PDHonline Course C363 (5 PDH)

Pavement Subdrains & Subgrade Improvements

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CHAPTER 7.0 DESIGN DETAILS AND CONSTRUCTION CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

7.1 INTRODUCTION

This chapter includes:

- Design details for key geotechnical components in the pavement system, including drainage and base layer requirements.
- Compaction of subgrades and unbound pavement layers.
- A general overview of the types of potential subgrade problems.
- Identification and treatment of select, widely occurring special geotechnical challenges, including collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils.
- Detailed guidelines on alternate stabilization methods, which are often used to mitigate special problems.

In this chapter, design details for the specific pavement features of base materials and drainage systems are provided. Compaction, one of the key geotechnical issues in pavement design and construction is also covered.

Special challenges generally relate to poor subgrade conditions that occur due to the type of soil and environmental conditions. In this chapter, the various types of problematic soil conditions are reviewed along with widely occurring specific subgrade problems. Although these problematic conditions can be detected with a detailed subsurface exploration, problematic conditions can potentially go unnoticed if they are located between borings.

7.2 SUBSURFACE WATER AND DRAINAGE REQUIREMENTS

The damaging effects of excess moisture on the pavement have long been recognized. Moisture from a variety of sources can enter a pavement structure. This moisture, in combination with heavy traffic loads and freezing temperatures, can have a profound negative effect on both material properties and the overall performance of a pavement system.

As was shown in Figure 3-3, Chapter 3, moisture in the subgrade and pavement structure can come from many different sources. Water may seep upward from a high groundwater table, or it may flow laterally from the pavement edges and shoulder ditches. Knowledge of groundwater and its movement are critical to the performance of the pavement as well as stability of adjacent sideslopes, especially in cut situations. Groundwater can be especially troublesome for pavements in low-lying areas. Thus, groundwater control, usually through interception and removal before it can enter the pavement section, is an essential part of pavement design.

In some cases, pavements are constructed beneath the permanent or a seasonally high watertable. Obviously, drainage systems must perform or very rapid pavement failure will occur. In such cases, redundancy in the drainage design is used (*e.g.,* installation of underdrains and edgedrains) and, often, some monitoring is used to ensure continual function of the drain system.

Capillary action and moisture-vapor movement are also responsible for water accumulating beneath a pavement structure (Hindermann, 1968). Capillary effects are the result of surface tension and the attraction between water and soil. Moisture vapor movement is associated with fluctuating temperatures and other climatic conditions.

As was previously indicated in Chapter 3, the most significant source of excess water in pavements is typically infiltration through the surface. Joints, cracks, shoulder edges, and various other defects in the surface provide easy access paths for water. A study by the Minnesota Department of Transportation indicates that 40% of rainfall enters the pavement structure (Hagen and Cochran 1995). Demonstration Project 87, *Drainable Pavement Systems*, indicates that surface infiltration is the single largest source of moisture-related problems in PCC pavements (FHWA 1994). Although AC pavements do not contain joints, surface cracks, longitudinal cold joints that crack, and pavement edges provide ample pathways for water to infiltrate the pavement structure.

The problem only worsens with time. As pavements continue to age and deteriorate, cracks become wider and more abundant. Meanwhile, joints and edges become more deteriorated and develop into channels through which moisture is free to flow. The result is more moisture being allowed to enter the pavement structure with increasing pavement age, which leads to accelerated development of moisture-related distresses and pavement deterioration.

7.2.1 Moisture Damage Acceleration

Excessive moisture within a pavement structure can adversely affect pavement performance. A pavement can be stable at a given moisture content, but may become unstable if the materials become saturated. High water pressures can develop in saturated soils when subjected to dynamic loading. Subsurface water can freeze, expand, and exert forces of considerable magnitude on a given pavement. Water in motion can transport soil particles and cause a number of different problems, including clogging of drains, eroding of embankments, and pumping of fines. These circumstances must be recognized and accounted for in the design of a pavement.

The detrimental effects of water on the structural support of the pavement system are outlined by AASHTO (1993), as follows:

- Water in the asphalt surface can lead to moisture damage, modulus reduction, and loss of tensile strength. Saturation can reduce the dry modulus of the asphalt by as much as 30% or more.
- Added moisture in unbound aggregate base and subbase is anticipated to result in a loss of stiffness on the order of 50% or more.
- Modulus reduction of up to 30% can be expected for asphalt-treated base and increase erosion susceptibility of cement or lime treated bases.
- Saturated fine-grain roadbed soil could experience modulus reductions of more than 50%.

As noted in Chapters 3, 4, 5 and 6, modulus is the key pavement design property!

The influence of saturation on the life of the pavement is illustrated in Figure 7-1. The severity factor (shown in the figure) is the anticipated relative damage during wet versus dry periods anticipated for the type of road. As an example, Figure 7-1 shows that if the pavement system is saturated only 10% of its life (*e.g.,* about one month per year), a pavement section with a moderate stability factor will be serviceable only about 50% of its fully drained performance period. Specific distresses caused by excessive moisture within flexible and rigid pavements are summarized in Table 7-1 and 7-2, respectively.

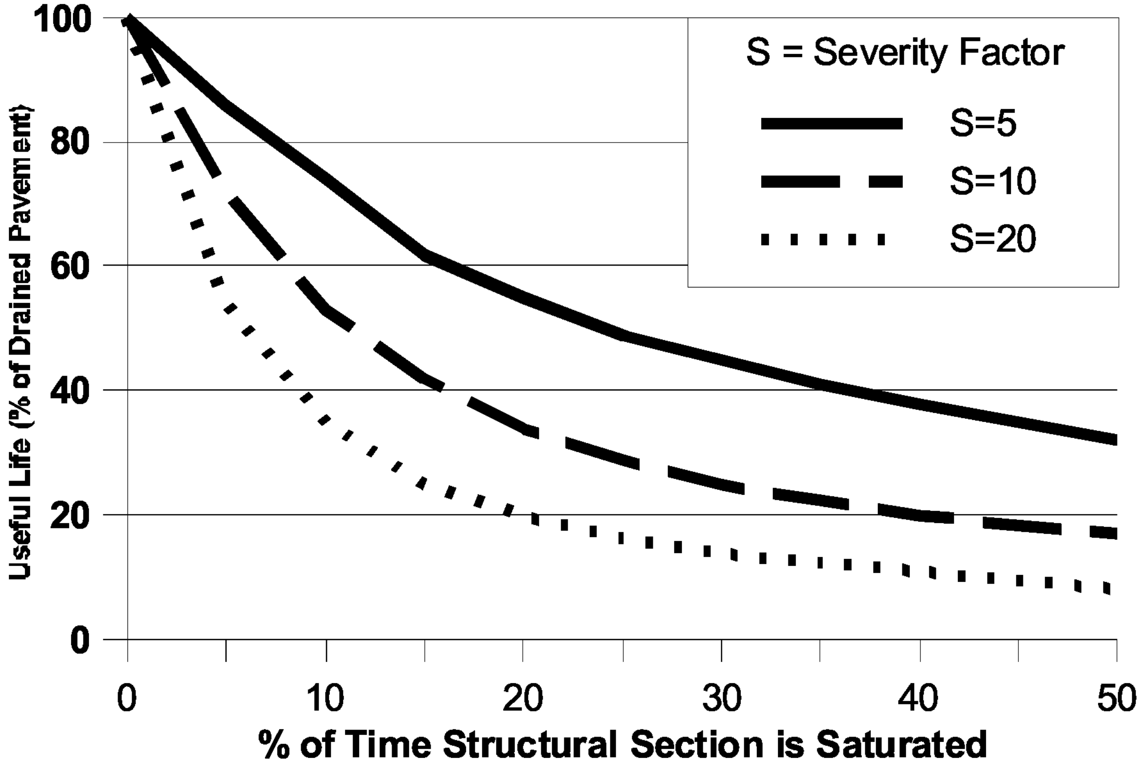

Figure 7-1. The influence of saturation on the design life of a pavement system (after Cedergren, 1987).

Table 7-1. Moisture-related distresses in flexible (AC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						AC	Base	Subgrade
Surface Deformation	Bump or Distortion	Excess Moisture	Frost Heave	Volume Increase	No	No	No	Yes
	Corrugation or Rippling	Slight	Moisture and Temperature	Unstable Mix	Yes	Yes	Yes	No
	Stripping	Yes	Moisture	Loss of Bond	No	Yes	No	No
	Rutting	Excess in Granular Layers or Subgrade	Moisture	Plastic Deformation, Stripping	Yes	Yes	Yes	Yes
	Depression	Excess Moisture	Suction & Materials	Settlement, Fill Material	No	No	No	Yes
	Potholes	Excess Moisture	Moisture, Temperature	< Strength > Moisture	Yes	Yes	Yes	Yes
Cracking	Longitudinal	No; Accelerates	No	Construction	No	Faulty Construction	No	No
	Alligator (fatigue)	Yes; Accelerates	Spring-Thaw Strength loss	Thickness	Yes	Yes, Mix	Yes	No
	Transverse	No; Accelerates	Low Temp. Freeze-Thaw Cycles	Thermal Properties	No	Yes, Temp. Susceptible	No	No
	Slippage	Yes	No	Loss of Bond	Yes	Yes, Bond	No	No

Table 7-2. Moisture-related distresses in rigid (PCC) pavements (NHI 13126; adapted after Carpenter et al. 1979).

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load Associated Distress	Structural Defect Begins in		
						PCC	Base	Subgrade
Surface Defects	Spalling	Possible	Freeze/Thaw Cycles	Mortar	No	Yes	No	No
	Scaling	Yes	Freeze/Thaw Cycles	Chemical Influence	No	Yes; Finishing	No	No
	D-Cracking	Yes	Freeze/Thaw Cycles	Aggregate Expansion	No	Yes	No	No
	Crazing	No	No	Rich Mortar	No	Yes; Weak Surface	No	No
Surface Deformation	Blow-up	No	Temperature	Thermal Properties	No	Yes	No	No
	Pumping and Erosion	Yes	Moisture	Inadequate Strength	Yes	No	Yes	Yes
	Faulting	Yes	Moisture-Suction	Erosion-Settlement	Yes	No	Yes	Yes
	Curling/Warping	Yes	Moisture & Temperature	Moisture and Temperature Differentials	No	Yes	No	No
Cracking	Corner	Yes	Moisture	Cracking Follows Erosion	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Moisture	Follows Erosion	Yes	No	Yes	Yes
	Punchout (CRCP)	Yes	Moisture	Deformation Follows Cracking	Yes	No	Yes	Yes

7.2.2 Approaches to Address Moisture in Pavements

As was indicated in Chapter 3, to avoid moisture-related problems, a major objective in pavement design should be to keep the base, subbase, subgrade, and other susceptible paving materials from becoming saturated or even exposed to constant high moisture levels. The three approaches described in detail in Chapter 3 for controlling or reducing the problems caused by moisture are

- prevent moisture from entering the pavement system.
- use materials and design features that are insensitive to the effects of moisture.
- quickly remove moisture that enters the pavement system

No single approach can completely negate the effects of moisture on the pavement system under heavy traffic loading over many years. For example, it is practically impossible to completely seal the pavement, especially from moisture that may enter from the sides or beneath the pavement section. While materials can be incorporated into the design that are insensitive to moisture, this approach is often costly and in many cases not feasible (*e.g.,* may require replacing the subgrade). Drainage systems also add cost to the road. Maintenance is required for both drainage systems and sealing systems, for them to effectively perform over the life of the system. Thus, it is often necessary to employ all approaches in combination to obtain the most effective design. The first two approaches involve the surficial pavement materials, which are well covered in the NHI courses on pavement design (*e.g.,* NHI 131060A "Concrete Pavement Design Details and Construction Practices" and the participant's manual) and will not be covered herein. The geotechnical aspects of these approaches include drainage systems for removal of moisture, the requirements of which will be reviewed in the following subsections. Durable base material requirements will be reviewed in the subsequent section, and followed by subgrade stabilization methods to mitigate moisture issues in the subgrade. A method of sealing to reduce moisture intrusion into the subgrade will also be reviewed in the subgrade stabilization section.

7.2.3 Drainage in Pavement Design

Removal of free water in pavements can be accomplished by draining the free water vertically into the subgrade, or laterally though a drainage layer into a system of collector pipes. Generally, the actual process will be a combination of the two (ASSHTO, 1993). Typically in wet climates, if the subgrade permeability is less than 3 m/day (10 ft/day), some form of subsurface drainage or other design features to combat potential moisture problems should be considered. Table 7-3 provides additional climatic conditions and traffic considerations to assist in the assessment of the need for subsurface drainage.

The quality of drainage is defined in both AASHTO 1993 and NCHRP 1-37A based on the principle of time-to-drain. Time-to drain is the time required following any significant rainfall event for a pavement system to drain from a saturated state to a specific saturation or drainage level (*e.g.,* 50% drainage level in AASHTO 1993). The concept can also be applied (at least qualitatively) to other significant moisture events that would saturate the pavement (*i.e.,* flood, snow melt, or capillary rise). The definitions of poor to excellent drainage provided by AASHTO (1993) are given in Table 7-4.

Table 7-3. Assessment of need for subsurface drainage in new or reconstructed pavements (NCHRP 1-37 A, adapted after NHI 13126).

Climatic Condition	**Greater than 12 million 20-yr design lane heavy trucks**			**Between 2.5 and 12 million 20-yr design lane heavy trucks**			**Less than 2.5 million 20-yr design lane heavy trucks**		
	$k_{subgrade}$ (m/day)								
	< 3	3 to 30	> 30	< 3	3 to 30	> 30	< 3	3 to 30	> 30
Wet-Freeze	R	R	F	R	R	F	F	NR	NR
Wet-No Freeze	R	R	F	R	F	F	F	NR	NR
Dry-Freeze	F	F	NR	F	F	NR	NR	NR	NR
Dry-No Freeze	F	NR	NR	NR	NR	NR	NR	NR	NR

LEGEND:

$k_{subgrade}$ = Subgrade permeability.

R = Some form of subdrainage or other design features are recommended to combat potential moisture problems.

F = Providing subdrainage is feasible. The following additional factors need to be considered in the decision making:

(1) Past pavement performance and experience in similar conditions, if any.
(2) Cost differential and anticipated increase in service life through the use of various drainage alternatives.
(3) Anticipated durability and/or erodibility of paving materials.

NR = Subsurface drainage is not required in these situations.

Wet Climate = Annual precipitation > 508 mm (20 in.)

Dry Climate = Annual precipitation < 508 mm (20 in.)

Freeze = Annual freezing index > 83 ºC-days (150 ºF-days)

No-Freeze = Annual freezing index < 83 ºC-days (150 ºF-days)

Table 7-4. AASHTO definitions for pavement drainage recommended for use in both flexible and rigid pavement design (AASHTO, 1993).

Quality of Drainage	Water Removed* Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Does not Drain

* Based on 50% time-to-drain.

As reviewed in Chapters 3, 5, and 6, drainage effects on pavement performance are incorporated into both the AASHTO 1993 and in the NCHRP 1-37A design methods. In AASHTO 1993, the effect of drainage is considered by modifying the structural layer coefficient (for flexible pavements) and the load transfer coefficient (for rigid pavements) as a function of the quality of drainage and the percent of time the pavement structure is near saturation. The influence of the drainage coefficient (C_d) for rigid pavement design and a drainage modifier (m) for flexible pavement design were demonstrated in the sensitivity studies shown in Chapter 6.

In the NCHRP 1-37A pavement design guide, the impact of moisture on the stiffness properties of unbound granular and subgrade materials is considered directly through the modeling of the interactions between climatic factors (rainfall and temperatures), groundwater fluctuations, and material characteristics of paving layers. Drainage coefficients are not used. However, the benefits of incorporating drainage layers are apparent in terms of distress predictions, which consider seasonal changes in unbound layers and subgrade properties due to moisture and coupled moisture-temperature effects.

Using either the AASHTO 1993 or NCHRP 1-37A method, the influence on design can be significant. For example, in high rainfall areas, the base section of a flexible pavement system (with a relatively thick base layer) can be reduced in thickness by as much as a factor of 2, or the design life extended by an equivalent amount, if excellent drainage is provided versus poor drainage. Likewise, an improvement in drainage leads to a reduction in Portland cement concrete (PCC) slab thickness.

Achieving poor drainage is relatively simple. If the subgrade is not free draining (e.g., not a clean sand or gravel), then the pavement section will require drainage features to drain. Even with edge drainage (*i.e.,* daylighted base or edgedrains), drainage could still be poor. Many designers choose to use dense graded base for its improved construction and presumed structural support over free-draining base. Unfortunately, dense graded base usually does not readily drain and, as a result, structural support will most likely decrease over time.

Due to the low permeability of dense graded base and long drainage path to the edge of the road, drainage in dense graded base is, at best, extremely slow. For example, consider that the permeability of a dense graded base with a very low percentage of fine-grain soil (less than 5% smaller than a 0.075 mm {No. 200 U.S. sieve}) is about 0.3 m/day (1ft/day)(as was reviewed in Chapter 5). Also consider that the length of the drainage path for a two-lane road (lane width of the road draining from the centerline to the edge) is typically 3.7 m (12 ft). An optimistic estimate of the time required to drain a base section that is 300 mm (1 ft) thick and has a slope of 0.02 is 2 days. According to AASHTO definitions of drainage, the pavement section has "good" to "fair" drainage. If the length of the drainage path is two lanes (*i.e.,* 7.3 m {24 ft}), it would take up to a week for the pavement to drain; a condition defined as "fair" drainage (AASHTO, 1993). Base materials often contain more than 5% fines, in which case the permeability and, correspondingly, the drainage can easily be an order of magnitude less than the estimated value for the example (AASHTO, 1993)[1]. In a recent study a Midwestern state found base materials from six different quarries to have 12% to 19% fines and corresponding field permeabilities measured at 2 to 0.01 m/day (7 to 0.03 ft/day) (Blanco et al., 2003). A month or more will then be estimated for pavement drainage; a condition defined as "poor" to "very poor" in AASHTO 1993. In reality, capillary effects and the absence of a driving head of water often cause dense graded base to act like a sponge at low hydraulic gradients. This results in trapped water in the pavement section and "very poor" drainage (*e.g.,* see Dawson and Hill, 1998).

In order to achieve good to excellent drainage, a more permeable, open-graded base and/or subbase will be required, which is tied into a subsurface drainage system. However, this approach only works for new or reconstructed pavements. For existing pavements, retrofitting drainage along the edges of the pavement is the only option, and the existing base material may not drain. However, a significant amount of water can enter the pavement at the crack between the shoulder and the pavement, as well as from lateral movement of water from outside the shoulder. Specific guidelines do not exist currently for retrofit pavements, as only limited data are available. Local experience should be used in selecting pavement candidates for retrofitting. Performance of similar retrofitted sections, if available, can be a valuable tool in the decision making process.

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[1] Based on hydraulic conductivity tests, AASHTO notes a decrease in permeability from 3 m/day (10 ft/day) with 0% fines down to 0.02 m/day (0.07 ft/day), with the addition of only 5% non-plastic fines and (0.0003 m/day (0.001 ft/day) with 10% non-plastic fines. An additional order of magnitude decrease was observed with base containing plastic fines.

7.2.4 Types of Subsurface Drainage

In the past, pavement systems were designed without any subdrainage system. These sections are commonly labeled “bathtub” or “trench” sections because infiltrated water is trapped in the base and subbase layers of the pavement system.

Many types of subsurface drainage have been developed over the years to remove moisture from the pavement system. These subsurface drainage systems can be classified into several groups. One criterion for classifying various subsurface drainage systems is the source of moisture that the system is designed to control. For example, a **groundwater control system** refers to a subsurface drainage system designed to remove and control the flow of groundwater. Similarly, an **infiltration control system** is designed to remove water that seeps into the pavement structural section. A **capillary break system** is designed to intercept and remove rising capillary water and vapor movement.

Probably the most common way to classify a subsurface drainage system is in terms of its location and geometry. Using this classification, subsurface drainage systems are typically divided into five distinct types:

- Longitudinal edgedrains.
- Transverse and horizontal drains.
- Permeable bases.
- Deep drains or underdrains.
- Interceptor drains.

Each type may be designed to control several sources of moisture and may perform several different functions. In addition, the different types of subsurface drainage system may be used in combination to address the specific needs of the pavement being designed. Drains constructed primarily to control groundwater general consist of underdrains and/or interceptor drains. The interceptor drains are usually placed outside the pavement system to intercept the lateral flow of water (*e.g.,* from cut slopes) and remove it before it enters the pavement section. Deep underdrains (greater than 1 m {3 ft} deep) are usually installed to lower the groundwater level in the vicinity of the pavement. The design and placement of these interceptor and underdrains should be addressed as part of the geotechnical investigation of the site.

Edgedrains placed in trenches under the shoulders at shallower depths are used to handle water infiltrating the edge of the pavement from above. Edgedrains are combined with permeable base and, in some cases, transverse and horizontal drains to form a drainable pavement system to control surface infiltration water. Drainable pavement systems generally consist of the following design features (as shown in Figure 7-2):

- a full-width permeable base under the AC- or PCC-surfaced travel lanes,
- a separator layer under the permeable base to prevent contamination from subgrade materials,
- longitudinal edgedrains with closely spaced outlets. An alternative to closely spaced outlets is dual drainage systems with parallel collector drains. An alternative to edgedrains is daylighting directly into a side ditch.

Designs not incorporating these combinations of features cannot be expected to function properly. Drainage systems for new construction and rehabilitation are described in more detail in the following sections.

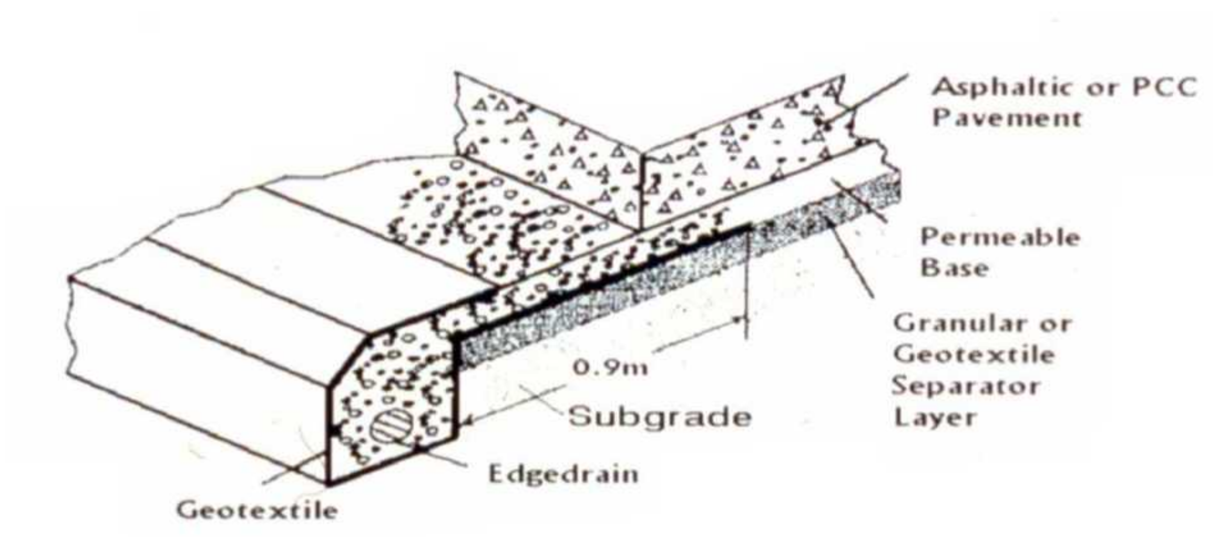


Figure 7-2. Design elements of a drainable pavement system (after FHWA, 1992).

