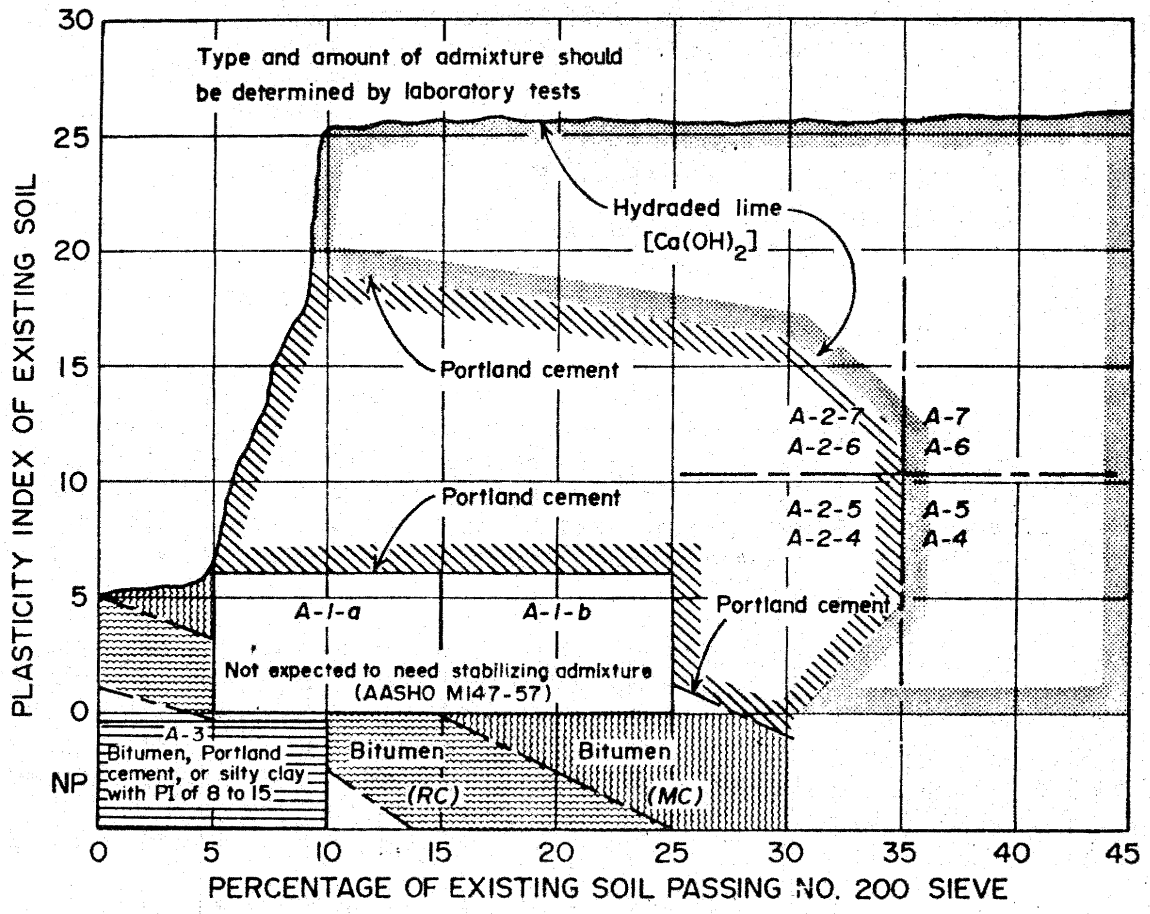


FIGURE NO. 7-1 - Suggested Stabilizers Suitable For Soils Use

Another selection criterion that can be used in the selection process is shown in Figure No. 7-2. This method, based on the PI and percent passing the No. 200 sieve, is found in FHWA-IP-80-2, "Soil Stabilization In Pavement Structures, A Users Manual." The student is directed to this manual for a complete discussion of stabilization selection criteria.

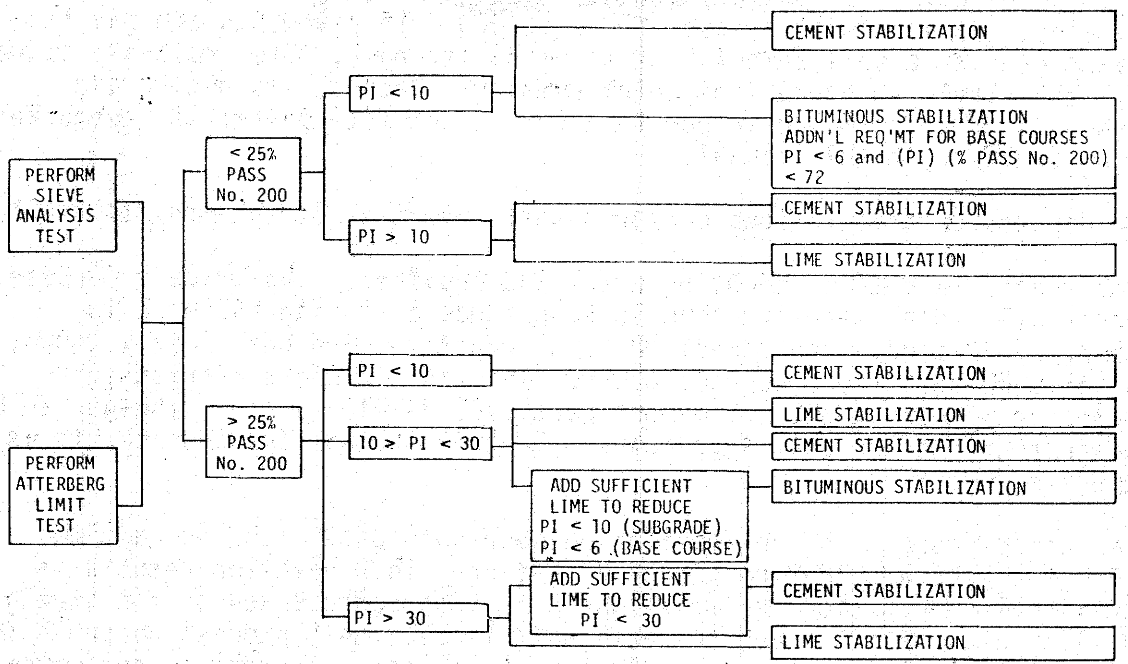


FIGURE NO. 7-2 - Stabilizer Selection

The actual percent of cement or lime is determined by various tests based on compression tests at optimum moisture and density and in the case of cement freeze-thaw AASHTO T-136 or wetting and drying tests AASHTO T-135.

7.2.1 Cement Stabilization

The terminology associated with cement stabilization can be confusing. A number of products are produced depending on the nature of the soil, the proportions of cement and water, and the placement method. Generally accepted definitions of three main types of cement-soil mixtures are as follows:

Soil Cement - mixtures of aggregate-soil and cement designed to meet stated AASHTO criteria such as moisture density relationship and durability. This is sometimes referred to as cement treated base when used as a base.

Cement Modified Base - soil with low treatment of cement to reduce plasticity, control swelling, and somewhat strengthen otherwise marginal material.

Plastic Soil Cement - soil cement with enough water to allow pouring as mortar or concrete.

Econocrete or lean concrete base has gained favor by some States on high type roadways. Econocrete is more expensive than other granular materials, treated or plain. However, it can make use of marginal or reclaimed aggregate and is designed and produced like regular concrete. It is constructed using a slip form paver, requires no compaction, and needs no trimming. It is almost always used with concrete pavements and has the advantage over a soil cement base of being stronger, more resistant to wear, and less likely to erode from joint pumping. Because its design and construction are similar to portland cement concrete pavements, econocrete is not covered in this manual.

The discussion that follows concerns soil cement or cement-modified soil.