7.2.5 Daylighted Base Sections

Daylighted bases were one of the first attempts to remove surface infiltration water from the pavement system. The original daylighted base consists of a dense-graded aggregate base that extends to the ditchline or side slope. Daylighted dense-graded bases are expected to intercept water that infiltrates through the pavement surface and drain the water through the base to the ditch. However, most dense-graded daylighted bases are slow draining and, therefore, not very effective in removing infiltrated water.

This situation led to the development of a new generation of daylighted bases—daylighted permeable bases (Fehsenfeld 1988), as illustrated in Figure 7-3. Several studies have reported that daylighted permeable bases are as effective in removing infiltrated water and reducing moisture-related distresses as permeable bases with edgedrains (Yu et al. 1998b). However, they require regular maintenance because the exposed edge of daylighted bases easily becomes clogged with fines, soil, vegetation, and other debris. Also, stormwater from ditch lines can easily backflow into the pavement structure. Further study into daylighted permeable bases is needed to verify long-term performance of this design.

7.2.6 Longitudinal Edgedrains

Longitudinal edgedrains consist of a drainage system that runs parallel to the traffic lane. The edgedrains collect water that infiltrates the pavement surface and drains water away from the pavement through outlets. Four basic types of edgedrains systems have been used:

- pipe edgedrains in an aggregate filled trench,
- pipe edgedrains with porous concrete (*i.e.,* cement treated permeable base) filled trench,
- prefabricated geocomposite edgedrains in a sand backfilled trench, and
- aggregate trench drain ("French" drain).

The most commonly used edgedrain is a perforated pipe varying in diameter from 100 – 150 mm (4 – 6 in.). The pipe is generally situated in an aggregate trench to allow water to flow toward the pipe. Another type of edgedrain that is often used in rehabilitation projects is a geocomposite drain in a sand filled trench with pipe outlets. Typical cross sections of edgedrains are illustrated in Figures 7-5 and 7-6.

Figure 7-3. Typical AC pavement with a daylighted base.

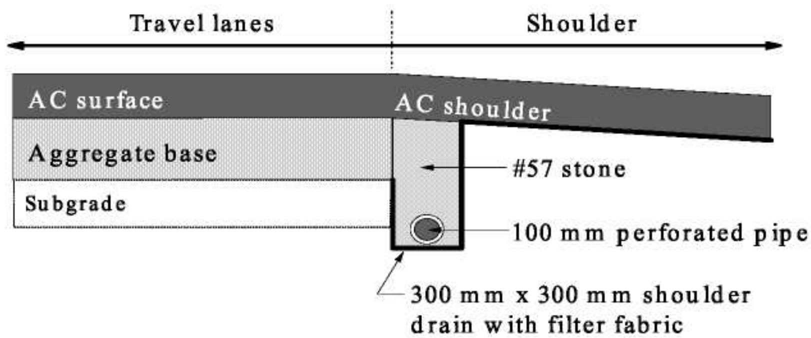


Figure 7-4. Typical AC pavement with pipe edgedrains (ERES, 1999).

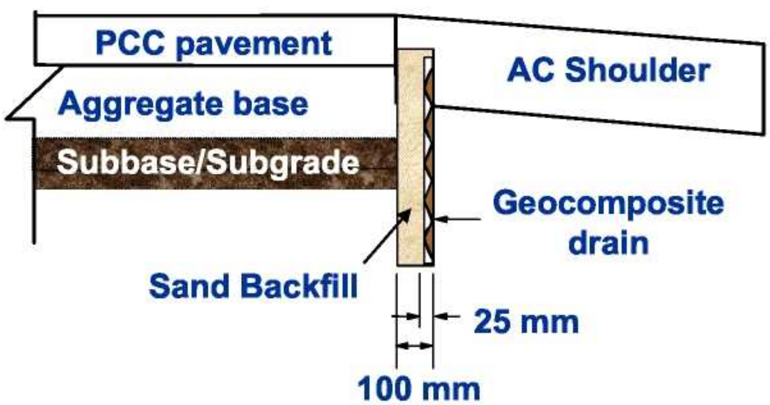


Figure 7-5. Typical PCC pavement with geocomposite edgedrains (ERES, 1999).

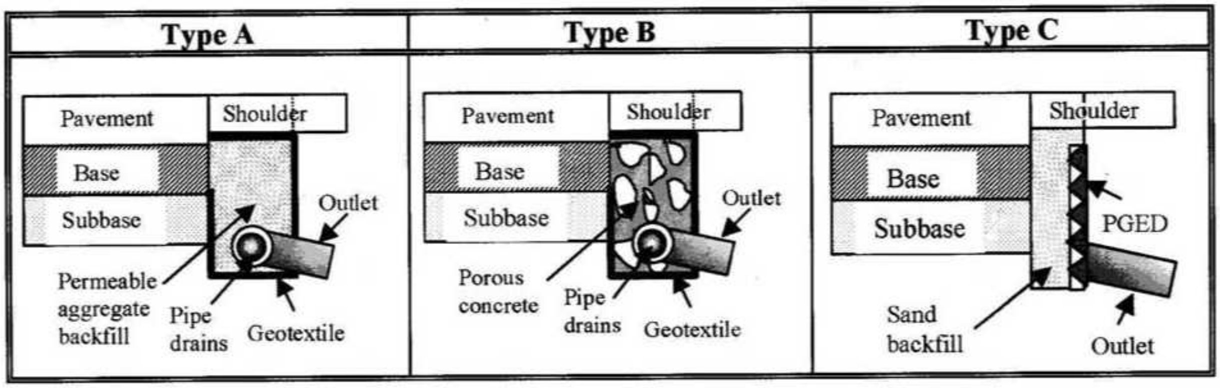


Figure 7-6. Typical edgedrains for rehabilitation projects (NCHRP 1-37A).

The effectiveness of longitudinal edgedrains depends on how they are used. Longitudinal edgedrains can be effective if used with other drainage features. Typical application of edgedrains include the following:

- New construction
  - Longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
  - Longitudinal edgedrains (pipe or geocomposite) with permeable bases.
- Existing pavement
  - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*.
- Rehabilitation projects
  - Retrofit longitudinal edgedrains (pipe or geocomposite) with nonerodible dense-graded bases*. (On projects using rubblized base or dense graded base with erodible fines, the geotextile filter in the trench should not be placed between the base and the edgedrain aggregate to avoid clogging the geotextile filter - see Figure 7-6.)

** Retrofit edgedrains are usually not recommended for pavements with dense-graded aggregate bases containing more than 15% fines (fraction passing the 0.075 mm (No. 200 sieve)). Excessive fines can clog the drains, and the loss of fines through the pipes can lead to significant base erosion. As previously indicated, dense graded base with greater than 5% fines is not anticipated to freely drain, however edgedrains can be used to effectively remove water entering the longitudinal joint (or crack) between the pavement and the shoulder.*

The field performance of edgedrains installed without a permeable base has been mixed. Some studies show little or no benefit, but others report significant improvement in pavement performance. Considering that only about 30% of all edgedrains in-service are functioning properly, mostly due to improper construction (Daleiden 1998; Sawyer 1995), the mixed report is not surprising. In many cases, the outlet pipes are crushed during construction or clogged due to inadequate maintenance. The performance of edgedrains placed in untreated dense-graded base sections seems to be dependent to a significant degree on local climatic conditions, natural drainage characteristics, subgrade type, pavement design, construction and construction inspection, and maintenance. Longitudinal edgedrains with permeable bases have been found to be effective in draining pavements and reducing moisture-related distresses when well designed, constructed, and maintained.

The type of geocomposite edgedrains used also affects performance. Older versions did not have sufficient hydraulic capacity and had not been recommended for draining permeable bases. However, some of the geocomposites available today do provide sufficient hydraulic capacity to drain permeable bases. The main disadvantage of geocomposite edgedrains is that they are difficult to maintain.

The use of aggregate trench drains, however, is not recommended because of low hydraulic capacity and inability to be maintained. An exception might be permeable cement stabilized aggregate placed in a trench.

The size of the longitudinal perforated pipe in the edgedrain is often based on maintenance requirements for cleaning capabilities and reasonable distance between outlets. Maintenance personnel should be consulted before finalizing these dimensions. The smallest diameter suitable for cleaning is 75 mm (3 in.), however many state highway agencies and the FHWA suggest a minimum pipe size of 100 mm (4 in.) based on maintenance considerations (FHWA, 1992). FHWA also recommends a maximum outlet spacing of 75 m (250 ft).

One of the most critical items for edgedrains is the grade of the invert. Construction control of very flat grades is usually not possible, leaving ponding areas that result in subgrade weakening and premature failures. Although not a popular concept, it may be more economical to raise the pavement grade to develop adequate drain slopes for the subsurface drainage facilities (*e.g.,* Florida). To achieve a desirable drainage capacity, a minimum slope may be required for the edgedrain that is greater than the slope of the road. However, this requirement may not be practical, and the pipe will mostly be sloped the same as the roadway. It is suggested that rigorous maintenance be anticipated, especially when adequate slopes cannot be achieved (FHWA, 1992).

The ditch or storm drain pipe must be low and large enough to accept the inflow from the edgedrain without backup. FHWA recommends the outlet be at least 150 mm (6 in.) above the ten-year storm flow line of the ditch or structure. The outlet should also be at a location and elevation that will allow access for maintenance activities (both cleaning and repair). Outlets and shallow pipes should be located well away from areas of expected future surface maintenance activities, such as sign replacement and catch basin cleanout or repair. FHWA also recommends angled or radius outlet connections to facilitate clean out and video inspection. Outlet headwalls, typically precast concrete, are also an essential part of the edgedrain system to prevent displacement of the outlet pipe and crushing during roadway and ditchline maintenance operations. Locations of guardrail, sign, signal, and light posts must be adjusted to prevent damage to the subsurface drainage facilities.

An offset dual pipe with a large diameter parallel collector drain line is an alternative to decrease the number of outlets (see Figure 7-7). The large diameter collector pipe (either heavy walled plastic or concrete) runs either adjacent to or below a perforated drainage pipe, as shown in Figure 7-7, to facilitate quick removal of subsurface water. The collector pipe can outflow into culverts or stormwater systems. Manholes can be installed for cleanout. These systems are quite common in Europe and have been used by a few U.S. agencies to reduce outlet maintenance issues (*e.g.,* California and, experimentally, in Kentucky).

7.2.7 Permeable Bases

A permeable base is designed to rapidly move surface infiltration water from the pavement structure to the side ditch through longitudinal edgedrains with outlets or by daylighting directly into the side ditch. Permeable bases contain no fines (0% passing the 0.075-mm (No. 200) sieve) to allow easy flow of water. In order to meet excellent drainage requirements (*i.e.,* time-to-drain of less than 2 hours from Table 7-4), permeable bases typically are required to have permeability values in excess of 300 m/day (1000 ft/day) and thicknesses of 100 mm (4 in.) (as recommended by FHWA, 1992). The performance of permeable base layers meeting these requirements will be demonstrated later in Section 7.2.12 on design of pavement drainage.

The structural capacity of angular, crushed aggregate permeable base, with a percentage of two-face crushing, is usually equivalent to the structural capacity of an equal thickness of dense-graded base. However, in order to meet these hydraulic requirements, a coarse uniform gravel must be used, which is often difficult to construct. Asphalt or cement treatments are often used to stabilize the gravel for construction, as discussed in Section 7.3. While stabilizing the base with a cement or asphalt binder will initially offer greater structural support than dense-graded base, it should be remembered that the primary purpose of the stabilizer is to provide stability of the permeable base during the construction phase. It is generally assumed that the binder will either break down or be removed by stripping with time. Thus, increase in structural support is generally not assumed for stabilized aggregate.

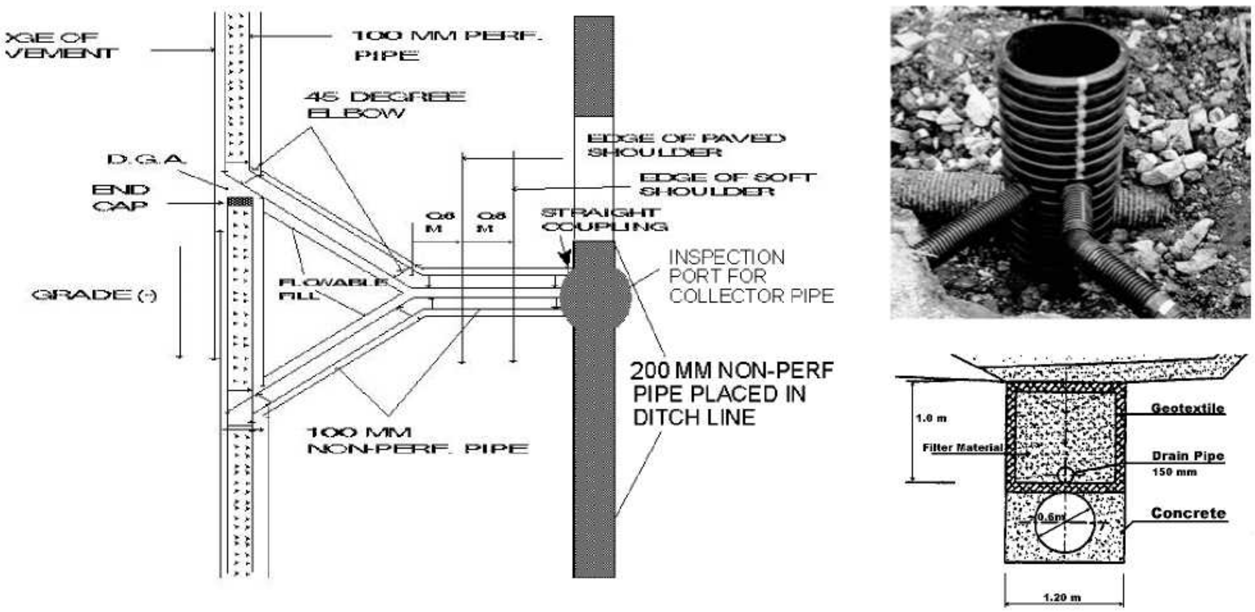


Figure 7-7. Dual pipe edgedrain systems showing alternate locations of the parallel collector pipe, either adjacent to or beneath the drain line (Christopher, 2000).

Typical cross-sections of AC and PCC pavements with permeable bases were illustrated in Figures 7-4, 7-5, and 7-6. Note that a geotextile filter should be wrapped around a portion of the trench, but not over the interface between the permeable base and drainage aggregate.

7.2.8 Dense-Graded Stabilized Base with Permeable Shoulders

This system consists of a nonerodible dense-graded base, typically lean concrete base (LCB) or asphalt treated base (ATB), under the traffic lanes and a permeable base under the shoulder. Longitudinal edgedrains are placed in the permeable base course to carry the excess moisture from the pavement structure. The recommended design for a dense-graded stabilized base with permeable shoulders is illustrated in Figure 7-8. This design offers better support under the traffic lanes where it is needed most, while still providing a means to remove water from the pavement structure. This design is now required for all high-type PCC pavements (pavements designed for more than 2.5 million equivalent single axle loads {ESALs}) in California (CALTRANS 1995).

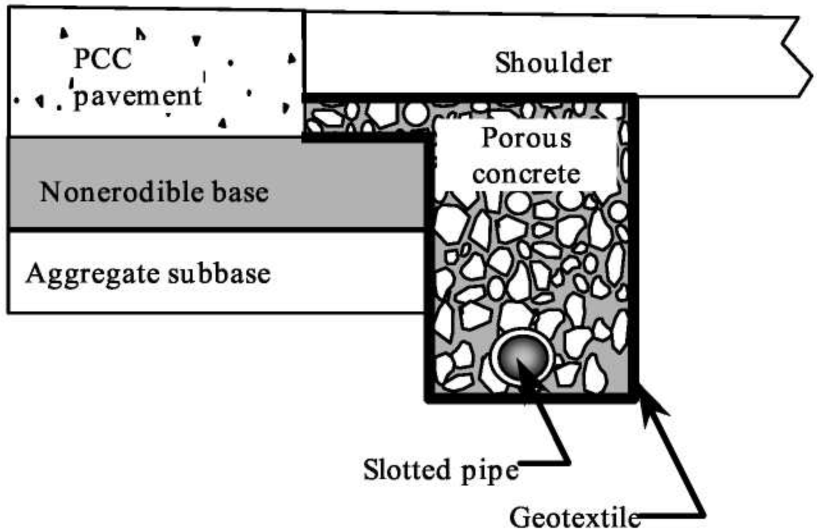


Figure 7-8. Recommended design of PCC pavement with a nonerodible dense-graded base and permeable shoulders (ERES, 1999).

7.2.9 Horizontal Geocomposite Drains

Several states (*i.e.,* Maine, Wisconsin, and Virginia) have experimented with the use of horizontal geocomposite drains, with properties sufficient to handle the estimated flow and support traffic loads, placed either below or above dense graded base, placed as a drainage layer beneath full depth asphalt, or placed between a crack and seat concrete surface and a new asphalt layer. When placed below the base aggregate, the geocomposite shortens the drainage path and reduces the time-to-drain. When placed directly beneath the pavement surface, the geocomposite intercepts and removes infiltration water before it enters the base and/or subgrade. The geocomposite is tied into an edgedrain system. Systems using this technology have been found to have excellent drainage (*i.e.,* time-to-drain of less than 2 hours from Table 7-4). Additional information, including preliminary performance information, is reported by Christopher et al. (2002).

7.2.10 Separator Layers

Separator layers play an essential role in the performance of a pavement with a permeable base by preventing fines in the underlying layers and subgrade soils from infiltrating into the permeable base, thus maintaining the permeability and effective thickness of the base course. Various combinations of materials have been used as separator layers, including the following (FHWA 1994a):

- Dense-graded aggregate (most used by far)
- Geotextiles
- Cement-treated granular material
- Asphalt chip seals
- Dense-graded asphalt concrete

These materials have been used with varying degrees of success. Lime- or cement-treated subgrades alone are not acceptable as separator layers over fine-grained soils. There have been some classic failures of lime-treated soils used as separator layers in which pumping into the permeable base caused excessive settlements.

According to a survey of 33 states, 27 used dense-graded aggregates or asphalt-treated mixtures as separator layers on a regular basis. Sixteen states used geotextiles sparingly, and 11 states used either dense-graded material or geotextiles as separator layers (Yu et al., 1998). Generally, a dense-graded aggregate or a dense-graded AC material separator layer is preferred over a geotextile for competent subgrades because the aggregate layer will provide a strong construction platform and distribute traffic loads to the subgrade. However, geotextile separator layers have been used directly beneath base layers where the additional support of a subbase is not required. For sensitive subgrades that are easily disturbed by construction (*e.g.,* silts and saturated cohesive soils), a geotextile separator layer used in conjunction with a granular subbase minimizes disturbance and provides a good construction platform. Geotextile separators also allow the use of a more open-graded, freer-draining subbase, reducing the potential for subbase saturation. Geotextiles can also be used as a separator layer in conjunction with compacted or treated subgrades, or granular subbases. If appropriately design, geosynthetics can also be used to increase subgrade support, as reviewed later in Section 7.6.5.

7.2.11 Performance of Subsurface Drainage

Many studies have shown the benefits of subsurface drainage in terms of improved performance. Cedergren (1988) believes that all important pavements should have internal drainage, claiming drainage eliminates damage, increases the life of the pavement, and is cost-effective.

Moisture-related damage to pavements has become more significant as traffic loadings have increased over the past 40 years. The annual rate of ESAL applications has virtually doubled every 10 years, causing tremendous problems related to moisture accelerated damage. A pavement may be adequately drained for one level of traffic, but as traffic increases, moisture-related damage may increase greatly. As a result, more and more states have begun to employ subsurface drainage systems (Yu et al. 1998b). Many preliminary studies indicate that drainage systems are indeed beneficial in terms of reducing certain types of pavement deterioration. However, due to some instances of poor design, construction, and/or maintenance, all have not performed as well as expected.

One example of unsatisfactory performance is some early cracking observed on a few PCC pavements with permeable bases. This occurs for a variety of reasons, including:

- Inadequate design of permeable bases and separator layers.
- Inadequate edgedrains.
- Lack of quality control during construction, such as inadequate joint sawing. Sometimes the concrete from the slab enters the permeable base, creating a thicker slab than was originally designed. Joints must be sawed deeper to ensure the proper depth is obtained to cause cracking through the joint.
- Lack of maintenance of the drainage system after the highway is open to traffic.
- Possible settlement of the PCC slab over untreated aggregate permeable bases.

Permeable bases must be constructed of durable, crushed aggregate to provide good stability through aggregate interlock. They must have a separator layer capable of preventing the pumping of fines into the permeable base from underlying layers and from preventing any intermixing of the permeable base and separator layer. Permeable bases must also have pipe edgedrains to drain the infiltrated water with suitable outlets at reasonable outlet spacing or must be daylighted directly into the ditch. Finally, to ensure good performance, the drainage system must be regularly maintained.

7.2.12 Design of Pavement Drainage

Design of pavement drainage consists of determining:

1. The hydraulic requirements for the permeable layer to achieve the required time-to-drain.
2. The edgedrain pipe size and outlet spacing requirements.
3. Either the gradation of requirements for a graded aggregate separation layer or the opening size, permeability, endurance, and strength requirements for geotextile separators.
4. The opening size, permeability, endurance, and strength requirements for geotextile filters, or the gradation of the granular filters (to be used in the edgedrain).

The following provides an outline of the design steps and procedures required for the design of each of these subsurface drainage components. Complete design details and supporting information can be found in NHI 13126 on Pavement Subsurface Drainage Design – Reference Manual (ERES, 1999).

7.2.13 Hydraulic Requirements for the Permeable Layer(s)

Basically there are two approaches to the hydraulic design of a permeable layer:

1. Time-to-drain
2. Steady-state flow.

The time-to-drain approach was previously discussed in Section 7.2.3 and simply means the time required for a percentage of the free water (*e.g.,* 50%) to drain, following a moisture event where the pavement section becomes saturated. In the steady-state flow approach, uniform flow conditions are assumed, and the permeable layer is designed to drain the water that infiltrates the pavement surface. The time-to-drain approach will be the basis for design in this manual, as it is currently the procedure recommended by the FHWA, AASHTO, and NCHRP 1-37A for pavement design. Elements of steady state flow will be used to determine outlet spacing. (For additional discussion of steady state flow methods see FHWA, 1992 and ERES, 1999.)

The time-to-drain approach assumes the flow of water into the pavement section until it becomes saturated (the drainage layer plus the material above the drainage layer). Excess precipitation will not enter the pavement section after it is saturated; this water will simply run off the pavement surface. After the rainfall event, the drainage layer will drain to the edgedrain system. Engineers must design the permeable layer to drain relatively quickly to prevent the pavement from being damaged.

A time-to-drain of 50% of the drainable water in 1 hour is recommended as a criterion for the highest class roads with the greatest amount of traffic (FHWA, 1992). For most other high use roadways, a time-to-drain of 50% of the drainable water in 2 hours is recommended. For secondary roads, a minimum target value of 1 day is recommended (U.S. Army Corps of Engineers, 1992). Remember, in all cases, the goal of drainage is to remove all drainable water as quickly as possible.

The time-to-drain is determined by the following equation:

$$t = T \times m \times 24 \qquad \textit{Eq. 7.1}$$

where,
t = time-to-drain in hours
T = Time Factor
m = "m" factor (see Eq. 7.3)

A simplified design chart for determining a time-to-drain of 50% time factor, T_{50}, is provided in Figure 7-9. This chart was developed for one degree (*i.e.,* direction) of drainage and is adequate for most designs. For expanded charts to cover additional degrees of drainage and desired percent drained see FHWA, 1992 and ERES, 1999.