Cement acts as both a cementing agent and modifier. The primary purpose of nearly all cement stabilization is to produce a lasting increase in strength. Although the actual chemical reactions are not clearly known, it is believed that two processes occur. The first process consists of hydration of cement whereby cement particles develop strong linkages to bind adjacent grains and soil aggregates. In this process the cement acts as a cementing agent.

The second process is the reaction between soil particles and calcium hydroxide liberated during cement hydration. This reaction results in additional cementitious material which is felt contributes to the strength of silty clay material constituents. It is felt this process accounts for reductions in plasticity and expansion properties. In this second process the cement acts as a modifier.

As a cementing agent, for equal strength, greater amounts of cement are required in fine grained soil than in granular soils since there is greater particle surface area to be covered to provide cementation at contact points. It is generally conceded that best results with cement stabilization are obtained with well-graded soils with less than 50 percent fines and a PI of less than 20 percent. Table No. 7-1 shows the approximate quantity of cement required to harden different soil classifications. Generally, cement stabilization should not be used on greater than 3 percent organic soils or A-6 and A-7 soils.

TABLE NO. 7-1 Required Quantity of Cement For Adequate Hardening of General Soils

Soil Class	Approximate Cement Required (by weight) %
A-1	3-8
A-2	5-9
A-3	5-10
A-4	7-12
A-5	8-14
A-6	9-15
A-7	9-15
Organic Soils	Not suited

Construction of cement stabilization must be keyed to the initial set of the cement. If mixing requires an excessive amount of time, less durable mixtures will result. As a result, the complete construction process must be completed within about 1 to 4 hours after cement is mixed. Delays in compaction after mixing may considerably reduce compaction density and seriously impair durability and strength of the mix. This effect is directly dependent on the setting time of the cement used. Hence, if it is determined after construction that the specified density has not been achieved, additional rolling should not be ordered. Generally, a delay of 2 hours or more in compaction after mixing can be damaging and no rolling after 3 hours from mixing should be allowed.

7.2.2 Lime Stabilization

Lime is a limestone derivative, not limestone itself.
Lime is an all-inclusive term which includes:

- o agricultural lime - calcium carbonate
- o quick lime - calcium oxide
- o hydrated lime - calcium hydroxide

Agricultural lime cannot provide the degree of alkalinity needed for stabilization to work. Quick lime which is a product of heating and pulverizing agricultural lime is a very effective stabilizer but is dangerous to use and can cause severe skin burns.

Hydrated lime is usually formed by adding water to quick lime and is widely used for lime stabilization. Lime by itself is not a good cementing agent compared to cement. It is, however, a good modifier. Lime depends on pozzolanic materials in the soil which consists of clay and other minerals. For soils with low plasticity (low clay and PI) such as sands and silty granular soils, which have low amounts of pozzolans, lime cementing stabilization is ineffective in relation to the use of cement. On the other hand, with highly plastic wet clay soils, lime modification stabilization is highly effective since the use of cement stabilization in such soils is very difficult due to mixing problems.

Adding lime can actually increase the PI of the material thus making it less stable. Generally this occurs if the PI is less than 10. In soils where the amount of natural pozzolan available for lime cementing stabilization is very low (low clay and PI) such as silty or sandy soils, artificial pozzolan can be added along with the lime to cause the lime-pozzolan reaction.

Fly ash is such an artificial pozzolan additive commonly used. Fly ash is a by-product from the burning of powdered coal. Because it is fine, it must be conditioned by the addition of water to prevent dusting when stockpiled in the open. However, the use of lime fly ash as a cementing agent for low plasticity materials must be economically weighed against cement stabilization.

Generally, the use of lime is restricted to southern States with moderate climates, since lime stabilized soils are susceptible to break up under freezing and thawing. Also, southern soils are more plastic than those encountered further north and lime will reduce the plasticity faster than cement and, therefore, is easier to mix. In addition, the cement content required from more plastic soils is high and mixing may be difficult. Like cement, lime should not be used in soils with more than 3 percent organic matter.