The time factor is based on the geometry of the drainage layer (e.g., the permeable base layer). The geometry includes the resultant slope (S_R) and length (L_R); the thickness of the drainage layer (H), which is the length the water must travel within a given layer; and, the percent drained (U), (i.e., 50%). S_R and L_R are based on the true length of drainage and are determined by finding the resultant of the cross and longitudinal pavement slopes (S_X and S, respectively) and lengths (L_x and L, respectively). The resultant length is measured from the highest point in the pavement cross-section to the point where drainage occurs (*i.e.,* edgedrain or daylighted section). First, the slope factor (S_l) must be calculated:

$$S_1 = \frac{L_R \, S_R}{H} \qquad \textit{Eq. 7.2}$$

where, S_R = $(S^2 + S_X^{\,2})^{½}$
L_R = $W\,[1 + (S/S_X)^2]^{½}$
W = width of permeable layer in m (ft)
H = thickness of permeable layer in m (ft)
1 ft = 0.3 m

Figure 7-9 is then entered with the S_l, and the resulting T_{50} to be used in Eq. 7.1 is determined.

The "m" factor in Eq 7.1 is determined by the equation:

$$m = \frac{N_o \, L_R^{\,2}}{kH} = \frac{N_o \, L_R^{\,2}}{\psi} \qquad Eq.7.3$$

where, N_o = the effective porosity of the drainage layer
k = permeability of drainage layer in m/day (ft/day)
H = thickness of drainage layer in m (ft)
ψ = the transmissivitty of the drainage layer in m^2/day (ft^2/day)
1 ft = 0.3 m

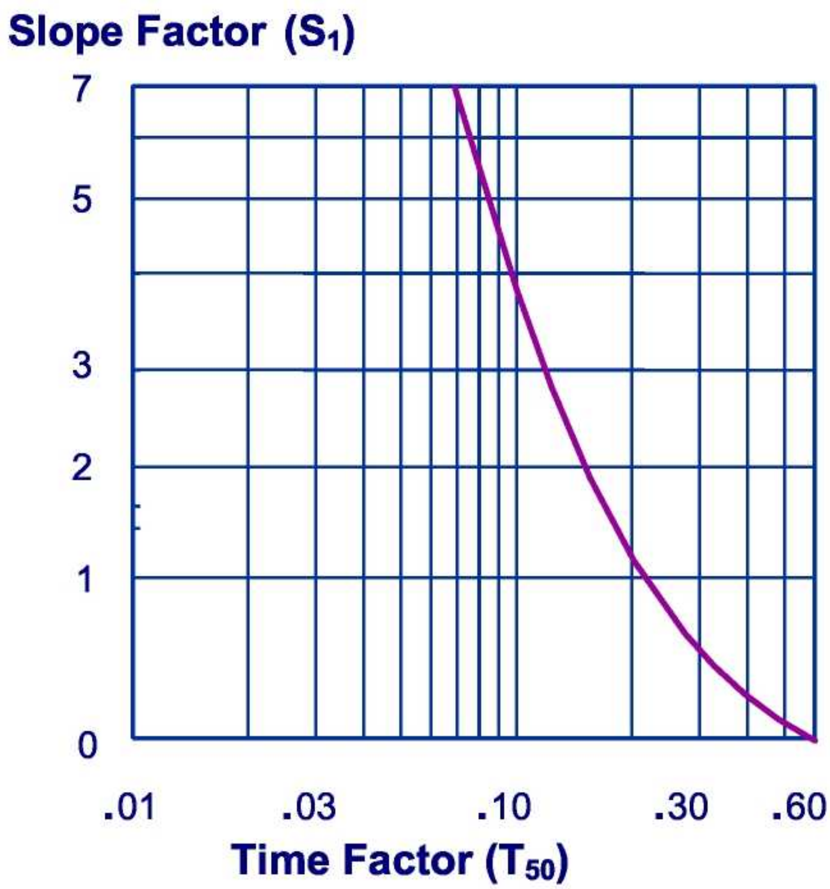


Figure 7-9. Time Factor for 50% Drainage (ERES, 1999).

The intrinsic factors that represent the drainage capabilities of drainage layer base are represented by the effective porosity (N_o) and the coefficient of permeability (k) or, if H is known, the transmissivity of the drainage layer. The effective porosity is the ratio of the volume of water that can drain under gravity from the material to the total volume of the material. It is a measure of the amount of water that can be drained from a material. The value can be easily determined by saturating a sample of material and measuring the amount of water that drains. Additional information on the determination of these characteristics for aggregate drainage layers are covered in detail in FHWA, 1992 and NHI 13126.

For example, using the recommended 4-inch-thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) at a cross slope of 2% in a relatively flat (1% grade) road alignment would produce the following time-to-drain for a four lane road draining from the center (W = 7.3 m (24 ft)):

$S_R = (S^2 + S_X^2)^{½} = (0.01^2 + 0.02^2)^{½} = 0.022$

$L_R = W((1 + (S/S_X)^2)^{½} = 24 \text{ ft } [1 + (0.01/0.02)^2]^{½} = 26.8 \text{ ft}$

$S_l = (L_R\ S_R)/H = (26.8 \text{ ft} \times 0.022)/\ 0.33 \text{ ft} = 1.8$

$m = (N_e L_R^2) / (kH) = (0.25 \times (26.8 \text{ ft})^2)/(1000 \text{ ft/day} \times 0.33 \text{ ft}) = 0.54 \text{ days}$

From Figure 7-9 with $S_l = 1.8$, $T = 0.16$

Therefore, $t = T \times m \times 24 = 0.16 \times 0.54 \text{ days} \times 24 \text{ hrs/day} = 2.1 \text{ hrs}$

Since the time-to-drain is close to 2 hrs, the drainage layer would provide excellent drainage, as defined in Table 7-4.

According to a sensitivity analysis on time-to-drain performed in ERES, 1999, time-to-drain is most sensitive to changes in the coefficient of permeability and the resultant slope, decreasing exponentially with increasing permeability and slope values. Time to drain increases linearly with increasing length and effective porosity, while thickness has very little effect.

The DRIP microcomputer program developed by FHWA can be used to rapidly evaluate the effectiveness of the drainage system and calculate the design requirements for the permeable base design, separator, and edgedrain design, including filtration requirements. The program can also be used to determine the drainage path length based on pavement cross and longitudinal slopes, lane widths, edgedrain trench widths (if applicable), and cross-section geometry crowned or superelevated. The software can be downloaded directly from the FHWA WEB page http://www.fhwa.dot.gov/pavement/library.htm and is included with the NCHRP 1-37A pavement design software.

7.2.14 Edgedrain Pipe Size and Outlet Spacing Requirements

The FHWA recommends a minimum pipe diameter of 100 mm (4 in.) and a maximum outlet spacing of 75 m (250 ft) to facilitate cleaning and video inspection. The adequacy of these requirements can be confirmed by evaluating the anticipated infiltration rate or, more conservatively, from the maximum flow capacity of the drainage layer.

With the flow capacity method, the estimated discharge rate from drainage layer is determined. For example, the conventional 100-mm (4-in.) thick open-graded base layer with a permeability of 300 m/day (1000 ft/day) used in the previous time-to-drain example provides excellent drainage for most conditions (FHWA, 1992). This 100-mm (4-in.) thick free-draining base layer has a transmissivity (*i.e.,* permeability multiplied by the thickness) of about 28 m^2/day (300 ft^2/day). For a typical roadway gradient of 0.02 (for a 2% grade), the open-graded base layer has a flow capacity of 0.13 ft^3/day (6 ft^3/day) per ft length of road. Thus at an outlet spacing of 75 m (250 ft), the quantity of flow at the discharge (Q) of the edgedrain system would be 33 m^3/day (1500 ft^3/day).

The capacity of a circular pipe flowing full can be determined by Manning's equation:

$$Q = \frac{53.01}{n} D^{8/3} S^{1/2} \qquad \textit{Eq. 7.4}$$

where,	Q	= Pipe capacity, cu ft/day
	D	= Pipe diameter, in.
	S	= Slope, ft/ft
	n	= Manning's roughness coefficient
		= 0.012 for smooth pipe
		= 0.024 for corrugated pipe
	1 ft	= 0.3 m
	1 in	= 25.4 mm

Thus, for a 100-mm (4-in.) smooth wall pipe at a 1% grade, the flow capacity is 504 m^3/day (17800 ft^3/day), which is more than adequate to handle the maximum quantity of flow anticipated for the edgedrain system. However, the 100-mm (4-in.) pipe is still recommended to facilitate inspection and cleaning.

In the infiltration method, a design rainfall and an infiltration ratio are selected. Pavement infiltration is determined by the equation

$$q_i = C \times R \times 1/12 \text{ (ft/in)} \times 24 \text{ (hr/day)} \times 1 \text{ ft} \times 1 \text{ ft)} \qquad \textit{Eq. 7.5}$$

which can be simplified to:

$$q_i = 2\,C\,R \qquad \textit{Eq. 7.5a}$$

where,	q_i	= Pavement infiltration, ft^3/day/ft^2 of pavement
	C	= Infiltration ratio
	R	= Rainfall rate, in./hr

The infiltration ratio C represents the portion of rainfall that enters the pavement through joints and cracks. The following design guidance for selecting the infiltration coefficient is suggested (FHWA, 1992):

Asphalt concrete pavements	0.33 to 0.50
Portland cement concrete pavements	0.50 to 0.67

To simplify the analysis and provide an adequate design, FHWA suggest using a value of 0.5. The design storm whose frequency and duration will provide an adequate design must be selected. A design storm of 2-year frequency, 1-hour duration, is suggested. Figure 7-10 provides a map of generalized rainfall intensity.

The analysis is then performed by substituting into the above equation for the specific region of the country. The drainage layer discharge rate q_d can then be determined by multiplying the infiltration rate by the resultant length of the pavement section L_R as follows:

$$q_d = q_i \, L_R \qquad \textit{Eq. 7.6}$$

This discharge rate can then be compared to the flow capacity of the drainage layer and the lower value of the two used to evaluate the outlet spacing and pipe size.

7.2.15 Separator Layer

As indicated in the previous section, the separator consists of a layer of aggregate material (treated or untreated) or a geotextile layer placed between the permeable base and the subgrade or other underlying layers. The separator layer has to maintain separation between permeable base and subgrade, and prevent them from intermixing and support construction traffic. It may also be desirable to use a low permeable layer that will deflect water from the permeable base horizontally toward the pavement edge (NCHRP 1-37A).

If dense-graded aggregate separator layers are used, the aggregate must be a hard, durable material. Based on FHWA guide specifications for materials selection and construction of aggregate separation layers, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces, as determined by the material retained on the 4.74 mm (No. 4) sieve; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 50%, as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine.*

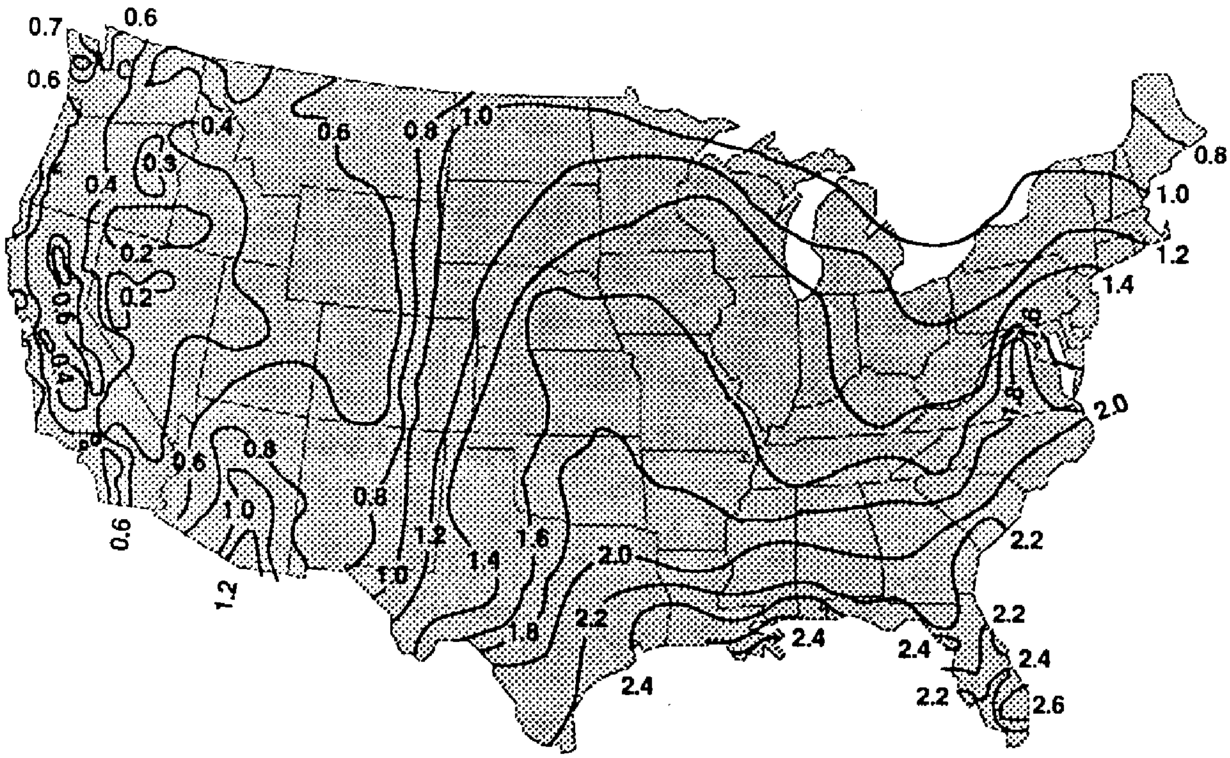


Figure 7-10. Rainfall Intensity in in./hr for a 2-year, 1-hour Storm Event (FHWA, 1992).

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test shall be in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate.*
- The gradation of this layer should be such that it allows a maximum permeability of approximately 5 m/day (15 ft/day) with less than 12% of the material passing the 0.075 mm (No. 200) sieve, by weight.
- Material passing the 425 mm (No. 40) sieve shall be nonplastic, in accordance with AASHTO T 90, "Determining the Plastic Limit and Plasticity Index of Soils."

7.2.16 Geotextile Separator and Filter Design

As a separator, just as with the granular layer, the geotextile must prevent the intermixing of the permeable base and the adjacent subgrade or subbase layer. Also as with aggregate separator layers, the geotextile layers will have to satisfy filtration criteria. In order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria. Additional requirements for subgrade improvement are reviewed in Section 7.6.5. Both woven and non-woven geotextiles have been used for the separation application. The criteria for filtration and survivability are outlined in the following paragraphs and are basically the same as that required for the edgedrain geotextile filters. The only notable exception is that the separation layer can have a much lower permeability (compatible with the subgrade) than the edgedrain filter (compatible with the permeable base).

As a filter for the edgedrain, the geotextile must be designed to allow unimpeded flow of water into edgedrain system over the life of the system. The geotextile must prevent soil from washing into the system without clogging over time. The FHWA presents three basic principles for geotextile design and selection (Holtz et al., 1998):

1. If the larger pores in the geotextile filter are smaller than the largest particles of soil, these particles will not pass the filter. As with graded granular filters, the larger particles of soil form a filter bridge on the geotextile, which, in turn, filters the smaller particles of the soil. Thus, the soil is retained and particle movement and piping is prevented.
2. If the smaller openings in the geotextile are sufficiently large so that the smaller particles of soil are able to pass through the filter, then the geotextile will not clog.
3. A large number of openings should be present in the geotextile so that proper flow can be maintained even if some of the openings later become clogged.

The geotextile filtration characteristics must be checked for compatibility with the gradation and permeability of the subgrade. The requirements for proper performance can be appropriately selected by using the following design steps.

Step 1. Determine the gradation of the material to be separated/filtered. The filtered material is directly above and below the geocomposite drainage layer. Determine D_{85}, D_{15} and percent finer than a 0.075 mm (No. 200) sieve.

Step 2. Determine the permeability of the base or subbase $k_{base/subbase}$, whichever is located directly above the geocomposite drainage layer. (For placement directly beneath the hot-mix or PCC pavement applications, the default permittivity requirement will be used.

Step 3. Apply design criteria to determine apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile (after Holtz et al., 1998)

$AOS \leq D_{85\ base/subbase}$ (For woven geotextile)

$AOS \leq 1.8\ D_{85\ subgrade}$ (For nonwoven geotextile)*

$k_{geotextile} \geq k_{base/subbase}$

$\psi \geq 0.1\ sec^{-1}$

* *For noncohesive silts and other highly pumping susceptible soils, a filter bridge may not develop, especially considering the potential for dynamic, pulsating flow. A conservative (smaller) AOS $\leq D_{85\ subgrade}$ is advised, and laboratory filtration tests are recommended.*

Step 4. In order to perform effectively, the geotextile must also survive the installation process. AASHTO M288 (1997) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 7-5. Use Class 2 where a moderate level of survivability is required (*i.e.,* for subgrade CBR > 3, where at least 150 mm (6 in.)) of base/subbase and normal weight construction equipment is anticipated, and where filters are used in edgedrains). Class 1 geotextiles are recommended for CBR < 3 and when heavy construction equipment is anticipated. For separation layers, a minimum of 150 mm (6 in.) of base/subbase should be maintained between the wheel and geotextile at all times.

In projects using recycled concrete, rubblizing, or crack-and-seat techniques, geotextiles and granular filters are susceptible to clogging by precipitate and should not be indiscriminately used to separate the permeable base from the drain or wrapped around pipes. Geotextiles should not be placed between the recycled material and the drain, but could be placed beneath and on the outside of the drain to prevent infiltration of the subgrade and subbase layers (see Figure 7-2.)

Table 7-5. Geotextile survivability requirements (AASHTO M 288-96).

Test	Test Method	Units	Geotextile Class			
			Class 1		Class 2	
			< 50%*	≥ 50%*	< 50%*	≥ 50%*
Grab Strength	ASTM D 4632	N	1400	900	1100	700
Seam Strength	ASTM D 4632	N	1200	810	990	630
Tear Strength	ASTM D 4533	N	500	350	400	250
Puncture Strength	ASTM D 4833	N	500	350	400	250
Burst Strength	ASTM D 3786	kPa	3500	1700	2700	1300

*Note: Elongation measured in accordance with ASTM D 4632 with < 50% typical of woven geotextiles and ≥ 50% typical of nonwoven geotextiles. (1 N = 0.22 lbs, 1 kPa = 0.145 psi)

7.3 BASE LAYERS: REQUIREMENTS, STABLILIZATION & REINFORCEMENT

The function of the base course varies according to the type of pavement, as was described in Chapter 1. Under rigid pavements, the base course is used to: (1) provide uniform and stable support, (2) minimize damaging effects of frost action, (3) provide drainage, (4) prevent pumping of fine-grained soils at joints, (5) prevent volume change of the subgrade, (5) increase structural capacity of the pavement, and (6) expedite construction. Under flexible pavements, the prime function of the base course is to structurally improve the load-supporting capacity of the pavement by providing added stiffness and resistance to fatigue, as well as to provide a relatively thick layer to distribute the load through a finite thickness of pavement. The base may also provide drainage and give added protection against frost action where necessary.

To meet these functional requirements, the base course as a minimum should have the following characteristics:

- To prevent pumping, a base course must be either free draining or it must be highly resistant to the erosive action of water. Erodibility is covered in more detail in the next section.
- To provide drainage, the base course may or may not be a well-graded material, but it should contain little or no materials finer than a 0.075 mm (No. 200) sieve. It may sometimes be stabilized with asphalt or cement.
- A base course design for frost action should be non-frost susceptible and free draining.
- To improve resistance to deformation and improve structural support or reduce the thickness, it may be desirable to stabilize the base course with asphalt or cement, as reviewed in Section 7.3.2 and 7.3.3, or to reinforce it with geosynthetics, as reviewed in Section 7.3.4.
- A base course need not be free draining to provide structural capacity, but it should be well-graded and should resist deformation due to loading.

The aggregate used for base must be hard, durable material. As a minimum, the aggregate should meet the following requirements:

- The aggregate should have at least two fractured faces; preferably, it should consist of 98% crushed stone.
- The L.A. abrasion wear should not exceed 45% as determined by AASHTO T 96, *Resistance to Abrasion of Small Size Coarse Aggregate by Use of Los Angeles Machine*.

- The soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively. The test should be performed in accordance with AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate* (see Chapter 5).
- For permeable base, the gradation of this layer should enable free movement of water with a minimum permeability value around 300 m/day (1,000 ft/day) (see Section 7.2) and the material passing the 0.425 mm (No. 40) sieve should be non-plastic in accordance with AASHTO T 90, *Determining the Plastic Limit and Plasticity Index of Soils*.

7.3.1 Erodibility of Bases

Preventing significant erosion of the base and subbase materials is very important for the control of moisture-related distresses, such as pumping and faulting in JPCP and punchouts in CRCP, as discussed in NCHRP 1-37A. Erodibility is the loss of base material due to hydraulic action, most often at the joints in rigid pavements, but also along the edge of both rigid and flexible pavements. The condition is related to the durability of the base in relation to its potential to break down under dynamic traffic loads, climatic conditions, environmental effects, as well as water action. As truck traffic increases, a more erosion resistant base is required, along with more adequate joint load transfer design (*e.g.,* use of dowels in joints). Traffic level is a very critical factor in the consideration of base/subbase course erosion, especially considering that the base/subbase under PCC slabs of reconstructed projects will likely receive 10 to 20 times more load repetitions over their design life than in the past.

While the base course is the layer most often affected by erosion, any layer directly beneath a treated base can experience serious erosion. There are many examples of the erosion of fine grained soils beneath a stabilized base course causing loss of support and joint faulting. Thus, some agencies now place a dense graded granular subbase layer between the base and compacted subgrade to reduce this problem. Other agencies stabilize the top layer of a fine-grained soil with lime to reduce this problem; however, this approach must produce a sufficiently hard material with adequate compressive strength and uniformity along the project. Geotextiles are also used as separation layers to hold the subgrade materials in place. Another alternative that has been used successfully is to place a layer of recycled crushed PCC beneath the dense treated base.

The NCHRP 1-37A guide provides guidance for assessing the erodibility potential of various materials used in new JPCP and CRCP design and in PCC overlays of existing flexible or rigid pavements. The effect of erosion is considered empirically in the form of erodibility classification assessment for specific design levels. The design procedure provides the framework for which erosion can be considered on a more mechanistic basis in the future (such as iterative month-by-month damage accumulation, and inclusion of Level 1 laboratory erosion test). Tables 7-6, 7-7, and 7-8 provide the Material Classification requirements for Level 1, Level 2, and Level 3 design, respectively.

7.3.2 Bound Bases

In order to achieve the highest erodibility levels, stabilized base or subbase materials often produced by the addition of a sufficient quantity of stabilizing agent (usually cement or asphalt) to produce materials with significant tensile strength (*e.g.,* Erodibility Class 1a in Table 7-7). Such materials are considered to be bound bases and have a substantial increase in structural capacity over that of unbound and modified (treated) bases. Bound bases or subbases are not considered to be geotechnical materials, and are not covered in this manual. Users are referred to NHI courses on pavements (*e.g.,* NHI 131033) for additional information.

Table 7-6. Level 1 recommendation for assessing erosion potential of base material (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
Class based on the material type and test results	Test not fully developed for nationwide uses; thus Level 1 cannot be implemented at this time. The tests currently being considered to assess the erodibility of paving materials include - Rotational shear device for cohesive or stabilized materials (Bhatti et al., 1996). - Jetting test (Bhatti et al., 1996). - Linear and rotational brush tests (Dempsey, 1982). - South African erosion test (DeBeer, 1990).