The lime content to provide maximum strength after prolonged curing may well exceed 8 to 10 percent. If conditions are such that prolonged curing time is not available, the lime content may be significantly higher.

Upon compaction of the lime stabilized material, an increase in optimum moisture content and a decrease in dry density relative to the unstabilized soil are usually noted. Upon exposure to water, the compacted mix shows very little free swell. This resistance to interaction with water causes contractors to favor lime stabilization since after a rain they can return to work on a stabilized material with minimum delay.

Cement-treated material hardens relatively rapidly, only once, and without a great dependence on temperature as long as near freezing temperatures are not approached. Also, for granular soils the greater the cement content, the greater the strength. None of these generalities hold for lime stabilization. Cementation in lime soil occurs only gradually during warm weather so that the better part of a construction season may be required to achieve a substantial part of the ultimate strength. A benefit of slow curing is flexibility in time for working the soil. It appears that not only can lime stabilized soil be reworked at almost any reasonable time after placement without ill effect, but disturbed soil in old projects will restabilize.

7.2.3 Bituminous Stabilization

Bituminous stabilized base courses range from high strength hot-mix bituminous concrete types that use penetration grade asphalt cements to simple spray application work using liquid asphalt.

Asphalt as a cementing agent does not work well in very poor materials with a large quantity of fines. Use of asphalt in very poor material can increase its plasticity and worsen its stability. Figure No. 7-2 places restrictions on its use and requires a PI less than 6 or the addition of lime to reduce PI. Asphalt stabilization requires a good, well graded granular material to even attempt its use as a cementing agent. Most bituminous base course construction is of the hot-mix type using an aggregate gradation that is relatively open. When sand materials are prevalent, the entire gradation may consist of sand mixed with asphalt. Since asphalt used as cementing agent basically is designed and constructed like hot-mix asphaltic concrete it is beyond the scope of this manual and is covered in other training.

7.3 CONSTRUCTION

The stabilization process involves the following construction steps:

- o material preparation
- o stabilizer application
- o mixing
- o compaction
- o curing

During these construction steps control and inspection should provide for:

- o degree of pulverization
- o checks on proportioning
- o uniformity of final mixing
- o density compaction
- o proper curing

In the following sections the construction steps will be described along with methods for making control and inspection checks.

7.3.1 Base Material Preparation

Having the material in proper condition for stabilization is important. Initial preparatory work will in most instances determine whether a uniform mixture of the material and additive can readily be obtained.

If the base course material is borrow which is often the case, it is most likely a pit or crusher run material which is otherwise granular and does not require pulverization prior to the addition of the stabilizer. However, if clay soil or lumps are observed partial pulverization may be necessary.

If in place material is to be stabilized as a base course it must first be brought to the proper grade, and then scarified and pulverized. During pulverization, deleterious materials like roots and large aggregate should be removed.

Initial scarification is generally done with a grader and/or disc harrow or other suitable type equipment. Pulverization is best accomplished with a rotary type mixer and several passes are normally required. Pulverization should be completed through the entire depth to be stabilized. Usually 6 to 8 inches is considered as the maximum effective depth that will assure complete pulverization and uniform mixing. When the material is unusually dry, water is added to aid pulverization; if extremely wet, the rotary mixer can be used to aerate and dry the soil.

If highly plastic clay materials are encountered, (PI greater than 40) it is advantageous to apply lime in two applications to facilitate pulverization. Although such material is seldom involved with high volume road base material, it may occur on low volume roads.

The initial application of lime is made to break down or modify the clay material thereby making pulverization easier. Typically, after scarification, 2 to 3 percent of the lime is added, partially mixed, and the layer lightly rolled to seal the surface. After 24 to 48 hours final pulverization is attempted, the final lime application made, and the mixing of the lime completed.