Table 7-7. Design Level 2 recommendations for assessing erosion potential of base material
(NCHRP 1-37A adapted after the Permanent International Association of Road Congresses, PIARC, 1987).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8% cement; or with long-term compressive strength > 17.2 MPa (2,500 psi) [> 13.8 MPa (2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the bound base and subgrade; otherwise Class 2. (b) Hot mixed asphalt concrete with 6% asphalt cement that passes appropriate stripping tests and aggregate tests and a granular subbase layer or a stabilized soil layer; otherwise Class 2. (c) (c) Permeable drainage layer (asphalt-treated aggregate or cement-treated aggregate) and with an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement-treated granular material with 5% cement manufactured in-plant, or long-term compressive strength 13.8 to 17.2 MPa (2,000 to 2,500 psi) [10.3 MPa to 13.8 MPa (1,500 to 2,000 psi) at 28-days] and a granular subbase layer or a stabilized soil layer or a geotextile fabric is placed between the treated base & subgrade; otherwise Class 3. (b) (b) Asphalt-treated granular material with 4% asphalt cement that passes appropriate stripping test and a granular subbase layer or a treated soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class 3.
3	(a) Cement-treated granular material with 3.5% cement manufactured in-plant, or with long-term compressive strength 6.9 MPa to 13.8 MPa (1,000 to 2,000 psi) [5.2 MPa to 10.3 MPa (750 to 1,500 psi) at 28-days]. (b) Asphalt-treated granular material with 3% asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade).

Table 7-8. Design Level 3 recommendations for assessing erosion potential of base material based on material description only (NCHRP 1-37A).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile layer is placed between the treated base and subgrade; otherwise Class B. (b) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade; otherwise Class B. (c) Permeable drainage layer (asphalt- or cement-treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade. Unbonded PCC Overlays: HMAC separation layer (either dense or permeable graded) is specified.
2	(a) Cement-treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile layer is placed between the treated base and subgrade; otherwise Class C. (b) Asphalt-treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile is placed between the treated base and subgrade; otherwise Class C.
3	(a) Cement-treated granular material that has exhibited some erosion and pumping in the past. (b) Asphalt-treated granular material that has exhibited some erosion and pumping in the past. Unbonded PCC Overlays: Surface treatment or sand asphalt is used.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated subgrade soils (compacted).

7.3.3 Modified (or Treated) Bases

The addition of cement or asphalt (typically less than 5%) to stabilize unbound base or subbase with the primary purpose of improving the stability for construction are considered to be modified or treated bases. Modified materials are usually considered to behave structurally as unbound granular material. These bases or subbases are considered to be geotechnical materials. Stabilization is most often required for open graded (permeable) bases (OGB), which tend to rut and weave under construction activities. Tables 7-9 and 7-10 provide the recommendations for asphalt-treated bases and cement-treated bases, respectively.

The strength of cement-treated bases will depend in part on adequate curing during construction. The mixture must be well compacted at optimum moisture content, and adequate density must be obtained throughout the layer. Density control will also be important for the uniformity of asphalt-treated base materials. Although stabilization is often used to reduce the thickness of the base, it should be recognized that thin bases (less than 150 mm (6 in.) thickness) are often extremely difficult to construct to the exact depth, creating the potential for very thin base layers in localized areas. Construction of thin bases requires a very competent subgrade or a good working platform (as reviewed in Section 7.6). Construction quality control for cement- and asphalt-treated materials is reviewed in Chapter 8.

Table 7-9. Recommended asphalt stabilizer properties for asphalt-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) hard, durable material with at least two fractured faces; preferably, consisting of 98% crushed stone. (b) L.A. abrasion wear should not exceed 45%. (c) Soundness loss percent should not exceed 12 as determined by the sodium sulfate, or 18% by the magnesium sulfate tests.	Visual Classification AASHTO T 96 AASHTO T 104, *Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate*
AC content	AC content must ensure that aggregates are well coated. Minimum recommended AC content is between 2.5 – 3% by weight. Final AC content should be determined according to mix gradation and film thickness around the coarse aggregates.	ASTM D 2489, *Test Method for Degree of Particle Coating of Bituminous-Aggregate Mixtures.*
AC grade	A stiff asphalt grade (typically 1 grade stiffer than the surface course is recommended).	Penetration, viscosity, or Superpave binder testing can be performed to determine AC grade.
Anti-stripping	Anti-stripping test should be performed on all AC treated materials.	AASHTO T283, *Resistance of Compacted Bituminous Mixture to Moisture Induced Damage.*
Anti-stripping Agents	Aggregates exhibiting hydrophilic characteristics can be counteracted with 0.5 – 1% lime.	NCHRP Report 274.
Permeability	Minimum mix permeability: 300 m/day (1000 ft/day).	AASHTO T 3637, *Permeability of Bituminous Mixtures.*

Table 7-10. Recommended Portland cement stabilizer properties for cement-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Aggregate	(a) Hard, durable material with at least two fractured faces; preferably, consisting of 98 percent crushed stone.	Visual Classification
	(b) (a) L.A. abrasion wear should not exceed 45%.	AASHTO T 96-94
	(c) (c) Soundness loss percent should not exceed 12 or 18%, as determined by the sodium sulfate or magnesium sulfate tests, respectively.	AASHTO T 104-86, "Soundness of Aggregate ...Use of Sodium Sulfate or Magnesium Sulfate"
Cement	Portland cement content selected must ensure that aggregates are well coated. An application rate of 130 to 166 kg/m^3 (220 to 285 lb/yd^3) is recommended.	Must conform to the specification of AASHTO M 85, *Portland Cement*
Water-to-cement ratio	Recommended water-to-cement ratio to ensure strength and workability: 0.3 to 0.5.	
Workability	Mix slump should range between 25 – 75 mm (1 – 3 in.).	
Cleanness	Use only clean aggregates	
Permeability	Minimum mix permeability: (300 m/day) 1,000 ft/day.	

7.3.4 Base Reinforcement

A more recent form of stabilization is the use of geosynthetics (primarily geogrids) to reinforce the base for flexible pavement systems, which has been found under certain conditions to provide significant improvement in performance of pavement sections. The principal effect of reinforcement in base-reinforced flexible pavements is to provide lateral confinement of the aggregate layer. Lateral confinement arises from the development of interface shear stresses between the aggregate and the reinforcement, which, in turn, transfers load to the reinforcement. The interface shear stress present when a traffic load is removed continues to grow with traffic load applications, meaning that the lateral confinement of the aggregate increases with increasing load applications. Increases in traffic volume up to a factor of 10 to reach the same distress level (25-mm (1 in.) rutting) have been observed for reinforced sections, versus unreinforced sections of the same design asphalt and base thickness (Berg et al., 2000). Table 7-11 provides a summary of the conditions for which various geosynthetic products should be considered for this application.

Table 7-11. Qualitative review of reinforcement application potential for paved permanent roads (after Berg et al., 2000).

Roadway Design Conditions		Geosynthetic Type					
Subgrade	Base / Subbase Thickness[1] (mm)	Geotextile		Geogrid [2]		GG-GT Composite	
		Nonwoven	Woven	Extruded	Knitted or Woven	Open-Graded Base[3]	Well-Graded Base
Soft (CBR < 3) (M_R <30 MPa)	150 – 300	❹	●	●	❑	●	⑤
	> 300	❹	❹	◗	◗	◗	⑤
Firm - Vy. Stiff (3 ≤ CBR ≤ 8) (30 ≤ MR ≤ 80)	150 – 300	❍	◗	●	❑	●	⑤
	> 300	❍	❍	❍	❍	❍	❍

KEY: ● — usually applicable ◗ — applicable for some conditions
❍ — usually not applicable ❑ — insufficient information at this time ⑤ — see note

NOTES: 1. Total base or subbase thickness with geosynthetic reinforcement. Reinforcement may be placed at bottom of base or subbase, or within base for thicker (usually > 300 mm (12 in.)) thicknesses. Thicknesses less than 150 mm (6 in.) not recommended for construction over soft subgrade. Placement of less than 150 mm (6 in.) over a geosynthetic not recommended.
2. For open-graded base or thin bases over wet, fine grained subgrades, a separation geotextile should be considered with geogrid reinforcement.
3. Potential assumes base placed directly on subgrade. A subbase also may provide filtration.
❹ Reinforcement usually applicable, but typically addressed as a subgrade stabilization.
⑤ Geotextile component of composite likely is not required for filtration with a well-graded base course; therefore, composite reinforcement usually not applicable.

Current design methods for flexible pavements reinforced with a geosynthetic in the unbound aggregate base layer are largely empirical methods based on a limited set of design conditions over which test sections have been constructed (*i.e.,* AASHTO 4E-SR Standard of Practice Guidelines for Base Reinforcement). These design methods have been limited in use due to 1) absence of nationally recognized reinforced base design procedure, 2) narrow range of test section design conditions from which the method was calibrated, and 3) proprietary design methods pertaining to a single geosynthetic product. Recently FHWA sponsored a study to develop an interface for including geosynthetic base reinforcement in mechanistic empirical design, consistent with the NCHRP 1-37A model. This work is currently in review, but shows excellent promise for the incorporation of these methods into pavement design.

In the interim, AASHTO 4E includes a design approach that relies upon the assessment of reinforcement benefit as defined by a Traffic Benefit Ratio (TBR) or a Base Course reduction Ratio (BCR). TBR is defined as the ratio of the number of traffic loads between an otherwise identical reinforced and unreinforced pavement that can be applied to reach a particular permanent surface deformation of the pavement. BCR defines the percentage reduction in the base course thickness of a reinforced pavement such that equivalent life (*e.g.,* surface deformation) is obtained between the reinforced and the unreinforced pavement with the greater aggregate thickness. The philosophy of this approach is one in which applicability of the technology and reinforcement benefits are assessed by empirical considerations. Reinforcement benefit defined in this manner is then used to modify an existing unreinforced pavement design.

The proposed design procedure in AASHTO 4E follows the steps listed below:

Step 1. Initial assessment of applicability of the technology.
Step 2. Design of the unreinforced pavement.
Step 3. Definition of the qualitative benefits of reinforcement for the project.
Step 4. Definition of the quantitative benefits of reinforcement (TBR or BCR).
Step 5. Design of the reinforced pavement using the benefits defined in Step 4.
Step 6. Analysis of life-cycle costs.
Step 7. Development of a project specification.
Step 8. Development of construction drawings and bid documents.
Step 9. Construction of the roadway.

Step 1 involves assessing the project-related variables given in Table 7-11 and making a judgment on whether the project conditions are favorable or unfavorable for reinforcement to be effective and what types of reinforcement products (as defined in Table 7-11) are appropriate for the project.

Step 2 involves the design of a conventional unreinforced typical pavement design cross section or a series of cross sections, if appropriate, for the project. Any acceptable design procedure can be used for this step.

Step 3 involves an assessment of the qualitative benefits that will be derived by the addition of the reinforcement. The two main benefits that should be assessed are whether the geosynthetic will be used for an extension of the life of the pavement (*i.e.,* the application of additional vehicle passes), a reduction of the base aggregate thickness, or a combination of the two. Berg et al. (2000) has listed additional secondary benefits that should also be considered.

Step 4 is the most difficult step in the design process and requires the greatest amount of judgment. This step requires the definition of the value, or values, of benefit (TBR and/or BCR) that will be used in the design of the reinforced pavement. The definition of these benefit values for a range of design conditions is perhaps the most actively debated and most currently studied topic within this field. Given the lack of a suitable analytical solution for the definition of these terms, Berg et al. (2000) has suggested that these values be determined by a careful comparison of project design conditions, as defined in previous steps, to conditions present in studies reported in the literature. The majority of these studies have been summarized in Berg et al. (2000) in a form that allows direct comparison to known project conditions. In the absence of suitable comparison studies, an experimental demonstration method involving the construction of reinforced and unreinforced pavement test sections has been suggested and described in Berg et al. (2000), and may be used for the definition of benefit for the project conditions. The reasonableness of benefit values should be carefully evaluated such that the reliability of the pavement is not undermined.

Step 5 involves the direct application of TBR or BCR to modify the unreinforced pavement design defined in Step 2. TBR can be directly used to define an increased number of vehicle passes that can be applied to the pavement, while BCR can be used to define a reduced base aggregate thickness such that equal life results. Within the context of an AASHTO pavement design approach, it is possible to calculate a BCR knowing a TBR and vise versa for the specific project design conditions, however this approach has not been experimentally or analytically validated.

With the unreinforced and reinforced pavement designs defined, a life-cycle cost analysis should be performed to assess the economic benefit of reinforcement. This step will dictate whether it is economically beneficial to use the geosynthetic reinforcement. Remaining steps involve the development of project specifications, construction drawings, bid documents, and plans for construction monitoring. Berg et al. (2000) has presented a draft specification that may be adopted for this application.

Even though the application of geosynthetic reinforcement of flexible pavements has been proposed and examined over the past 20 years, research in this area is quite active, meaning that new design methods should be expected in the near future. These new design methods will hopefully provide less empirical methods for assessing reinforcement benefit and be expressed as a function of the variables that are known to influence benefit.

7.4 COMPACTION

Compaction of the subgrade, unbound base, and subbase materials is a basic design detail and is one of the most fundamental geotechnical operations for any pavement project. Compaction is used to increase the stiffness and strength, decrease the permeability, and increase the erosion resistance of geomaterials. Compaction can also reduce the swelling potential for expansive soils. Thus, the intent of compaction is to maximize the soil strength (and minimize the potential volume change) by the proper adjustment of moisture and the densification at or near the ideal moisture content, as discussed in this section.

In most instances, once heavy earthwork and fine grading is completed, the uppermost zone of subgrade soil (roadbed) is improved. The typical improvement technique is by means of water content adjustments and densification by compaction. Higher density requirements are routinely established for the top two feet of at-grade roadbeds and for embankments. The soil in cut areas may need to be undercut and backfilled to obtain the strength and uniformity desired. Heavy proof rolling equipment (270 to 450 kN (30 to 50 tons)) can be used to identify areas of non-uniform support in prepared subgrades. Proofrolling and other field construction aspects of compaction are covered in Chapter 8. Perhaps the most common problem arising from deficient construction is related to moisture-density control, which can be avoided or at least minimized with a thorough plan and execution of the plan as it relates to QC/QA during construction, as reviewed in Chapter 8. This plan should pay particular attention to proper moisture content, proper lift thickness for compaction, and sufficient configuration (e.g., weight and width) of the compaction equipment utilized.

7.4.1 Compaction Theory

The basic engineering principles of soil compaction date back to work by Proctor in the 1930s. Compaction can be performed in the laboratory using static, kneading, gyratory, vibratory, or impact compactors. Each method has its advantages and disadvantages, but impact compaction using a falling hammer is the standard in practice today. Standard laboratory compaction tests are described in more detail in Chapter 5. In these tests, soil is mixed with water at a range of moisture contents w and compacted using a specified compaction energy (e.g., ft-lbs/ft^3 or joules/m^3). Figure 7-11 illustrates the effect of compaction energy on laboratory compaction curves. As described in Chapter 5, the Modified Proctor compaction test (ASTM D1557/ AASHTO T-180) has a compaction energy of 2,700 kN-m/m^3 (56,000 ft-lb/ft^3), which is nearly 5 times the compaction energy of 600 kN-m/m^3 (12,400 ft-lb/ft^3) in the Standard Proctor test (ASTM D698/AASHTO T-99). Likewise, increased compaction energy in the field will increase the maximum dry unit weight and decrease the associated optimum water content.

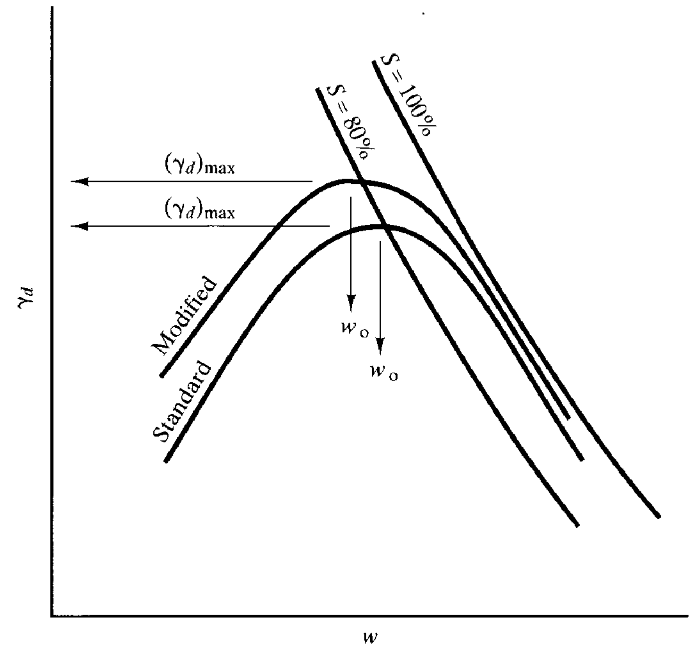


Figure 7-11. Effect of compaction energy on compaction curves (Coduto, 1999).

Different soils will generally have differently shaped compaction curves. This fact will aid in identifying the corresponding laboratory curve for materials encountered in the field. Figure 7-12 shows typical compaction curves for several different soils. Coarser, granular soils typically have fairly steep compaction curves, with large changes in density for small changes in moisture content, while highly plastic clays exhibit fairly flat compaction curves. The maximum dry density is higher for coarser soils and the optimum moisture content is lower. Some cohesionless soils will also exhibit two peaks in the compaction curve; one at very dry conditions, where there are no capillary tensions to resist the compaction effort, and the other at the optimum moisture content, where optimum lubrication between particles occurs.

Nearly all compaction specifications are based on achieving a minimum dry unit weight in the field. This is usually expressed in terms of the relative compaction C_R:

$$C_R = \frac{\gamma_d}{(\gamma_d)_{max}} \times 100\% \qquad \text{Eq. 7-5}$$

in which γ_d is the dry unit weight achieved in the field and $(\gamma_d)_{max}$ is the maximum dry unit weight as determined from a specified laboratory compaction test.

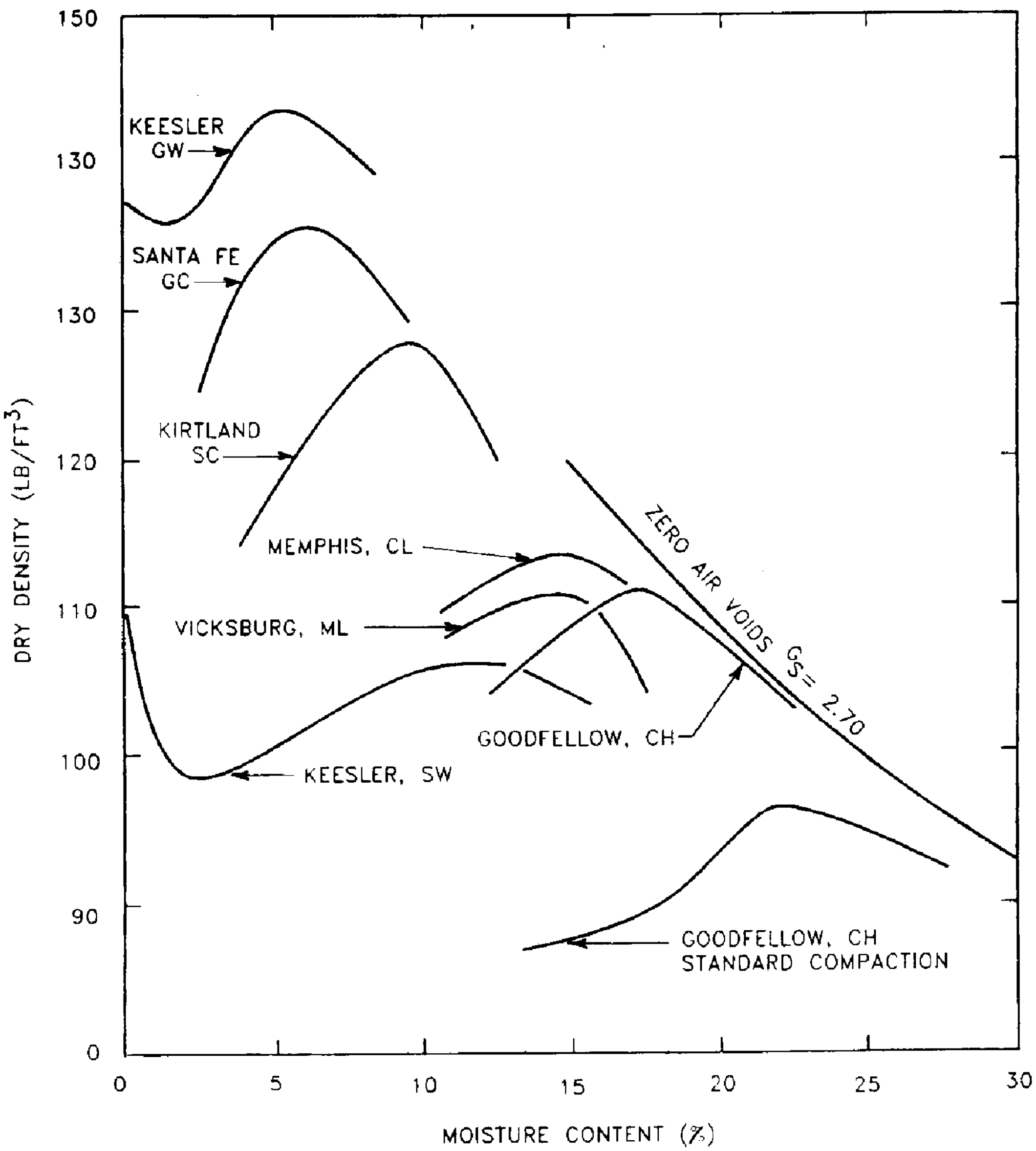


Figure 7-12. Laboratory compaction curves for different soils (Rollings and Rollings, 1996).

The water content at compaction is also sometimes specified because of its effect on soil fabric, especially for clays. Clays compacted dry of optimum have a flocculated fabric (see Figure 7-13), which generally corresponds to higher permeability, greater strength and stiffness, and increased brittleness. Conversely, clays compacted wet of optimum to the same equivalent dry density tend to have a more oriented or dispersed fabric, which typically corresponds to lower permeability, lower strength and stiffness, but more ductility.

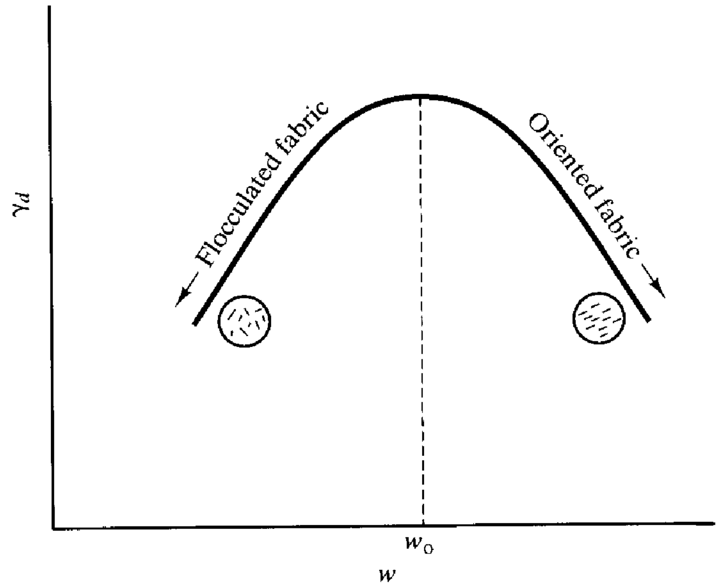


Figure 7-13. Effect of compacted water content on soil fabric for clays (Coduto, 1999).