For in-place material, pulverization should continue until all the material passes thru a 1-inch sieve and 60 to 80 percent passes the No. 4 sieve, exclusive of gravel or stone retained on those sieves. The degree of pulverization is calculated as follows:

$$\% \text{ Pulverization} = \frac{\text{Dry Weight of Soil-Cement Mixture Passing the No. 4 Sieve}}{\text{Dry Weight of Soil-Cement Sample (Exclusive of Gravel) Retained on No. 4 Sieve}}$$

For practical purposes in the field, wet weights of material are often used instead of the correct dry weights.

7.3.2 Placing and Mixing

The choice of method for placing and mixing will depend primarily on whether the material to be stabilized is in-place or borrow along with other local job conditions such as equipment availability.

It is important to remember that the primary objective of the stabilizer-spreading operation is to achieve uniform distribution in the proper proportion. Field experience has indicated that mixing by itself will not ensure proper stabilizer distribution. Hence, a very important part of quality control is stabilized spreading.

For mix in-place operations, mixture uniformity is usually less than that obtained for central plant-mixing operations. However, satisfactory results can be obtained with road-mixing equipment. Stabilizer application for in-place mixing can be done by:

- bag
- bulk
- slurry

Bag placement is generally the simplest method but requires greatest labor costs and slower production. It is seldom used today because project sizes make it uneconomical and mechanical equipment to do the job is generally available. However, on small jobs it can be utilized and involves placing bags of stabilizer after initial pulverization according to a predetermined grid pattern to meet the required stabilized content.

On large projects, where dusting is not a problem, placement in most cases can be done by bulk directly from the transport truck. This method is quite rapid but it is difficult to accurately ensure the placement rate which must be monitored in the field. Bulk spreading may be done directly from a suitably equipped self-unloading transport or from the bulk haul units through a mechanical spreader. Field control to ensure proper spread rate must be employed. This can be done by several methods. One is to use the known weight of material in the transport over a calculated area based on the design spread-rate, spreading in increments with multiple passes.

Another method requires a large pan or piece of canvas (1 to 2 s.y.) and a scale. This involves calibrating the mechanical spreader spread-rate at constant speed by catching the material on the grade and weighing it to determine spread-rate. After suitable low rate of spread is established, the required material is placed using multiple passes. In either process, it is very important that the spreading equipment maintain a uniform rate of speed.

Slurry distribution methods can be used for lime distribution but are not applicable to cement because of rapid hydration. Hydrated lime and water are mixed in a central mixing tank or tank truck in proportions of about 1 ton of lime to 500 gallons of water. The slurry is spread over the scarified roadbed through tank truck spray bars either by gravity or pressure, or the slurry can be added to the soil during mixing operations.

The major advantage of the slurry method is the prevention of the dusting problem. In addition, this method combines lime spreading and watering operations into one. Also, use of slurry generally promotes more uniform distribution. Disadvantages include need for slurry mixing equipment, and unsatisfactory use with wet soils.

If the base material is borrow, the contractor may prefer, or the specifications may require that the proportioning and mixing of the soil, stabilizer, and water be done with a central mix plant. The proportion is much more accurately done at a central mix plant and the resulting mixture will be more uniform than could be obtained by mixing on the grade. Use of central mix plants requires that it be calibrated to ensure proper material proportions. Calibration is normally the plant's responsibility with State inspectors checking that it is properly and accurately done.

One calibration method requires that the plant or base material output be calibrated by running it through the plant for various periods of time (2, 3, and 5 minutes) for specific gate openings. The discharged material is collected in a truck and weighed. From these weights and the moisture content, dry-weight plant discharge calibration curves are determined. The stabilizer admixture (cement or lime) feeder calibration chart is similarity determined by discharge into a truck or other suitable container. With plant base and stabilizer calibration charts the proper settings can be made to ensure the proper proportions.

A second method is to operate the plant with only soil aggregate material feeding the main conveyer belt. The material on a selected length of conveyer belt is sampled, weighed and its dry weight determined. The plant is then operated with only cement feeding onto the main conveyer belt. If a variable speed screw or vane cement feeder is being used, several trials are made at different RPM settings on the cement feeder.