7.4.2 Effect on Soil Properties

The principal effects of compaction on soil properties are as follows:

- *Density:* As described in the preceding sections, the most direct measurable effect of compaction is an increase in soil density. Typical laboratory values of maximum dry density values and optimum moisture contents for different soils were summarized in Chapter 5, Table 5-18 and 5-19.
- *Strength:* Intuitively, one expects strength to increase with compaction energy and to be larger at low water contents than at high values. Figure 7-14 summarizes typical strength versus water content and compaction energy for a lean clay where strength is quantified by CBR (Rollings and Rollings, 1996). The data in the figure generally confirm intuitive expectations. The strength dry of the optimum water content is larger for higher compaction energies, as expected, and is up to an order of magnitude higher than the strength when compacted wet of optimum. Note, however, that higher compaction energies can produce slightly lower strength values when a fine-grained soil is compacted at water contents higher than the optimum. **Also note that the strength in the figure is based on unsaturated soils. If material compacted dry of optimum becomes saturated, a significant decrease in strength can occur, with strengths even less than that of the same soil compacted wet of optimum**. Large changes in strength upon wetting are associated with fine-grained silts and clays, and are less pronounced or even negligible in coarse-grained soils (Rollings and Rollings, 1996).

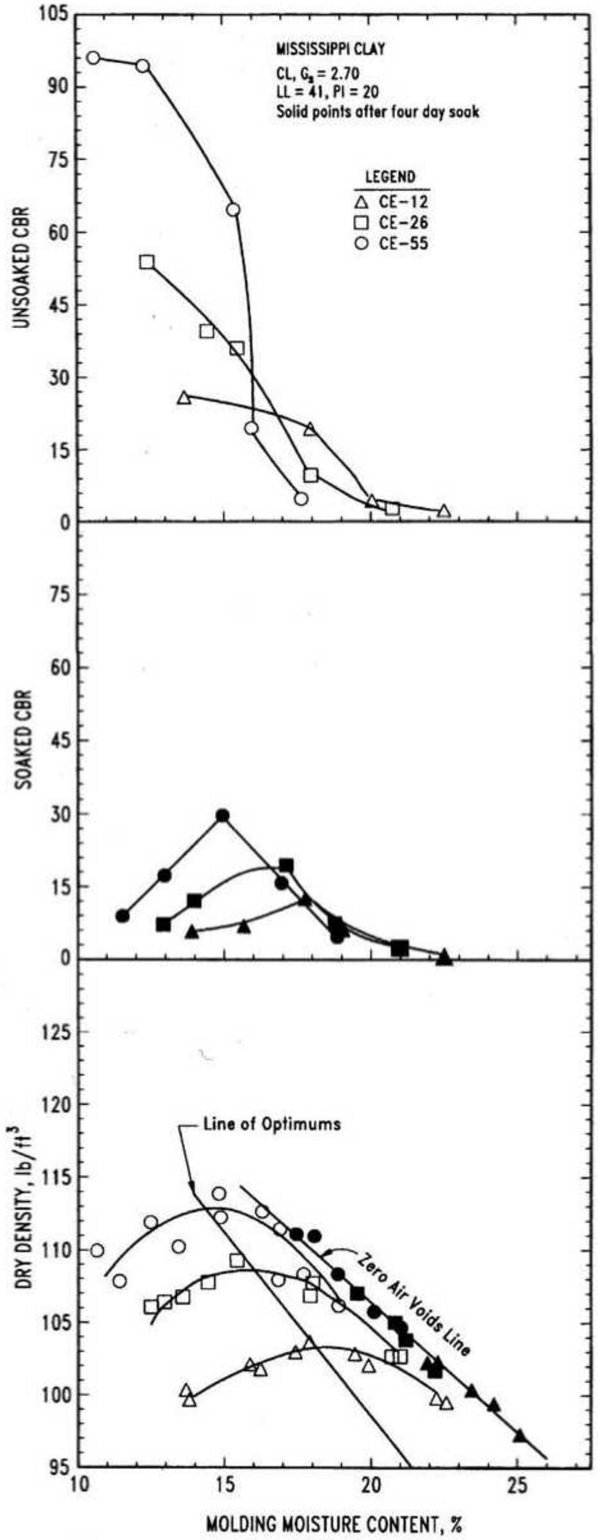


Figure 7-14. Strength as measured by CBR and dry density vs. water content for laboratory impact compaction (Rollings and Rollings, 1996).

- *Stiffness:* Figure 7-15 summarizes typical stiffness versus water content and compaction energy behavior for clays, where stiffness is defined as the stress required to case 5% and 25% axial strain in a triaxial compression test (Seed and Chan, 1959). Stiffness increases with compaction energy when compacted dry of optimum and is largely independent of compaction energy when compacted wet of optimum. The stiffness dry of optimum is also substantially larger than when compacted wet of optimum, as would be expected. **Again however, a significant decrease in stiffness can occur if the material becomes saturated to the extent that the stiffness could be less than that of the soil compacted wet of optimum.**
- *Permeability:* Permeability at constant compactive effort decreases with increasing water content and reaches a minimum at about the optimum moisture content. The permeability when compacted dry of optimum is about an order of magnitude higher than the value when compacted wet of optimum.
- *Swelling/Shrinkage Potential:* Swelling of compacted clays is greater when compacted dry of optimum. Dry clays have a greater capacity to absorb water, and thus swell more. Soils dry of optimum are in general more sensitive to environmental influences, such as changes in water content. The situation is just the opposite for shrinkage (Figure 7-16), where samples compacted wet of optimum exhibit the highest shrinkage strains as water is removed from the soil.

7.5 SUBGRADE CONDITIONS REQUIRING SPECIAL DESIGN ATTENTION

Considering variables such as soil type or mineralogy along a length of roadway, the geology (soil genesis and deposition method) and groundwater and flow properties make each project unique with respect to subgrade conditions. It is not surprising that certain conditions will exist that are not conducive to support, or even construction, of pavement systems. This section provides an overview of subgrade conditions that require special design attention. These subsurface conditions are often regional in nature and have usually been identified as problematic by the agency. Several foundation problems, such as collapsible or highly compressible soils, expansive or swelling soils, subsurface water and saturated soils, and frost-susceptible soils, occur extensively across the U.S. and are not specific to one region. For example, frost heave occurs in over half of the states in the U.S. and damage may be most severe in the central states, where many more frost cycles occur than in the most-northern states. Identification of these widely variable problematic subgrade conditions are also reviewed in this section, along with design and construction alternatives to achieve an adequate foundation on which to build the pavement structure.

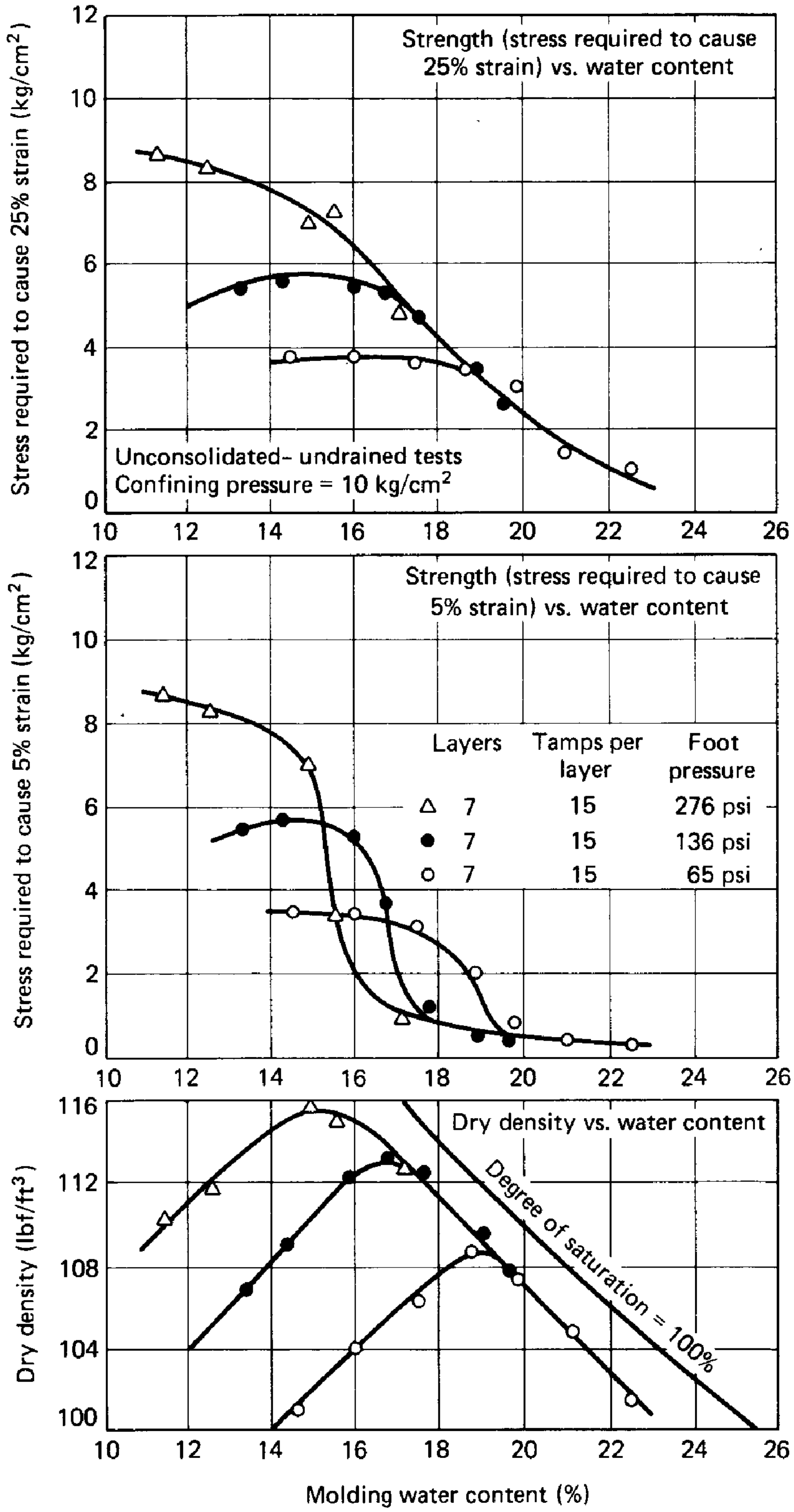


Figure 7-15. Stiffness as a function of compactive effort and water content (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

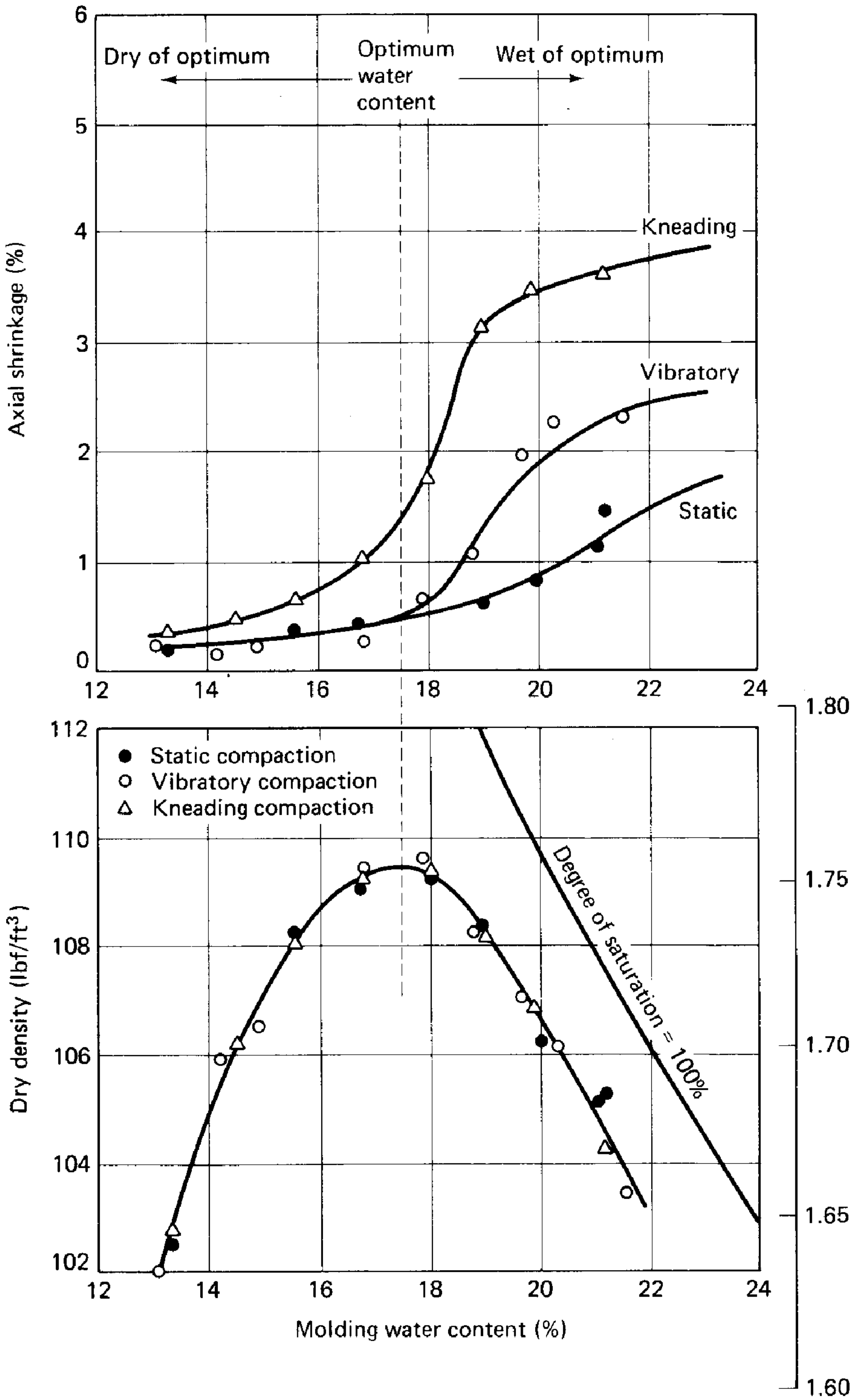


Figure 7-16. Shrinkage as a function of water content and type of compaction (after Seed and Chan, 1959; from Holtz and Kovacs, 1981).

Most of the subgrade conditions presented in this section can be anticipated through a complete exploration program, as described in Chapter 4, and mitigated or at least minimized via well-conceived designs. By identifying such subgrade issues in the design stage, or even the potential for such problems along an alignment, alternative designs can be established. Alternate designs can then be placed in the bid documents with indicators clearly identified that show where these alternatives should be considered, and then implemented if and where such conditions are encountered. When these special subgrade conditions are not recognized in design, they are often identified during construction, usually resulting in claims and overruns. However, identifying problems in construction is still somewhat fortunate, considering the impact such problems may have on the pavement performance. If the soil conditions described in this section go undetected, there typically is decreased serviceability, usually resulting in premature localized rehabilitation or, not uncommon, reconstruction of the pavement within the first few years of the pavement performance period.

7.5.1 Problematic Soil Types

Obviously, a pavement is to be constructed on whatever material and condition is naturally occurring. The strength and stability of some soils can present problems during construction and certainly can affect the long-term performance of the pavement during its service life. In order to properly discuss these potential problems, it is necessary to define some terms as they relate to problematic mineralogy (Sowers, 1979). Some of the terms are true geological terminology, while some are local or regional terminology. The terms may describe a particular material or condition, but all are problematic and care must be taken when constructing pavements in regions containing these materials.

Adobe. Sandy clays of medium plasticity found in the semiarid regions of the southwestern U.S. These soils have been used for centuries to make sun-dried brick. The name is also applied to some highly plastic clays of the West, which swell significantly when wet.

Bentonite. Highly plastic clay, usually montmorillonite, resulting from the decomposition of volcanic ash. It may be hard when dry, but swells considerably when wet.

Buckshot clay. Applied to clays of the southern and southwestern United States. Cracks into small, hard, relatively uniform sized lumps on drying. Dry lumps will degrade upon wetting (*e.g.,* after they have been used as fill). These soils also tend to swell when wet.

Caliche. A silt or sand of the semiarid areas of the southwestern United States that is cemented with calcium carbonate. The calcium carbonate is deposited by the evaporation of water brought to the ground surface by capillary action. The consistency of caliche varies from soft rock to firm soil.

Coquina. A soft, porous limestone made up largely of shells, coral, and fossils cemented together. Very friable, and breaks down during construction.

Gumbo. A fine-grained, highly plastic clay of the Mississippi Valley. It has a sticky, greasy feel, highly expansive, and forms large shrinkage cracks on drying.

Kaolin. A white or pink clay of low plasticity. It is composed largely of minerals of the kaolinite family.

Loam. A surface soil that may be described as a sandy silt of low plasticity or a silty sand that is well suited to tilling. It applies to soils within the uppermost horizons and should not be used to describe deep deposits of parent material. Loam-type soils are typically sensitive to moisture, easily disturbed in construction, and frost susceptible.

Loess. A deposit of relatively uniform, windblown silt. It has a loose structure, with numerous rootholes that produce vertical cleavage and high vertical permeability. It consists of angular to subrounded quartz and feldspar particles cemented with calcium carbonate or iron oxide. Upon saturation, it becomes soft and compressible because of the loss of cementing. Loess altered by weathering in a humid climate often becomes more dense and somewhat plastic (*loess loam*). Loess is also highly frost susceptible.

Marine clay. Clays deposited in a marine environment, which, if later uplifted, tend to be extra sensitive due to salt leaching, dramatically losing strength when disturbed.

Marl. A water-deposited sand, silt, or clay containing calcium carbonate. Marls are often light to dark gray or greenish in color and sometimes contain colloidal organic matter. They are often indurated into soft rock.

Muck or mud. An extremely soft, slimy silt or organic silt found on river and lake bottoms. The terms indicate an extremely soft consistency rather than any particular type of soil. Muck implies organic matter.

Peat. A naturally occuring highly organic substance derived primarily from plant materials (ASTM D 5715). Peats are dark brown or black, loose (void ratio may be 5 to 10), and extremely compressible. When dried, they will float. Peat bogs often emit quantities of inflammable methane gas. These soils will experience significant short-term and long-term settlement, even under light loads, and are often moisture sensitive, losing significant strength when wet. They are easily disturbed under construction activities. Peat containing a high degree of easily identifiable fibers is often called *fibrous peat* for geotechnical applications. Peat containing highly decomposed fibers and a significant highly organic soil component is often called *amorphous peat*.

Quicksand. Refers to a condition, not a soil type. Gravels, sands, and silts become "quick" when an upward flow of groundwater and/or gas takes place to such a degree that the particles are lifted.

Saprolites. Soils developed from in-situ weathering of rocks. Relic joints from the parent rock often control the weathered soils' strength, permeability, and stability. Fragments may appear sound, but prove to be weak. Identifying the transition of soil to weathered rock to sound rock is difficult, often resulting in claims.

Shale. Indurated, fine grained, sedimentary rocks, such as mudstones, siltstone, and claystone, which are highly variable and troublesome. Some are hard and stable, while others are soft and degrade into clay soon after exposure to the atmosphere or during the design life of the structure. Clays developed from shale are often highly plastic.

Sulfate. A mineral compound characterized by the sulfate radical SO_4, which may be contained in soil. It creates significant expansion problems in lime-stabilized soil and, in some cases, distress in concrete.

Sulfide. A mineral compound characterized by the linkage of sulfur with a metal, such as lead or iron, creating galena and pyrite, respectively.

Till. A mixture of sand, gravel, silt, and clay produced by the plowing action of glaciers. The name boulder clay is often given such soils, particularly in Canada and England. The characteristics of glacial till vary depending on the sediments and bedrock eroded. The tills in New England are typically coarser and less plastic that those from the Midwest. The tills in the Northeast tend to be broadly graded and often unstable under water action. The complex nature of their deposition creates a highly unpredictable material.

Topsoils. Surface soils that support plant life. They usually contain considerable organic matter. These soils tend to settle over time as organic matter continues to degrade. They are often moisture sensitive, losing significant strength when wet, and are easily disturbed under construction activities.

Tuff. The name applied to deposits of volcanic ash. In humid climates or in areas in which ash falls into bodies of water, the tuff becomes cemented into a soft, porous rock.

Varved clays. Sedimentary deposits consisting of alternate thin layers of silt and clay. Ordinarily, each pair of silt and clay layers is from 3 – 13 mm (1/8 – 1/2 in.) thick. They are the result of deposition in lakes during periods of alternating high and low water in the inflowing streams, and are often formed in glacial lakes. These deposits have a much higher horizontal than vertical permeability, with the horizontal seams holding water. They are often sensitive, and will lose strength when remolded.

7.5.2 Compressible Soils

Effect of Compressible Soils on Pavement Performance

Highly compressible (very weak) soils are susceptible to large settlements and deformations with time that can have a detrimental effect on pavement performance. Highly compressible soils are very low density, saturated soils, usually silts, clays, and organic alluvium or wind blown deposits and peats. If these compressible soils are not treated properly, large surface depressions with random cracking can develop. The surface depressions can allow water to pond on the pavement's surface and more readily infiltrate the pavement structure, compounding a severe problem. More importantly, the ponding of water will create a safety hazard to the traveling public during wet weather.

Treatments for Compressible Soils

The selection of a particular technique depends on the depth of the weak soil, and the difference between the in-situ conditions and the minimum compaction or strength requirements to limit the amount of anticipated settlement to a permissible value that will not adversely affect pavement performance. When constructing roadways in areas with deep deposits of highly compressible layers, the specific soil properties must be examined to calculate the estimated settlement. Under these conditions, a geotechnical investigation and detailed settlement analysis must be completed prior to the pavement design. When existing subgrade soils do not meet minimum compaction requirements and are susceptible to large settlements over time, consider the following alternatives:

- Remove and process soil to attain the approximate optimum moisture content, and replace and compact.

- Remove and replace subgrade soil with suitable borrow or select embankment materials. All granular fill materials should be compacted to at least 95% of the maximum density, with moisture control, as defined by AASHTO T180. Cohesive fill materials should be compacted to no less than 90%, near or slightly greater than optimum moisture content (*e.g.,* -1% to +2% of optimum), as defined by AASHTO T99.
- Consider mechanical stabilization using geosynthetics as covered in Section 7.5 to reduce the amount of undercut required.
- If soils are granular (*e.g.,* sands and some silts), consider compaction of the soils from the surface to increase the dry density through dynamic compaction techniques. Identification of soil characteristics and detailed procedures for the successful implementation of this technique covered in FHWA/NHI course 132034 on *Ground Improvement Techniques* (FHWA NHI-04-001).
- If the soil is extremely wet or saturated, consider dewatering using well points or deep horizontal drains. If horizontal drains cannot be daylighted, connection to storm drainage pipes or sump pumps may be required.
- Consolidate deep deposits of very weak saturated soils with large fills prior to pavement construction (surcharge). After construction, the fills can either be left in-place or removed, depending on the final elevation. Consider wick drains to accelerate consolidation (see FHWA NHI-04-001).
- Other techniques for deep deposits of compressible soil include piled embankments and use of lightweight fill, such as geofoam, as covered in the FHWA *Ground Improvement Techniques* manual (FHWA NHI-04-001). Although more costly than most of the previous techniques in terms of construction dollars, these techniques offer immediate improvement, thus accelerating construction. On some projects, the time savings may be more valuable than the construction cost differential.

7.5.3 Collapsible Soils

As with highly compressible soils, collapsible soils can lead to significant localized subsidence of the pavement. Collapsible soils are very low density silt type soils, usually alluvium or wind blown (loess) deposits, and are susceptible to sudden decreases in volume when wetted. Often their unstable structure has been cemented by clay binders or other deposits, which will dissolve on saturation, allowing a dramatic decrease in volume (Rollings and Rollings, 1996). Native subgrades of collapsible soils should be soaked with water prior to construction and rolled with heavy compaction equipment. In some cases, residual soils may also be collapsible due to leaching of colloidal and soluble materials. Figure 7-17 provides a method of identifying the potential for collapsible soils. Other local methods for identification may be available. Collapsible soils can also be created in fills when sand type soils are compacted on the dry side of optimum moisture. Meniscus forces between particles can create a soil fabric susceptible to collapse.

If pavement systems are to be constructed over collapsible soils, special remedial measures may be required to prevent large-scale cracking and differential settlement. To avoid problems, collapse must be induced prior to construction. Methods include

1. ponding water over the region of collapsible soils.
2. infiltration wells.
3. compaction - conventional with heavy vibratory roller for shallow depths (within 0.3 or 0.6 m (1 or 2 ft))
4. compaction - dynamic or vibratory for deeper deposits of more than half a meter (a few feet) (could be combined with inundation)
5. excavated and replaced.

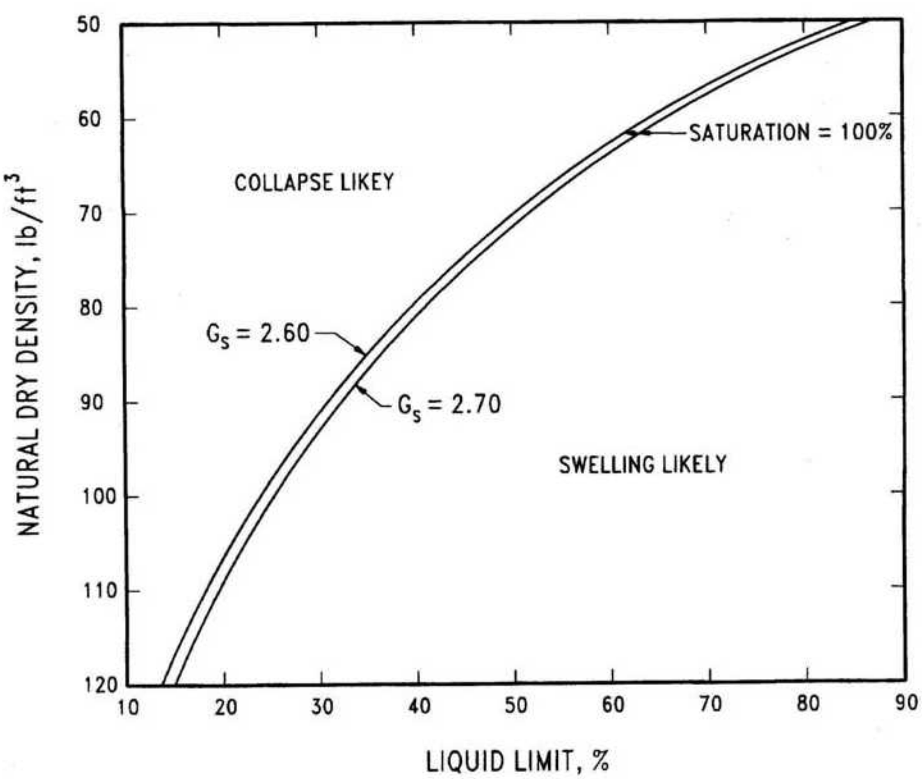


Figure 7-17. Guide to collapsible soil behavior (Rollings and Rollings, 1996).

7.5.4 Swelling Soils

Effect of Swelling Soils on Pavement Performance

Swelling or expansive soils are susceptible to volume change (shrink and swell) with seasonal fluctuations in moisture content. The magnitude of this volume change is dependent on the type of soil (shrink-swell potential) and its change in moisture content. A loss of moisture will cause the soil to shrink, while an increase in moisture will cause it to expand or swell. This volume change of clay type soils can result in longitudinal cracks near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length.

Expansive soils are a very significant problem in many parts of the United States (see Figure 7-18) and are responsible for the application of premature maintenance and rehabilitation activities on many miles of roadway each year. Expansive soils are especially a problem when deep cuts are made in a dense (over-consolidated) clay soil.

Identification of Swelling Soils

Various techniques and procedures exist for identifying potentially expansive soils. AASHTO T 258 can be used to identify soils and conditions that are susceptible to swell. Two of the more commonly used documents are listed below:

- *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils*, Report No. FHWA-RD-77-94, Federal Highway Administration, Washington, D.C., June 1977.
- *Design and Construction of Airport Pavements on Expansive Soils*, Report No. FAA-RD-76-66, Federal Aviation Administration, U.S. Department of Transportation, Washington, D.C., June 1976.

Clay mineralogy and the availability of water are the key factors in determining the degree to which a swelling problem may exist at a given site. Different clay minerals exhibit greater or lesser degrees of swell potential based on their specific chemistry. Montmorillonitic clays tend to exhibit very high swell potentials due to the particle chemistry, whereas illitic clays tend to exhibit very low swell potentials. Identification of clay minerals through chemical or microscopic means may be used as a method of identifying the presence of high swell potential in soils. The soil fabric will also influence the swell potential, as aggregated particles will tend to exhibit higher swell than dispersed particles, and flocculated higher than deflocculated. Generally, the finer-grained and more plastic the soil, the higher the swell potential the soil will exhibit.

The identification of swelling soils in the subgrade is a key component of the geotechnical investigation for the roadway. Soils at shallow depths beneath the proposed pavement elevation are generally sampled as part of the investigation, and their swell potential may be identified in a number of ways. Index testing is a common method for identifying swell potential. Laboratory testing to obtain the plastic and liquid limits and/or the shrinkage limit will usually be conducted. The soil activity (ASTM D 4318), defined as the ratio of the plasticity index to the percentage of the soil by weight finer than 0.002 mm (0.08 mils) is also used as an index property for swell potential, since clay minerals of higher activity exhibit higher swell. Activity calculation requires measurement of gradation using hydrometer methods, which is not typical in geotechnical investigations for pavement design in many states. In addition to index testing, agency practice in regions where swelling soils are a common problem may include swell testing (e.g., ASTM D 4546), for natural or compacted soil samples. Such testing generally includes measurement of the change in height (or volume) of a sample exposed to light loading similar to that expected in the field and then allowed free access to water.

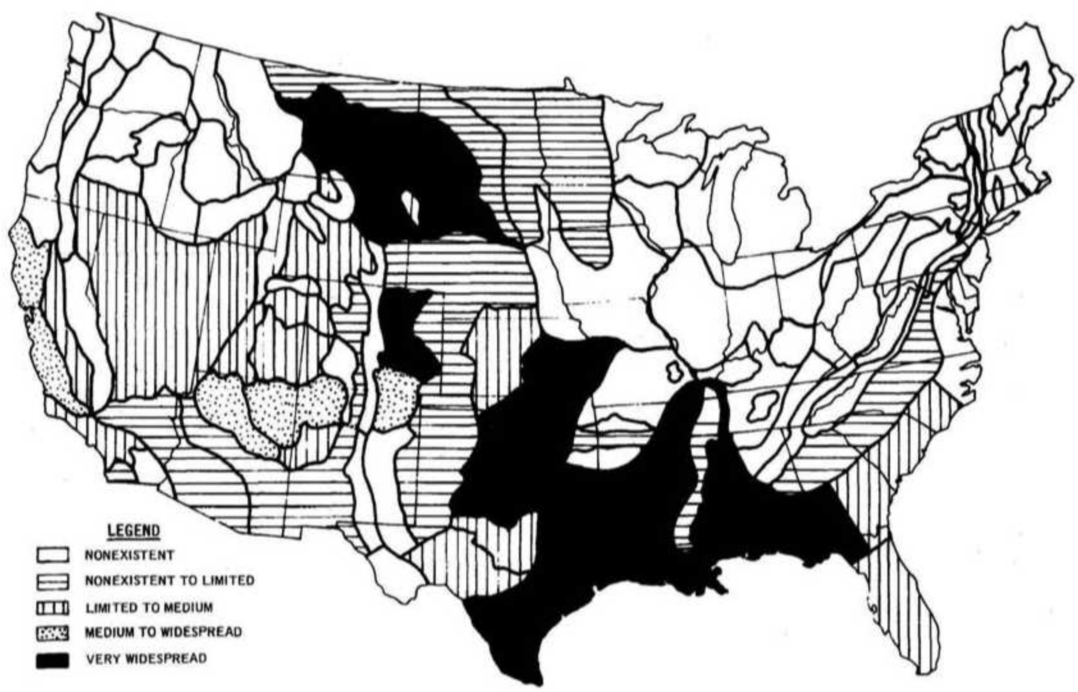


Figure 7-18. Estimated location of swelling soils (from Witczak, 1972).

Treatment for Swelling Soils

When expansive soils are encountered along a project in environments and areas where significant moisture fluctuations in the subgrade are expected, consideration should be given to the following alternatives to minimize future volume change potential of the expansive soil:

- For relatively thin layers of expansive clays near the surface, remove and replace the expansive soil with select borrow materials.
- Extend the width of the subsurface pavement layers to reduce the change (*i.e.,* wetting or drying) in subgrade moisture along the pavement's edge, and increase the roadway crown to reduce infiltration moisture.
- Partial encapsulation along the edge of the pavement or full encapsulation can also be used to reduce change in subgrade moisture, as described in greater detail in Section 7.5.
- Scarify, stabilize, and recompact the upper portion of the expansive clay subgrade. Lime or cement stabilization is an accepted method for controlling the swelling of soils, as discussed in Section 7.6. (*Stabilization*, as used for expansive soils, refers to the treatment of a soil with such agents as bitumen, Portland cement, slaked or hydrated lime, and flyash to limit its volume change characteristics. This can substantially increase the strength of the treated material.)
- In areas with deep cuts in dense, over-consolidated expansive clays, complete the excavation of the subsurface soils to the proper elevation, and allow the subsurface soils to rebound prior to placing the pavement layers.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of swelling soils on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the swelling is anticipated to be relatively uniform, then the procedures do not apply.

7.5.5 Subsurface Water

It is important to identify any saturated soil strata, the depth to groundwater, and subsurface water flow between soil strata. Subsurface water is especially important to recognize and identify in the transition areas between cut and fill segments. If allowed to saturate unbound base/subbase materials and subgrade soils, subsurface water can significantly decrease the strength and stiffness of these materials. Reductions in strength can result in premature surface depressions, rutting, or cracking. Seasonal moisture flow through selected soil strata can also significantly magnify the effects of differential volume change in expansive soils. Cut areas are particularly critical for subsurface water.

Treatments for Subsurface Water

When saturated soils or subsurface water are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

- For saturated soils near the surface, dry or strengthen the wet soils through the use of mechanical stabilization techniques to provide a construction platform for the pavement structure, as described in Section 7.6.
- Remove and replace the saturated soils with select borrow materials or soils. (May not be an option if excavation is required below the groundwater level).
- Place and properly compact thick fills or embankments to increase the elevation of the subgrade, or in other words, increase the thickness between the saturated soils or water table depth and pavement structure.
- Consideration should also be given to the use of subgrade drains as previously detailed in Section 7.2 whenever the following conditions exist:
    - High ground-water levels that may reduce subgrade stability and provide a source of water for frost action.
    - Subgrade soils consisting of silts and very fine sands that may become quick or spongy when saturated.
    - Water seeps from underlying water-bearing strata or from subgrades in cut areas (consider intercepting drains).

7.5.6 Frost-Susceptible Soils

Effect of Frost Action on Pavement Performance

Frost action can cause differential heaving, surface roughness and cracking, blocked drainage, and a reduction in bearing capacity during thaw periods. These effects range from slight to severe, depending on types and uniformity of subsoil, regional climatic conditions (*i.e.,* depth of freeze), and the availability of water.

One effect of frost action on pavements is frost heaving caused by crystallization of ice lenses in voids of soils containing fine particles. As shown in Figure 7-19, three conditions must be present to cause frost heaving and associated frost action problems:

- frost-susceptible soils;
- subfreezing temperatures in the soil; and,
- source of water.

If these conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur, causing surface irregularities, roughness, and ultimately cracking of the pavement surface.

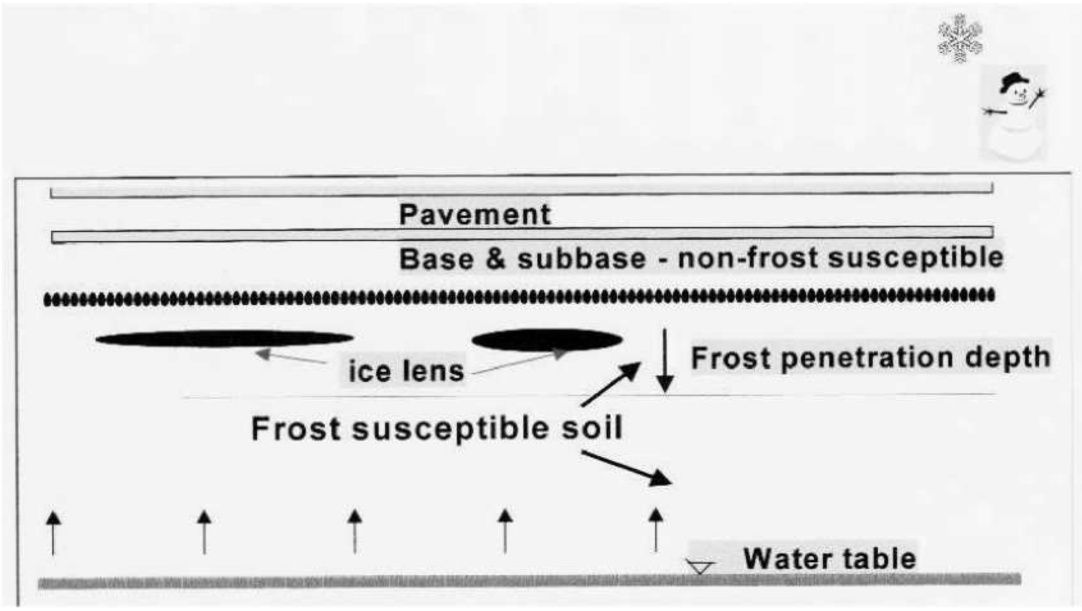


Figure 7-19. Elements of frost heave.

A second effect of frost action is thaw weakening. The bearing capacity may be reduced substantially during mid-winter thawing periods, and subsequent frost heaving is usually more severe because water is more readily available to the freezing zone. In more-southerly areas of the frost zone, several cycles of freeze and thaw may occur during a winter season and cause more damage than one longer period of freezing in more-northerly areas. Spring thaws normally produce a loss of bearing capacity to well below summer and fall values, followed by a gradual recovery over a period of weeks or months. Water is also often trapped above frozen soil during the thaw, which occurs from the top down, creating the potential for long-term saturated conditions in pavement layers.

Identification of Frost-Susceptible Soils

Frost-susceptible soils have been classified into four general groups. Table 7-12 provides a summary of the typical soils in each of these four groups based on the amount of fines (material passing the 0.075 mm (No. 200) sieve. Figure 7-20 graphically displays the expected average rate of frost heave for the different soil groups based on portion of soil finer than 0.02 mm (0.8 mils).

Little to no frost action occurs in clean, free draining sands, gravels, crushed rock, and similar granular materials, under normal freezing conditions. The large void space permits water to freeze in-place without segregation into ice lenses. Conversely, silts are highly frost-susceptible. The condition of relatively small voids, high capillary potential/action, and relatively good permeability of these soils accounts for this characteristic.

Table 7-12. Frost susceptibility classification of soils (NCHRP 1-37A).

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer than 0.075 mm (# 200) by wt.	Typical Soil Classification
F1	Negligible to low	Gravelly soils	3-10	GC, GP, GC-GM, GP-GM
F2	Low to medium	Gravelly soils	10-20	GM, GC-GM, GP-GM
		Sands	3-15	SW, SP, SM, SW-SM, SP-SM
F3	High	Gravelly Soils	Greater than 20	GM-GC
		Sands, except very fine silty sands	Greater than 15	SM, SC
		Clays PI>12	—	CL, CH
F4	Very high	All Silts	—	ML-MH
		Very Fine Silty Sands	Greater than 15	SM
		Clays PI<12	—	CL, CL-ML
		Varied clays and other fine grained, banded sediments	—	CL, ML, SM, CH

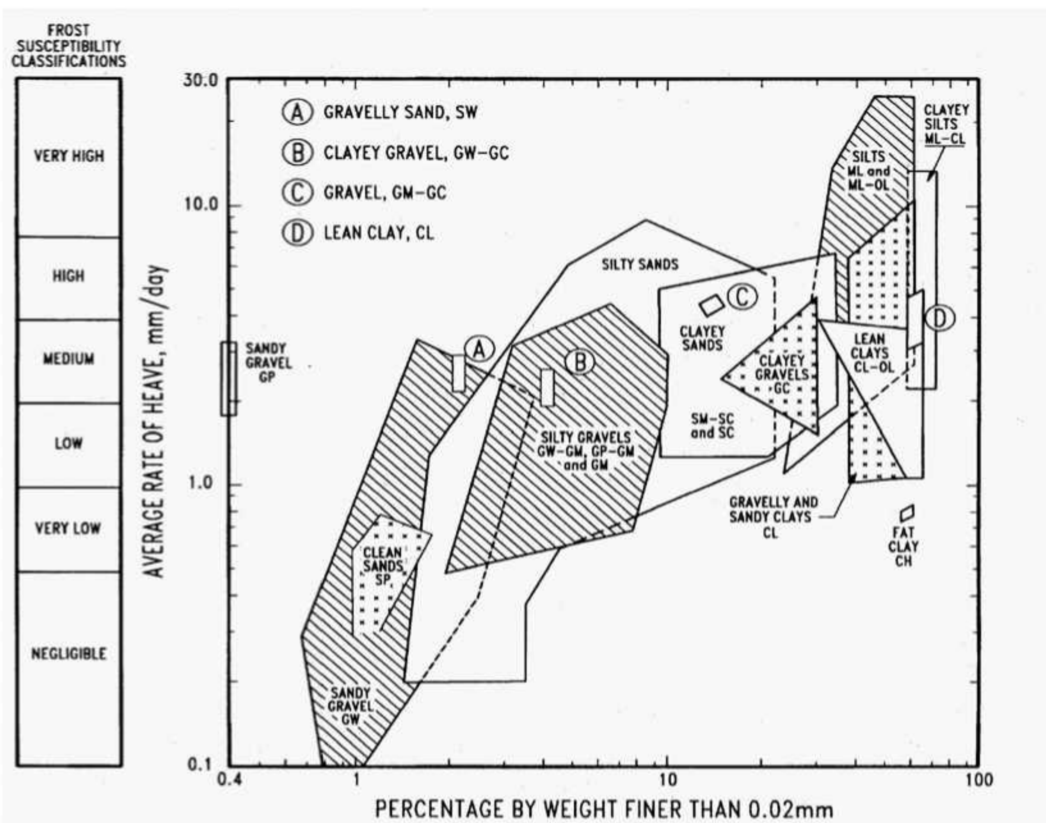


Figure 7-20. Average rate of heave versus % fines for natural soil gradations (Kaplar, 1974).

Clays are cohesive and, although their potential capillary action is high, their capillary rate is low. Although frost heaving can occur in clay soils, it is not as severe as for silts, since the impervious nature of the clays makes passage of water slow. The supporting capacity of clays must be reduced greatly during thaws, even in the absence of significant heave. Thawing usually takes place from the top downward, leading to very high moisture contents in the upper strata.

A groundwater level within 1.5 m (5 ft) of the proposed subgrade elevation is an indication that sufficient water will exist for ice formation. Homogeneous clay subgrade soils also contain sufficient moisture for ice formation, even with depth to groundwater in excess of 3 m (10 ft). However, the magnitude of influence will be highly dependent on the depth of the freezing front (*i.e.,* frost depth penetration). For deep frost penetration, groundwater at even a greater depth could have an influence on heave.

Identification of Frost-Susceptible Conditions

The most distinguishing factor for identifying a pavement frost hazard condition is water supply. For frost susceptible soils within the frost zone, the frost hazard may be rated as high or low, according to the following conditions. An unknown rating may be appropriate when conditions for both high and low ratings occur and cannot be resolved, or when little or no information is available. The inclusion of a frost hazard rating in the site evaluation documentation verifies that an evaluation of frost action has been attempted and has not been overlooked. When the rating is unknown, a decision to include frost action mitigation measures in a design will be based more upon the unacceptable nature of frost damage than the probability of occurrence.

The conditions associated with a high frost hazard potential include

1. A water table within 3 m (10 ft) of the pavement surface (depth of influence depends on the type of soil and frost depth).
2. Observed frost heaves in the area.
3. Inorganic soils containing more than 3% (by weight) or more grains finer than 0.02 mm (0.8 mils) in diameter according to the U.S. Army Corps of Engineers.
4. A potential for the ponding of surface water and the occurrence of soils between the frost zone beneath the pavement and the surface water with permeabilities high enough to enable seepage to saturate soils within the frost zone during the term of ponding.

The conditions associated with a low frost hazard potential include

1. A water table greater than 6 m (20 ft) below the pavement surface (again, could be much shallower depending on the type of soil and frost depth).

2. Natural moisture content in the frost zone low versus the saturation level.
3. Seepage barriers between the water supply and the frost zone.
4. Existing pavements or sidewalks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have not experienced frost damage.
5. Pavements on embankments with surfaces more than 1 – 2 m (3 – 6 ft) above the adjacent grades (provides some insulation and a weighting action to resist heave).

Treatment for Frost Action

When frost-susceptible soils are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

1. Remove the frost-susceptible soil (generally for groups F3 and F4, Table 7-12) and replace with select non-frost susceptible borrow to the expected frost depth penetration.
2. Place and compact select non-frost-susceptible borrow materials to a thickness or depth to prevent subgrade freezing for frost susceptible soil groups F2, F3, and F4, Table 7-12.
3. Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions.
4. Stabilize the frost-susceptible soil by eliminating the effects of soil fines by three processes: a) mechanically removing or immobilizing by means of physical-chemical means, such as cementitious bonding, b) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages, or c) altering the freezing point of the soil moisture.
    a. Cementing agents, such as Portland cement, bitumen, lime, and lime-flyash, as covered in Section 7.5. These agents effectively remove individual soil particles by bonding them together, and also act to partially remove capillary passages, thereby reducing the potential for moisture movement. Care must be taken when using lime and lime-flyash mixtures with clay soils in seasonal frost areas (see Section 7.5 & Appendix F).
    b. Soil moisture available for frost heave can be mitigated through the installation of deep drains and/or a capillary barrier such that the water table is maintained at a sufficient depth to prevent moisture rise in the freezing zone. Capillary barriers can consist of either an open graded gravel layer sandwiched between two geotextiles, or a horizontal geocomposite drain. The installation of a capillary barrier requires the removal of the frost susceptible material to a depth either below frost penetration or sufficiently significant to reduce the influence of frost heave on the pavement. The capillary break must

be drained. The frost susceptible soil can then be replaced and compacted above the capillary barrier to the required subgrade elevation.

5. Increase the pavement structural layer thickness to account for strength reduction in the subgrade during the spring-thaw period for frost-susceptible groups F1, F2, and F3.