If a belt cement feeder is being used, trials are made at different cement feeder gate openings. A calibration graph can then be drawn by plotting the RPM setting or gate opening of the cement feeder on one graph axis and the computed percent of cement by dry weight of soil aggregate on the other axis. Thus, for a constant supply of soil aggregate, the setting on the cement feeder for the required quantity of cement can be determined from the graph.

Whatever application method is used, there should be checks as to the amount of stabilizer actually used vs. the design stabilizer rate to verify plant calibration and operations. The main feeder belt is a good place to check for proper proportioning by sampling and weighing base material and stabilizer separately and calculating the percent of stabilizer.

Additive cut off, that is computations of additive received vs. additive remaining in storage to give the actual amount of additive used for comparison against theoretical amount required for the work completed at the time of the cutoff, is another check method to determine the accuracy of proportioning. Cutoffs should be made at some convenient point, such as when a storage bin has been empty, where reasonable accurate figures can be attained.

If plant mixing is used the stabilized mixture should be delivered and spread as uniformly as possible with minimum manipulation. Dumping and spreading with a motor grader can be used but is not recommended due to segregation. Use of a spreader box on other equipment with automatic grade control is recommended because of more uniform depth and minimum segregation.

For in-place mixing, once the stabilization material is placed, mixing can be done by:

- o flat transverse-shaft mixers, single-shaft or multiple-shaft

- o windrow-type pugmill

Multiple or single-shaft mixers require only that the pulverized base material be spread to the approximate finish base profile with the stabilizer uniformly spread on top. Mixing may require several passes of the single-shaft mixer where as the multiple-shaft mixer may do it all in one pass. Shaft mixers leave the mixed material in a layer ready for compaction, whereas the windrow-type pugmill requires further material uniform spreading.

The windrow-type mixer requires the material be placed in windrows and the stabilizer placed on the windrow in front of the machine. After the machine mixes the windrow materials they are spread with a motor grader to approximate grade and compacted.

Whatever grade application and mixing procedure is used, field uniformity should be judged by digging full depth trenches or holes noting the material color. Unmixed streaks or layers are sufficient reason for remixing. At the same time the thickness of the stabilized layer should be measured to see if it meets specification.

Construction Joints

After each day's construction a transverse vertical construction joint must be formed cutting back into the completed material. This is usually done the last thing at night or the first thing the following morning, using the toe of the motor-grader blade or mixer. The joint must be vertical and perpendicular to the centerline.

After the next day's placing has been completed, the joint must be cleaned of all dry and unmixed material and retrimmed if necessary. Mixed moist material is then bladed into the area and compacted thoroughly. The joint is left slightly high until final rolling when it is trimmed to grade with the motor grader and rerolled.

Joint construction requires special attention to make sure the joints are vertical and the material in the joint area is adequately mixed and thoroughly compacted.

If a mix-in-place method is used, a longitudinal joint adjacent to the partially hardened material can be constructed with transverse shaft mixers by merely cutting back a few inches with the mixer into the previously constructed area. The amount of overlap is determined by digging back into the completed work until solid material and proper crown and grade are reached.

7.3.3 Moisture And Density

Moisture control of both lime and cement stabilization must be closely controlled. Figure No. 7-3 shows the importance and effect of density on the compressive strength of a cement stabilized material. Optimum moisture and density are determined by tests almost identical to the standard proctor method. Maximum density of lime or cement stabilized material will differ from the maximum density of the basic material. This change can be higher or lower depending on basic material characteristics but usually is lower.

It is very important that the target density and moisture be determined for the mix and not to assume it is the same as the basic material. For cement stabilization AASHTO T134 is used which is very similar to the standard AASHTO T99 proctor test. For lime stabilization, AASHTO T99 density moisture test is commonly specified.

The water that is used in the mix may be added directly in front of the in-place mixing machine or it may be added to the mix in the mixing machine. The more sophisticated multiple-shaft mixing machines normally inject the proper amount of water into the mixer immediately after the materials are picked up. The single-shaft and windrow-type mixers may have water injection systems or they may require that the mixing water be placed on the aggregate just prior to mixing.

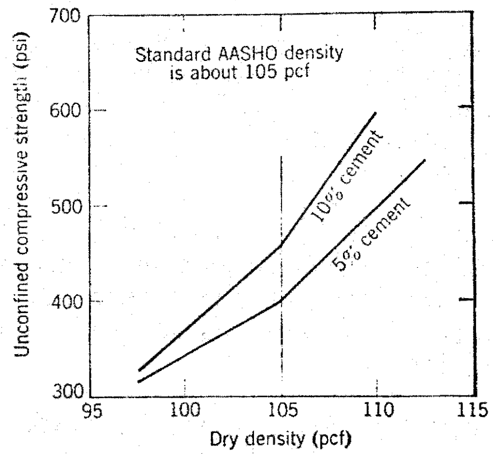


FIGURE NO. 7-3 - Effect of Compacted Density On Strength Of Cement Stabilized Silty Clay

Since there is considerable material mixing aeration, it is generally recommended that water content be slightly above optimum to provide for evaporation loss. Generally, this increase is 2 to 3 percent for both lime or cement stabilization. When applying a double application of lime as indicated in Section 7.3.1, the moisture content should be raised at least 5 percent above optimum and the mixture sealed by light rolling and allowed to mellow. The mixture is subsequently scarified, put through the final mixing process and compacted. During final mixing moisture content should be adjusted to optimum for compaction purposes.

When water is field-applied by tank truck it is important that it be uniformly applied for accurate results. Overlapping water-spreading passes and missed areas will result in nonuniform moisture content and low or nonuniform densities.

7.3.4 Compaction

Compaction should commence as soon as possible after uniform mixing of water and stabilizer. Since the reaction associated with lime is long term as compared to cement, additional time is available for mixing and compaction. However, it should normally be completed the same day to avoid loss of moisture.

With fine cohesive materials, the use of lime often makes it behave as if basically granular in nature, with little or no cohesion at the time of compaction. Therefore, pneumatic, steel wheel or vibratory rollers are usually most effective in producing initial densification.

Stabilized base rolling equipment and procedures are similar to unstabilized base material so long as the basic material is granular. However, because the basic material to be stabilized is sometimes marginal and, therefore, fine grained and plastic, initial compaction rolling is often done with a sheepsfoot roller. Sheepsfoot rollers used in such cohesive material compact through the kneading action of the feet and compact from the bottom up. This is followed by a pneumatic roller with a steel wheel roller used for finishing.

In order to get uniform consistent compaction the stabilized material must be initially spread with uniform depth and even surface. When the sheepsfoot roller has compacted and 2/3 of the loose depth of material and the roller feet are about 2 or 3 inches from the surface, a motor grader should move in to reshape and maintain an even uniform surface. When the roller feet are about 1 inch from the surface, the grader should give final shaping and the roller allowed to walk completely out. The surface should then be roughed up with a spiked tooth harrow prior to pneumatic-tire rolling.

The prime control on compaction is the end density result. Density measurement is probably the most important part of compaction since strength and hardening depend on it. Ordinarily, 95 to 100 percent of the standard AASHTO density is specified. In place density tests are performed by the same methods as for unstabilized base material as outlined in Section 6.3. Because of the limited time for stabilization compaction, such tests must be made in a very timely manner as soon as the contractor feels the required density has been achieved.

In both compacting and checking density, particular attention must be given to the materials near the edges of the pavement (outer 3 feet). Since there is often little side support for the materials near the edge of the pavement, there is a tendency during compaction for the materials to be shoved sideways rather than become compacted. This causes a reduced thickness of materials and low density in the most critical portion of the pavement, the pavement edge.

7.3.5 Weather Constraints

Favorable temperature and moisture conditions are essential for proper strength development and performance of lime and cement stabilization.