Pavement design for frost action often determines the required overall thickness of flexible pavements and the need for additional select material beneath both rigid and flexible pavements. Three design approaches have been used for pavement in seasonal frost areas:

- The Complete Protection approach—requires non-frost susceptible materials for the entire depth of frost (*e.g.,* treatment methods 1, 2, and 3 above).

- Limited Subgrade Frost Penetration approach—permits some frost penetration into the subgrade, but not enough to allow unacceptable surface roughness to develop.

- Reduced Subgrade Strength approach—allows more frost penetration into the subgrade, but provides adequate strength during thaw weakened periods.

AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of frost heave on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the frost is anticipated to be relatively uniform, then the procedures do not apply.

For the most part, local frost-resistant design approaches have been developed from experience, rather than by application of some rigorous theoretical computational method. A more rigorous method is available in the NCHRP 1-37A design procedure to reduce the effects of seasonal freezing and thawing to acceptable limits, as discussed in Chapter 6. The Enhanced Integrated Climatic Model is used to determine the maximum frost depth for the pavement system at a particular location. Various combinations of layer thicknesses and material types can be evaluated in terms of their impact on the maximum frost depth and total amount of base and select materials necessary to protect the frost susceptible soils from freezing.

7.5.7 Summary

Problematic soils can be treated using a variety of methods or a combination thereof. Improvement techniques that can be used to improve the strength and reduce the climatic variation of the foundation on pavement performance include

1. Improvement of subsurface drainage (see Section 7.2, and should always be considered).
2. Removal and replacement with better materials (*e.g.,* thick granular layers).
3. Mechanical stabilization using thick granular layers.
4. Mechanical stabilization of weak soils with geosynthetics (geotextiles and geogrids) in conjunction with granular layers.
5. Lightweight fill.
6. Stabilization of weak soils with admixtures (highly plastic or compressible soils).
7. Soil encapsulation.

Details for most of these stabilization methods will be reviewed in the next section.

7.6 SUBGRADE IMPROVEMENT AND STRENGTHENING

Proper treatment of problem soil conditions and the preparation of the foundation are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. Some agencies have recognized certain materials simply do not perform well, and prefer to remove and replace such soils (*e.g.,* a state specification dictating that frost susceptible loess cannot be present in the frost penetration zone). However, in many cases, this is not the most economical or even desirable treatment (*e.g.,* excavation may create disturbance, plus additional problems of removal and disposal). Stabilization provides an alternate method to improve the structural support of the foundation for many of the subgrade conditions presented in the previous section. In all cases, the provision for a uniform soil relative to textural classification, moisture, and density in the upper portion of the subgrade cannot be over-emphasized. This uniformity can be achieved through soil sub-cutting or other stabilization techniques. Stabilization may also be used to improve soil workability, provide a weather resistant work platform, reduce swelling of expansive materials, and mitigate problems associated with frost heave. In this section, alternate stabilization methods will be reviewed, and guidance will be presented for the selection of the most appropriate method.

7.6.1 Objectives of Soil Stabilization

Soils that are highly susceptible to volume and strength changes can cause severe roughness and accelerate the deterioration of the pavement structure in the form of increased cracking and decreased ride quality when combined with truck traffic. Generally, the stiffness (in terms of resilient modulus) of some soils is highly dependent on moisture and stress state (see Section 5.4). In some cases, the subgrade soil can be treated with various materials to improve the strength and stiffness characteristics of the soil. Stabilization of soils is usually performed for three reasons:

1. As a construction platform to dry very wet soils and facilitate compaction of the upper layers—for this case, the *stabilized* soil is usually not considered as a structural layer in the pavement design process.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil—for this case, the *modified* soil is usually given some structural value or credit in the pavement design process.
3. To reduce moisture susceptibility of fine grain soils.

A summary of the stabilization methods most commonly used in pavements, the types of soils for which they are most appropriate, and their intended effects on soil properties is presented in Table 7-13.

Mechanical stabilization using thick gravel layers or granular layers in conjunction with geotextiles or geogrids is an effective technique for improving roadway support over soft, wet subgrades. Thick granular layers provide a working platform, but do not provide strengthening of the subgrade. In fact, construction of thick granular layers in some cases results in disturbance of the subgrade due to required construction activities. Thick granular layers are also used to avoid or reduce frost problems by providing a protection to the underlying subgrade layers.

Table 7-13. Stabilization Methods for Pavements (after Rollings and Rollings, 1996).

Stabilization Method	Soil Type	Improvement	Remarks
Mechanical			
- More Gravel	Silts and Clays	None	Reduce dynamic stress level
- Blending	Moderately plastic	None	Too difficult to mix
	Other	Improve gradation Reduce plasticity Reduce breakage	
- Geosynthetics	Silts and Clays	Strength gain through minimum disturbance and consolidation	Fast, plus provides long-term separation
- Lightweight fill	Very weak silts, clays, peats	None Thermal barrier for frost protection	Fast, and reduces dynamic stress level
Admixture			
- Portland cement	Plastic		Less pronounced hydration of cement
	Coarse		Hydration of cement
- Lime	Plastic	Drying Strength gain Reduce plasticity Coarsen texture Long-term pozzolanic cementing	Rapid Rapid Rapid Rapid Slow
	Coarse with fines	Same as plastic	Dependent on quantity of plastic fines
	Nonplastic	None	No reactive material
- Lime-flyash	Same as lime	Same as lime	Covers broader range
- Lime-cement- flyash	Same as lime	Same as lime	Covers broader range
- Bituminous	Coarse	Strengthen/bind waterproof	Asphalt cement or liquid asphalt
	Some fines	Same as coarse	Liquid asphalt
	Fine	None	Can't mix
- Pozzolanic and slags	Silts and coarse	Acts as a filler Cementing of grains	Dense and strong Slower than cement
- Chemicals	Plastic	Strength increase and volume stability	See vendor literature Difficult to mix
Water proofers			
- Asphalt	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem
- Geomembranes	Plastic and collapsible	Reduce change in moisture	Long-term moisture migration problem

A common practice in several New England and Northwestern states is to use a meter (3.3 ft) or more of gravel beneath the pavement section. The gravel improves drainage of surface infiltration water and provides a weighting action that reduces and results in more uniform heave. Washington State recently reported the successful use of an 0.4 m (18 in.) layer of cap rock beneath the pavement section in severe frost regions (Ulmeyer et al., 2002).

Blending gravel and, more recently, recycled pavement material with poorer quality soils also can provide a working platform. The gravel acts as filler, creating a dryer condition and decreasing the influence of plasticity. However, if saturation conditions return, the gravel blend can take on the same poorer support characteristics of the subgrade.

Geotextiles and geogrids used in combination with quality aggregate minimize disturbance and allow construction equipment access to sites where the soils are normally too weak to support the initial construction work. They also allow compaction of initial lifts on sites where the use of ordinary compaction equipment is very difficult or even impossible. Geotextiles and geogrids reduce the extent of stress on the subgrade and prevent base aggregate from penetrating into the subgrade, thus reducing the thickness of aggregate required to stabilize the subgrade. Geotextiles also act as a separator to prevent subgrade fines from pumping or otherwise migrating up into the base. Geosynthetics have been found to allow for subgrade strength gain over time. However, the primary long-term benefit is preventing aggregate-subgrade mixing, thus maintaining the thickness of the base and subbase. In turn, rehabilitation of the pavement section should only require maintenance of surface pavement layers.

Stabilization with admixtures, such as lime, cement, and asphalt, have been mixed with subgrade soils used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For admixture stabilization or modification of cohesive soils, hydrated lime is the most widely used. Lime is applicable in clay soils (CH and CL type soils) and in granular soils containing clay binder (GC and SC), while Portland cement is more commonly used in non-plastic soils. Lime reduces the Plasticity Index (PI) and renders a clay soil less sensitive to moisture changes. The use of lime should be considered whenever the PI of the soil is greater than 12. Lime stabilization is used in many areas of the U.S. to obtain a good construction platform in wet weather above highly plastic clays and other fine-grained soils. It is important to note that changing the physical properties of a soil through chemical stabilization can produce a soil that is susceptible to frost heave. Following is a brief description of the characteristics of stabilized soils followed by the treatment procedures. Additional guidance on soil stabilization with admixtures and stabilization with geosynthetics can be obtained from the following resources:

- "Lime Stabilization – Reactions, Properties, Design, and Construction," *State of the Art Report 5*, Transportation Research Board, Washington D.C., 1987.
- *Soil Stabilization for Pavements,* Joint Departments of the Army and Air Force, USA, TM 5-822-14/AFMAN 32-8010, 1994.
- *Geosynthetics Design and Construction Guidelines*, FHWA HI-95-038, 1998.
- *Standard Specifications for Geotextiles* - AASHTO M288, 1997.

7.6.2 Characteristics of Stabilized Soils

Although mechanical stabilization with thick granular layers or geosythetics and aggregate subbase provides the potential for strength improvement of the subgrade over time, this is generally not considered in the design of the pavement section, and no increase in structural support is attributed to the geosynthetic. However, the increase in gravel thickness (minus an allowance for rutting) can contribute to the support of the pavement. Alternatively, the aggregate thickness used in conjunction with the geosynthetic is designed to provide an equivalent subgrade modulus, which can be considered in the pavement design, discounting the additional aggregate thickness of the stabilization layer. Geosynthetics also allow more open graded aggregate, thus providing for the potential to drain the subbase into edgedrains and improving its support value.

The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above (*i.e.,* construction platform, subgrade strengthening, and control of moisture). These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Admixtures used as subgrade stabilizing agents may fill or partially fill the voids between the soil particles. This reduces the permeability of the soil by increasing the tortuosity of the pathways for water to migrate through the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water sensitive soils from the intrusion of surface water. This mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to change the volume change under shear from a contractive to a dilative condition. The admixture type stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by chemical reaction between the soil and stabilizing agent (as in the case of lime or Portland cement). Additional improvement can arise from other chemical-physical reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange). The down side of admixtures is that they require up front lab testing to confirm their performance and very good field control to obtain a uniform, long lasting product, as outlined later in this section. There are also issues of dust control and weather dependency, with some methods that should be carefully considered in the selection of these methods.

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, anticipated traffic loads, the importance of the transportation network, constructability, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone and/or short roadway length is subject to improvement, removal and replacement will usually be the preferred alternative by most agencies, unless a suitable replacement soil is not economically available. Note that in this context, the use of the qualitative term "thin" is intentional, as the thickness of the zone can be described as thick or thin, based primarily on the project economics of the earthwork requirements and the depth of influence for the vehicle loads.

7.6.3 Thick Granular Layers

Many agencies have found that a thick granular layer is an important feature in pavement design and performance. Thick granular layers provide several benefits, including increased load-bearing capacity, frost protection, and improved drainage. While the composition of this layer takes many forms, the underlying strategy of each is to achieve desired pavement performance through improved foundation characteristics. The following sections describe the benefits of thick granular layers, typical characteristics, and considerations for the design and construction of granular embankments.

Objectives of Thick Granular Layers

Thick granular layers have been used in design for structural, drainage, and geometric reasons. Many times, a granular layer is used to provide uniformity and support as a construction platform. In areas with large quantities of readily accessible, good quality aggregates, a thick granular layer may be used as an alternative to soil stabilization. Whatever the reason, thick granular layers aim to improve the natural soil foundation. By doing this, many agencies are recognizing that the proper way to account for weak, poorly draining soils is through foundation improvement, as opposed to increasing the pavement layer thicknesses. The following is a list of objectives and benefits of thick granular layers:

- To increase the supporting capacity of weak, fine-grained subgrades.
- To provide a minimum bearing capacity for the design and construction of pavements.
- To provide uniform subgrade support over sections with highly variable soil conditions.

- To reduce the seasonal effects of moisture and temperature variations on subgrade support.
- To promote surface runoff through geometric design.
- To improve subsurface drainage and the removal of moisture from beneath the pavement layers.
- To increase the elevation of pavements in areas with high water tables.
- To provide frost protection in freezing climatic zones.
- To reduce subgrade rutting potential of flexible pavements.
- To reduce pumping and erosion beneath PCC pavements.
- To meet elevation requirements of geometric design.

Characteristics of Thick Granular Layers

Thick granular layers have been incorporated in pavement design in several ways. They can be referred to as fills or embankments, an improved or prepared subgrade, and select or preferred borrow. Occasionally, a thick granular layer is used as the pavement subbase. The two most important characteristics for all of these layers are material properties and thickness. While geometric requirements (*e.g.,* vertical profile) and improved surface runoff can be achieved by embankments constructed of any soil type, the most beneficial effects are produced through utilization of good quality, granular materials. Several methods are used to characterize the strength and stiffness of granular materials, including the California Bearing Ratio (CBR) and resilient modulus testing. In addition, several types of field plate load tests have been used to determine the composite reaction of the embankment and soil combination. In general, materials with CBR values of 20% or greater are used, corresponding to resilient moduli of approximately 120 MPa (17,500 psi). These are typically sand or granular materials, or coarse-grained materials with limited fines, corresponding to AASHTO A-1 and A-2 (GW, GP, SW and SP) soils.

Aggregate gradation and particle shape are other important properties. Typically, embankment materials are dense-graded, with a maximum top-size aggregate that varies depending on the height of the embankment. Many times, the lowest embankment layer may contain cobbles or aggregates of 100 – 200 mm (4 – 8 in.) in diameter. Granular layers placed close to the embankment surface have gradations, including maximum size aggregates, similar to subbase material specifications. Although dense-graded aggregate layers do not provide efficient drainage relative to open-graded materials, a marginal degree of subsurface seepage can be achieved by limiting the fines content to less than 10%. The type of granular material used is normally a function of material availability and cost. Pit-run gravels and crushed stone materials are the most common. The high shear strength of crushed stone is more desirable than rounded, gravelly materials; however, the use of crushed materials may not always be economically feasible.

The thicknesses of granular layers vary, depending upon their intended use. Granular layers 150 – 300 mm (6 – 12 in.) thick may be used to provide uniformity of support, or act as a construction platform for paving of asphalt and concrete layers. To increase the composite subgrade design values (*i.e.,* combination of granular layer over natural soil), it is usually necessary to place a minimum of 0.5 – 1.5 m (1½ -- 5 ft) of embankment material, depending on the strength of the granular material relative to that of the underlying soil. Likewise, granular fills placed for frost protection may also range from 0.5 – 1.5 m (1½ – 5 ft). In most cases, embankments greater than 2 m (6 ½ ft) thick have diminishing effects in terms of strength, frost protection, and drainage. Granular embankments greater than 2 – 3 m (6½ – 10 ft) thick are usually constructed for purposes of geometric design.

Considerations for Pavement Structural Design

The use of a thick granular layer presents an interesting situation for design. The placement of a granular layer of substantial thickness over a comparatively weak underlying soil forms, essentially, non-homogeneous subgrade in the vertical direction. Pavement design requires a single subgrade design value, for example CBR, resilient modulus, or k-value. This is generally determined through laboratory or field tests, when the soil mass in the zone of influence of vehicle loads is of the same type, or exhibits similar properties. In the case of a non-homogeneous subgrade, the composite reaction of the embankment and soil combination can vary from that of the natural soil to that of the granular layer. Most commonly, the composite reaction is a value somewhere between the two extremes, dependent upon the relative difference in moduli between the soil and embankment, and the thicknesses of the granular layer. The actual composite subgrade response is not known until the embankment layer is placed in the field, and it may be different once the upper pavement layers are placed.

To account for non-homogenous subgrades in pavement structural design, it is recommended to characterize the individual material properties by traditional means, such as resilient modulus or CBR testing, and to compare these results to field tests performed over the constructed embankment layers, as well as the completed pavement section. Analytical models, such as elastic layer programs, can be used to make theoretical predictions of composite subgrade response, and these predictions can then be verified by field testing. Some agencies use in-situ plate load tests to verify that a minimum composite subgrade modulus has been achieved. Deflection devices, including the Falling Weight Deflectometer (FWD), can be used for testing over the compacted embankment layer and over the constructed pavement surface.

It is advisable to use caution when selecting a design subgrade value for a non-homogenous subgrade. Experience has shown that a good-quality embankment layer must be of significant height, say 1 m (3 ft) or more, before the composite subgrade reaction begins to resemble that of the granular layer. This means that, for granular layers up to 1 m (3 ft) in height, the composite reaction can be much less than that of the embankment layer itself. If too high a subgrade design value is selected, the pavement will be under-designed. Granular layers less than 0.5 m (1.6 ft) thick have minimal impact on the composite subgrade reaction, when loaded under the completed pavement section.

7.6.4 Geotextiles and Geogrids

Geosynthetics are a class of geomaterials that are used to improve soil conditions for a number of applications. They consist of manufactured polymeric materials used in contact with soil materials or pavements as an integral part of a man-made system (after ASTM D4439). The most common applications in general use are in pavement systems for both paved and unpaved roadways, for reinforcing embankments and foundation soils, for creating barriers to water flow in liners and cutoffs, and for improving drainage. The generic term "geosynthetic" is often used to cover a wide range of different materials, including geotextiles, geogrids, and geomembranes. Combinations of these materials in layered systems are usually called geocomposites.

Geotextile and geogrid materials are the most commonly used geosynthetics in transportation, although certainly others are sometimes used. This generality is more accurate when only the pavement itself (not including the adjoining fill or cut slopes, retaining walls, abutments, or drainage facilities) is considered. Table 7-14 provides a list of transportation applications for specific basic functions of the geosynthetic. Each of these functional classes, while potentially related by the specific application being proposed, refers to an individual mechanism for the improvement of the soil subgrade. Stabilization, as reviewed in this section, is a combination of the separation, filtration, and reinforcement functions. Drainage can also play a role.

The *separation function* prevents the subgrade and the subbase from intermixing, which would most likely occur during construction and in-service due to pumping of the subgrade. The *filtration function* is required because soils requiring stabilization are usually wet and saturated. By acting as a filter, the geotextile retains the subgrade without clogging, while allowing water from the subgrade to pass up into the subbase, thus allowing destabilizing pore pressure to dissipate and promote strength gain due to consolidation. If the subbase is dirty (contains high fines), it may be desirable to use a thick, nonwoven geotextile, which will allow for *drainage* in its plane (*i.e.,* in this case, pore water pressure dissipates through the plane of the geotextile).

Geotextiles and geogrids also provide some level of *reinforcement* by laterally restraining the base or subbase and improving the bearing capacity of the system, thus decreasing shear stresses on the subgrade. Soft, weak subgrade soils provide very little lateral restraint, so when the aggregate moves or shoves laterally, ruts develop on the aggregate surface and also in the subgrade. A geogrid with good interlocking capabilities or a geotextile with good frictional capabilities can provide tensile resistance to lateral aggregate movement. The geosynthetic also increases the system bearing capacity by forcing the potential bearing surface under the wheel load to develop along alternate, longer mobilization paths and, thus, higher shear strength surfaces.

Geotextiles serve best as separators, filters and, in the case of nonwoven geotextiles, drainage layers, while geogrids are better at reinforcing. Geogrids, as with geotextiles, prevent the subbase from penetrating the subgrade, but they do not prevent the subgrade from pumping into the base. When geogrids are used, either the subbase has to be designed as a separator or a geotextile must be used in conjunction with the geogrid, either separately or as a geocomposite.

Table 7-14. Transportation uses of geosynthetic materials (after Koerner, 1998).

General Function	Typical Application
Separation of Dissimilar Materials	Between subgrade and aggregate base in paved and unpaved roads and airfields Between subgrade and ballast for railroads Between old and new asphalt layers
Reinforcement of weak materials	Over soft soils for unpaved roads, paved roads, airfield, railroads, construction platforms
Filtration	Beneath aggregate base for paved and unpaved roads and airfields or railroad ballast
Drainage	Drainage interceptor for horizontal flow Drain beneath other geosynthetic systems

Table 7-15. Appropriate subgrade conditions for stabilization using geosynthetics (after FHWA HI-95-038).

Condition	Related Measures
Poor soils	USCS of SC, CL, CH, ML, MH, OL, OH, PT or AASHTO of A-5, A-6, A-7, A-7-6
Low strength	c_u<13 psi or CBR<3 or M_R < 4500 psi
High water table	Within zone of influence of surface loads
High sensitivity	High undisturbed strength compared to remolded strength

As defined by AASHTO M288, geotextiles or geogrids in conjunction with an appropriately designed thickness of subbase aggregate provide stabilization for soft, wet subgrades with a CBR of less than 3 (a resilient modulus less than 30 MPa (4500 psi)). Table 7-15 provides subgrade conditions that are considered to be the most appropriate for geosynthetic use. These are conditions where the subgrade will not support conventional construction without substantial rutting. Engineers have compiled over 20+ years of successful use for this application in these types of conditions. Geosynthetics do not provide improvements for expansive soils, and use in stabilization for subgrade conditions that are better than those defined in Table 7-15 is questionable. However, geosynthetics may still provide a valuable function as separators for any subgrade containing large amounts of fines or as base reinforcement, even with competent subgrades, as discussed in Section 7.2.

Separation is a viable function, for soils that are seasonally weak (*e.g.*, from spring thaw) or for high fines content soils, which are susceptible to pumping. This is especially the case for permeable base applications, as covered in Section 7.2. A greater range of geotextile applicability is recognized in the M288 specification (AASHTO, 1997). With a CBR ≥ 3, the geotextile application is identified as separation. By simply maintaining the integrity of the subbase and base layers over the life of the pavement, the serviceability of the roadway section will be extended, and substantial cost benefits can be realized. Research is ongoing to quantify the cost-benefit life cycle ratio of using geosynthetics in permanent roadway systems. Initial work by Al-Qadi, 1997 indicates that the use a geosynthetic separator may increase the number of allowable design vehicles (ESALs) by a factor of two. Considering the cost of a geosynthetic is generally \$1.25/m^2, while the cost of a modern pavement section is on the order of \$25/m^2, the life extension of the roadway section will more than make up for the cost of the geosynthetic. In addition, as previously indicated, the geosynthetic maintains the integrity of the base such that rehabilitation should only require surface pavement restoration. **The ability of a geosynthetic to prevent premature failure and reduce long-term maintenance costs provides extremely low-cost performance insurance.**

The design of the geosynthetic for stabilization is completed using the design-by-function approach in conjunction with AASHTO M288, in the steps from FHWA HI-95-038 outlined below. A key feature of this method is the assumption that the structural pavement design is not modified at all in the procedure. The pavement design proceeds exactly according to standard procedures, as if the geosynthetic was not present. The geosynthetic instead replaces additional unbound material that might be placed to support construction operations, and replaces no part of the pavement section itself. However, this unbound layer will provide some additional support. If the soil has a CBR of less than 3, and the aggregate thickness is determined based on a low rutting criteria in the following steps, the support for the composite system is theoretically equivalent to a CBR = 3 (resilient modulus of 30 Mpa (4500 psi)). As with thick aggregate fill used for stabilization, the support value should be confirmed though field testing using, for example, a plate load test or FWD test to verify that a minimum composite subgrade modulus has been achieved. Note that the FHWA procedure is controlled by soil CBR, as measured using ASTM C4429.