The most common obstacle to proper compaction is the presence of excess water in the material. For this reason, rainy weather usually halts construction and can have serious effects on attained strength. Also, if the subgrade material is excessively wet the stabilized material can be over-wetted from below. The remedy to such overwetting can be bad news to the contractor, i.e., tear out everything and start over. Stabilized material torn up and aerated will require addition of more stabilizer if (1) the chemical has a quick set, such as portland cement, (2) the chemical is destroyed, such as lime coming in contact with carbon dioxide in rain water, (3) the chemical is leached out.

Both lime and cement stabilization must be suspended during cold or near freezing temperatures. Lime stabilization material is subject to break up under freezing and thawing. Since cement hydration practically ceases when temperatures approach freezing, cement should not be applied when air temperature is 40° F or lower. Also, cement should not be applied when the basic material or subgrade is frozen. Lime, however, has a great advantage over cement in this area because it can be recovered after freezing whereas cement generally cannot.

7.3.6 Curing

Proper curing of lime, cement and lime/cement fly ash are extremely important because the strength gain is dependent upon time, temperature, and the presence of moisture. The effect of curing time is shown in Figure No. 7-4 and is most critical for the first 7 days. Two types of curing of the stabilized layer are used: sprinkling and membrane. Sprinkling with water to keep the surface damp, together with light rolling with a steel wheel roller to keep the surface knitted together, has proven successful. However, the preferred method is membrane curing. The stabilized soil is either sealed with one shot of cutback asphalt (0.1 to .25 gal/sy) after final rolling or primed with increments of asphalt emulsion applied several times during the curing period.

Cracking may develop due to shrinkage and the richer the mix the more the cracking will develop. Proper and timely curing will minimize such cracks. Such cracks are detrimental in that they can permit infiltration of surface water or can reflect up through a flexible pavement surface.

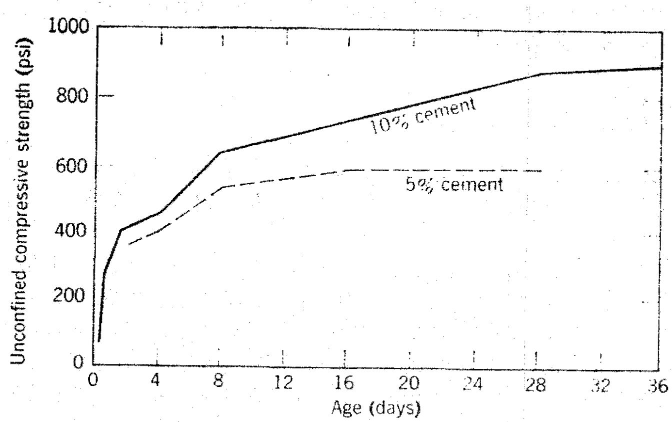


FIGURE NO. 7-4 - Influence Of Curing Time On Compressive Strength Of Cement Stabilized Silty Clay

Name: ______________________
Office: ______________________
Course Date: ______________________

CONSTRUCTION INSPECTION TECHNIQUES FOR BASE COURSE CONSTRUCTION

1. List the principal problems in your State relating to the performance of base course construction.

   a.

   b.

   c.

   d.

   e.

   f.

   g.

   h.

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CONSTRUCTION INSPECTION TECHNIQUES FOR
BASE COURSE CONSTRUCTION
(Continued)

2. Identify the most likely root cause(s) corresponding to each of these performance problems.

   a.

   b.

   c.

   d.

   e.

   f.

   g.

   h.

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CONSTRUCTION INSPECTION TECHNIQUES FOR
BASE COURSE CONSTRUCTION
(Continued)

3. Identify what you are currently doing to affect or cause improvement for each of the above listed performance problems.

a.

b.

c.

d.

e.

f.

g.

h.

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CONSTRUCTION INSPECTION TECHNIQUES FOR
BASE COURSE CONSTRUCTION
(Continued)

4. Identify the most important thing you could do to affect change or improvement with respect to the previously listed performance problems.

   a.

   b.

   c.

   d.

   e.

   f.

   g.

   h.

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