1. Identify properties of the subgrade, including CBR, location of groundwater table, AASHTO and/or USCS classification, and sensitivity.
2. Compare these properties to those in Table 7-15, or with local policies. Determine if a geosynthetic will be required.
3. Design the pavement without consideration of a geosynthetic, using normal pavement structural design procedures.
4. Determine the need for additional imported aggregate to ameliorate mixing at the base/subgrade interface. If such aggregate is required, determine its thickness, t_1, and reduce the thickness by 50%, considering the use of a geosynthetic.
5. Determine additional aggregate thickness t_2 needed for establishment of a construction platform. The FHWA procedure requires the use of curves for aggregate thickness vs. the expected single tire pressure and the subgrade bearing capacity, as shown in Figure 7-21, modified for highway applications. For the purposes of this manual, the curves have been correlated with common pavement construction traffic. Select N_c based on allowable subgrade ruts, where:

   $N_c = 5$ for a low rut criteria (< 50 mm (< 2 in.)),
   $N_c = 5.5$ for moderate rutting (50 – 100 mm (2 – 4 in.)), and
   $N_c = 6$ for large rutting (> 100 mm (> 4 in.)).
   (For comparison without a geotextile: $N_c = 2.8$, 3.0, or 3.3 respectively for low to large ruts.)
   Alternatively, local policies or charts may be used.
6. Select the greater of t_2 or 50% t_1.
7. Check filtration criteria for the geotextile to be used. For geogrids, check the aggregate for filtration compatibility with the subgrade (see Section 7.2), or use a

geotextile in combination with the grid meeting the following criteria. The important measures include the apparent opening size (AOS), the permeability (k), and permittivity (ψ) of the geotextile, and the 95% opening size, defined as the diameter of glass beads for which 95% will be retained on the geosynthetic. These values will be compared to a minimum standard or to the soil properties as follows

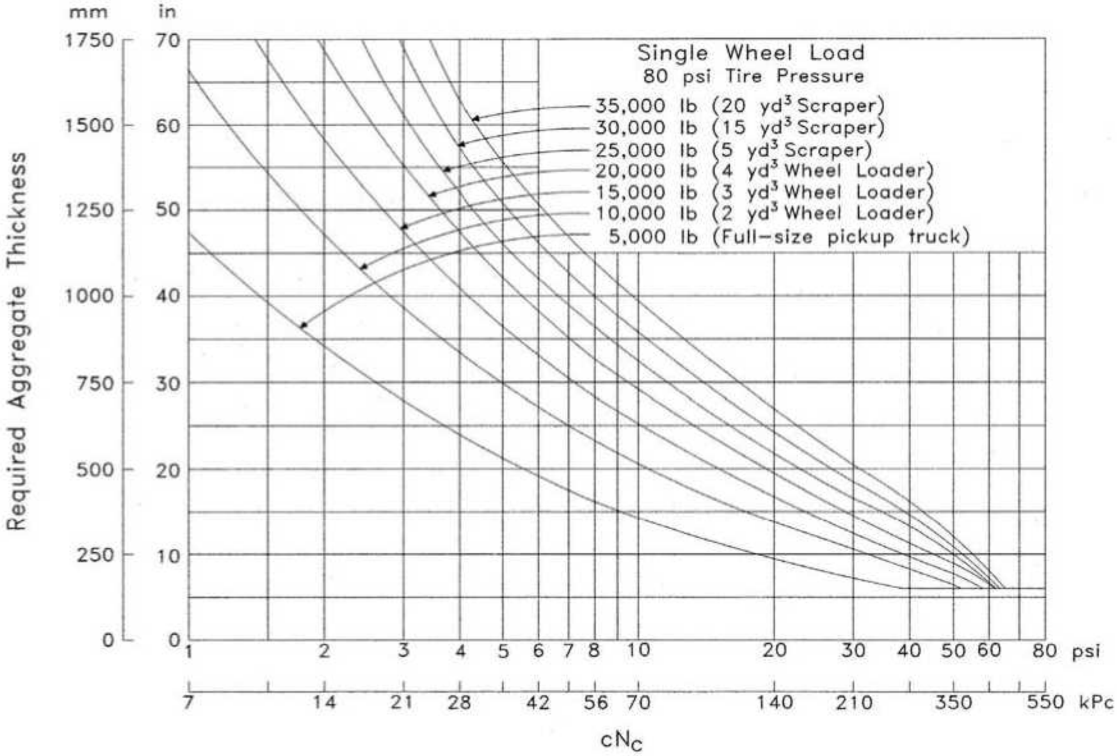
- AOS $\leq D_{85}$ (Wovens)
- AOS $\leq 1.8\ D_{85}$ (Nonwovens)
- $k_{geotextile} \geq k_{soil}$
- $\psi \geq 0.1\ sec^{-1}$

8. Determine geotextile survival criteria. The design is based on the assumption that the geosynthetic cannot function unless it survives the construction process. The AASHTO M288-99 standard categorizes the requirements for the geosynthetic based on the survival class. The requirements for the standard include the strength (grab, seam, tear, puncture, and burst), permittivity, apparent opening size, and resistance to UV degradation, based on the survival class. The survival class is determined from Table 7-5 (Section 7.2.12). For stabilization of soils, the default is Class 1, and for separation, the default is Class 2. These requirements may be reduced based on conditions and experience, as detailed in AASHTO M288. For geogrid survivability, see AASHTO PP46 and Berg et al. (2000).

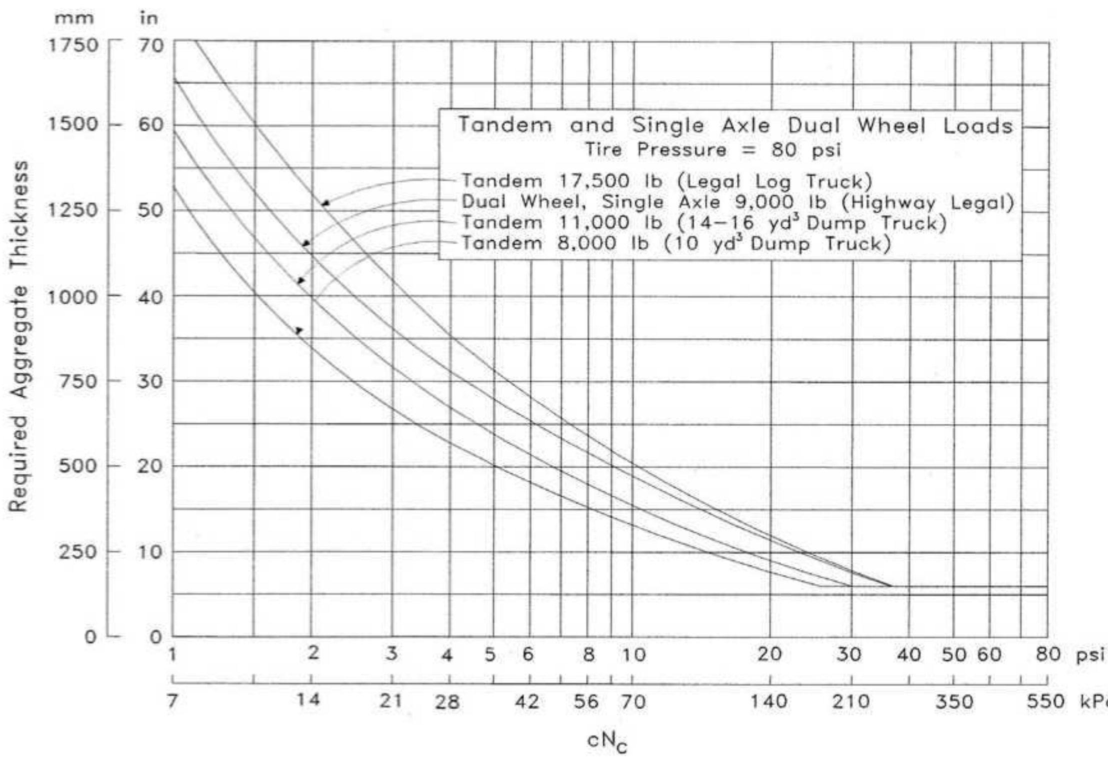
Field installation procedures introduce a number of special concerns; the AASHTO M288 standard includes a guide specification for geotextile construction. FHWA HI-905-038 (Holtz et al. 1998) recommends that this specification be modified to suit local conditions and contractors and provides example specifications. Concerns and criteria for field installation include, for example, the seam lap and sewing requirements, and construction sequencing and quality control.

7.6.5 Admixture Stabilization

As previously indicated in Section 7.6.1, there are a variety of admixtures that can be mixed with the subgrade to improve its performance. The various admixture types are shown in Table 7-15, along with initial guidance for evaluating the appropriate application of these methods. Following is a general overview of each method, followed by a generalized outline for determining the optimum admixture content requirements. Design details for each specific method are contained in Appendix F.



(a)



(b)

Figure 7-21. Thickness design curves with geosynthetics for a) single and b) dual wheel oads (after USFS, 1977, and FHWA NHI-95-038, 1998).

Table 7-16. Guide for selection of admixture stabilization method(s) (Austroads, 1998).

	MORE THAN 25% PASSING 75μm			LESS THAN 25% PASSING 75μm		
Plasticity Index	PI ≤ 10	10 < PI <20	PI ≥ 20	PI ≤ 6 PI x % passing 75μm ≤ 60	PI ≤ 10	PI > 10
Form of Stabilisation						
Cement and Cementitious Blends	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Bitumen/ Cement Blends	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not suitable	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Miscellaneous Chemicals*	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable

Key: Usually suitable; Doubtful; Usually not Suitable

* Should be taken as a broad guideline only. Refer to trade literature for further information.

Note: The above forms of stabilisation may be used in combination, e.g. lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation.

Lime Treatment

Lime treatment or modification consists of the application of 1 – 3% hydrated lime to aid drying of the soil and permit compaction. As such, it is useful in the construction of a "working platform" to expedite construction. Lime modification may also be considered to condition a soil for follow-on stabilization with cement or asphalt. Lime treatment of subgrade soils is intended to expedite construction, and no reduction in the required pavement thickness should be made.

Lime may also be used to treat expansive soils, as discussed in Section 7.3. Expansive soils as defined for pavement purposes are those that exhibit swell in excess of 3%. Expansion is characterized by heaving of a pavement or road when water is imbibed in the clay minerals. The plasticity characteristics of a soil often are a good indicator of the swell potential, as indicated in Table 7-17. If it has been determined that a soil has potential for excessive swell, lime treatment may be appropriate. Lime will reduce swell in an expansive soil to greater or lesser degrees, depending on the activity of the clay minerals present. The amount of lime to be added is the minimum amount that will reduce swell to acceptable limits. Procedures for conducting swell tests are indicated in the ASTM D 1883 CBR test and detailed in ASTM D 4546.

The depth to which lime should be incorporated into the soil is generally limited by the construction equipment used. However, 0.6 – 1 m (2 – 3 ft) generally is the maximum depth that can be treated directly without removal of the soil.

Lime Stabilization

Lime or pozzolonic stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be significantly improved with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective in improving workability and reducing swelling potential with highly plastic clay soils containing montmorillonite, illite, and kaolinite. Lime is also used to reduce the water content of wet soils during field compaction. In treating certain soils with lime, some soils are produced that are subject to fatigue cracking.

Lime stabilization has been found to be an effective method to reduce the volume change potential of many soils. However, lime treatment of soils can convert the soil that shows negligible to moderate frost heave into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been reported that this adverse effect has been caused by an insufficient curing period. Adequate curing is also important if the strength characteristics of the soil are to be improved.

Table 7-17. Swell potential of soils (Joint Departments of the Army & Air Force, 1994).

Liquid Limit	Plasticity Index	Potential Swell
> 60	> 35	High
50 - 60	25 – 35	Marginal
< 50	< 25	Low

The most common varieties of lime for soil stabilization are hydrated lime [$Ca(OH)_2$], quicklime [CaO], and the dolomitic variations of these high-calcium limes [$Ca(OH)_2 \cdot MgO$ and $CaO \cdot MgO$]. While hydrated lime remains the most commonly used lime stabilization admixture in the U.S., use of the more caustic quicklime has grown steadily over the past two decades. Lime is usually produced by calcining[2] limestone or dolomite, although some lime—typically of more variable and poorer quality—is also produced as a byproduct of other chemical processes.

For lime stabilization of clay (or highly plastic) soils, the lime content should be from 3 – 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength of at least 0.34 MPa (50 psi) within 28 days. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. As discussed later in this section, pH can be used to determine the initial, near optimum lime content value. The pozzolanic strength gain in clay soils depends on the specific chemistry of the soil – *e.g.,* whether it can provide sufficient silica and alumina minerals to support the pozzolanic reactions. Plasticity is a rough indicator of reactivity. A plasticity index of about 10 is commonly taken as the lower limit for suitability of inorganic clays for lime stabilization. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by AASHTO T99.

Typical effects of lime stabilization on the engineering properties of a variety of natural soils are shown in Table 7-18 and Figure 7-22. These are the result of several chemical processes that occur after mixing the lime with the soil. Hydration of the lime absorbs water from the soil and causes an immediate drying effect. The addition of lime also introduces calcium (Ca^{+2}) and magnesium (Mg^{+2}) cations that exchange with the more active sodium (Na^{+}) and potassium (K^{+}) cations in the natural soil water chemistry; this cation exchange reduces the plasticity of the soil, which, in most cases, corresponds to a reduced swell and shrinkage potential, diminished susceptibility to strength loss with moisture, and improved workability. The changes in the soil-water chemistry also lead to agglomeration of particles and a coarsening of the soil gradation; plastic clay soils become more like silt or sand in texture after the addition of lime. These drying, plasticity reduction, and texture effects all occur very rapidly (usually with 1 hour after addition of lime), provided there is thorough mixing of the lime and the soil.

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[2] Calcining is the heating of limestone or dolomite to a high temperature below the melting or fusing point that decomposes the carbonates into oxides and hydroxides.

Table 7-18. Examples of the effects of lime stabilization on various soils (Rollings and Rollings, 1996).

Soil	Lime %	Atterberg Limits *LL*	*PL*	*PI*	Strength q_u[a]	CBR
1. CH, residual clay[b]						
(a) Site 1, Dallas–Ft.	0	63	33	30	76	
Worth Airport,	2	62	48	14	123	
residuum from Eagle	3	60	47	13	202	
Ford shale, Britton member	4	56	46	10	323	
(b) Site 2, Dallas–	0	60	27	33	70	
Ft Worth Airport,	2	48	32	16	171	
residuum from Eagle Ford	3	45	32	13	177	
shale, Tarrant member	5	48	34	14	184	
(c) Site 3, Irving, Texas,	0	76	31	45	64	
residuum from Eagle Ford	2	61	45	16	116	
shale, Britton	3	56	45	11	193	
member	5	57	45	12	302	
2. CH, Bryce silty clay,[c]	0	53	24	29	81	
Illinois, B-horizon	3	48	27	21	201	
	5	NP	NP	NP	212	
3. CH, Appling sandy loam,[d]	0	71	33	38	92	
South Carolina, residuum	3				147	
from granite	6				171	
	8				206	
4. CH, St Ann red bauxite	0	58	25	33	119	
clay loam,[d] Jamaica,	3				127	
limestone residuum	5				334	
5. CL,[e] Pelucia Creek Dam,	0	29	18	11		
Mississippi	1	32	19	13		
	2	31	22	9		
	3	30	21	9		
6. CL, Illinoian till, Illinois,[c]	0	26	15	11	43	
glacial till	3	27	21	6	126	
	5	NP	NP	NP	126	
7. SC, sandy clay, San Lorenzo,	0	54	23	31		8
Honduras[f]	5	61	38	23		20
8. MH, Surinam red earth,[d]	0	60	32	28	72	
Surinam,	3				130	
residuum from acidic	5				136	
metamorphic rock						
9. OH, organic soil with 8.1%	0	63	27	36	4	
organics[g]	2			36	4	
	4			24	8	
	8			25	7	

[a]Unconfined compressive strength in psi at 28 days unless otherwise noted; different compaction efforts used by investigators.
[b]McCallister and Petry, 1990, accelerated curing.
[c]Thompson, 1966.
[d]Harty, 1971, 7-day cure.
[e]McElroy, 1989.
[f]Personal communication, Dr. Newel Brabston, Vicksburg, Mississippi.
[g]Arman and Munfakh, 1972, limits at 48 hours, q_u at 28 days, strength samples prepared with moisture content at the *LL*.

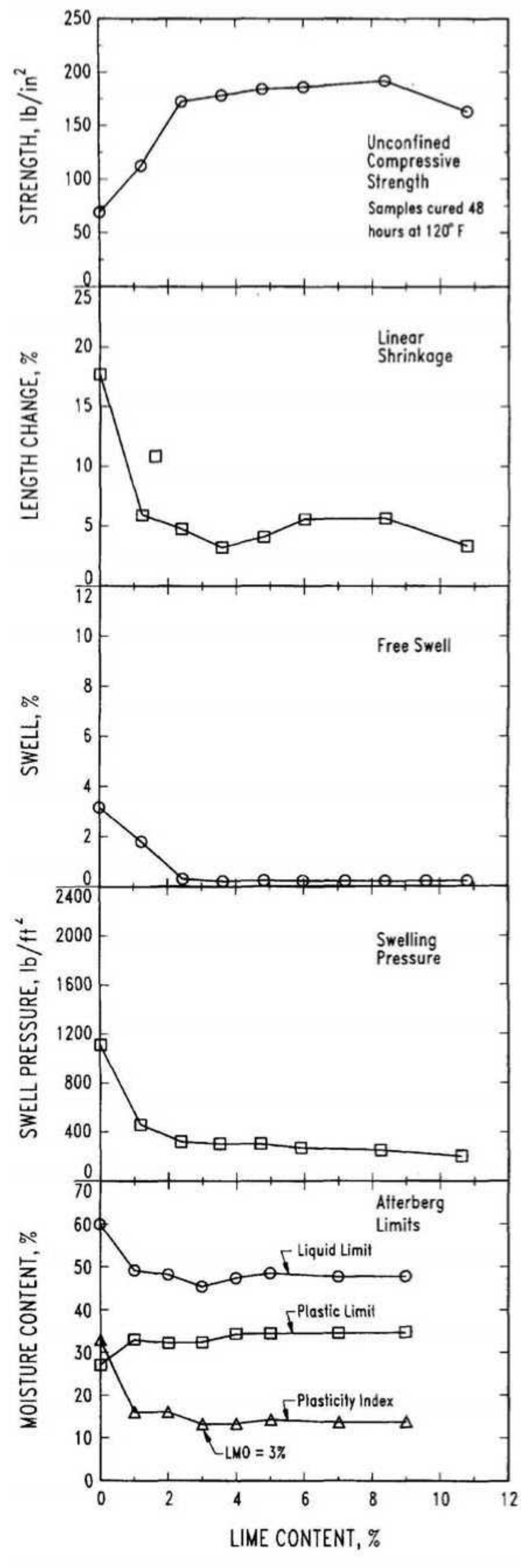

Figure 7-22. Effect of lime content on engineering properties of a CH clay (from Rollings and Rollings, 1996; from data reported by McCallister and Petry, 1990).

When soils are treated properly with lime, it has been observed that the lime-soil mixture may be subject to durability problems, the cyclic freezing and thawing of the soil. The durability of lime stabilization on swell potential and strength may be adversely affected by environmental influences:

- *Water:* Although most lime stabilized soils retain 70% to 85% of their long-term strength gains when exposed to water, there have been reported cases of poor strength retention for stabilized soils exposed to soaking. Therefore, testing of stabilized soils in the soaked condition is prudent.

- *Freeze/thaw cycles:* Freeze/thaw cycles can lead to strength deterioration, but subsequent healing often occurs where the strength loss caused by winter freeze/thaw reverses during the following warm season. The most common design approach is to specify a sufficiently high initial strength gain to retain sufficient residual strength after freeze/thaw damage.

- *Leaching:* Leaching of calcium can decrease the cation exchange in lime stabilized soil, which, in turn, can reverse the beneficial reduction in plasticity and swell potential. The potential for these effects is greater when low lime contents are used.

- *Carbonation:* If atmospheric carbon dioxide combines with lime to form calcium carbonate, the calcium silicate and calcium aluminate hydrate cements may become unstable and revert back to their original silica and alumina forms, reversing the long-term strength increase resulting from the pozzolanic reactions. Although this problem has been reported less in the United States than in other countries, its possibility should be recognized and its potential minimized by use of ample lime content, careful selection, placement, and compaction of the stabilized material to minimize carbon dioxide penetration, as well as prompt placement after lime mixing, and good curing.

- *Sulfate attack:* Sulfates present in the soil or groundwater can combine with the calcium from the lime or the alumina from the clay minerals to form ettringite, which has a volume that is more than 200% larger than that of its constituents. Massive irreversible swelling can therefore occur, and the damage it causes can be quite severe. It is difficult to predict the combinations of sulfate content, lime content, clay mineralogy and content, and environmental conditions that will trigger sulfate attack. Consequently, if there is a suspicion of possible sulfate attack, the lime stabilized soil should be tested in the laboratory to see whether it will swell when mixed and exposed to moisture.

Soils classified as CH, CL, MH, ML, SC, and GC with a plasticity index greater than 12 and with 10% passing the 0.425 mm (No. 40) sieve are potentially suitable for stabilization with lime. Lime-flyash stabilization is applicable to a broader range of soils because the cementing action of the material is less dependent on the fines contained within the soil. However, long-term durability studies of pavements with lime-flyash stabilization are rather limited.

Hydrated lime, in powder form or mixed with water as a slurry, is used most often for stabilization.

Cement Stabilization

Portland cement is widely used for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement or cement-treated base, subbase, or subgrade.

For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20% and a minimum of 45% passing the 0.425 mm (No. 40) sieve. However, highly plastic clays that have been pretreated with lime or flyash are sometimes suitable for subsequent treatment with Portland cement. For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 – 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 1 MPa (150 psi) within 7 days. The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95% as defined by AASHTO M 134.

Several different types of cement have been used successfully for stabilization of soils. Type I normal Portland cement and Type IA air-entraining cements were used extensively in the past, and produced about the same results. At the present time, Type II cement has largely replaced Type I cement as greater sulfate resistance is obtained, while the cost is often the same. High early strength cement (Type III) has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than do the other cement types. Chemical and physical property specifications for Portland cement can be found in ASTM C 150.

The presence of organic matter and/or sulfates may have a deleterious effect on soil cement. Tests are available for detection of these materials and should be conducted if their presence is suspected.

(1) *Organic matter.* A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of organic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds of lower molecular weight, such as nucleic acid and dextrose, act as hydration retarders and reduce strength. When such organics are present, they inhibit the normal hardening process. If the pH of a 10:1 mixture (by weight) of soil and cement 15 minutes after mixing is at least 12.0, it is probable that any organics present will not interfere with normal hardening.

(2) *Sulfates.* Although sulfate attack is known to have an adverse effect on the quality of hardened Portland cement concrete, less is known about the sulfate resistance of cement stabilized soils. The resistance to sulfate attack differs for cement-treated, coarse-grained and fine-grained soils, and is a function of sulfate concentrations. Sulfate-clay reactions can cause deterioration of fine-grained soil-cement. On the other hand, granular soil-cements do not appear susceptible to sulfate attack. In some cases, the presence of small amounts of sulfate in the soil at the time of mixing with the cement may even be beneficial. The use of sulfate-resistant cement may not improve the resistance of clay-bearing soils, but may be effective in granular soil-cements exposed to adjacent soils and/or groundwater containing high sulfate concentrations. The use of cement for fine-grained soils containing more than about 1% sulfate should be avoided.

Stabilization with Lime-Flyash (LF) and Lime-Cement-Flyash (LCF)

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of LF or LCF combinations. Flyash, also termed coal ash, is a mineral residual from the combustion of pulverized coal. It contains silicon and aluminum compounds that, when mixed with lime and water, forms a hardened cementitious mass capable of obtaining high compressive strengths. Lime and flyash in combination can often be used successfully in stabilizing granular materials, since the flyash provides an agent with which the lime can react. Thus LF or LCF stabilization is often appropriate for base and subbase course materials.

Flyash is classified according to the type of coal from which the ash was derived. Class C flyash is derived from the burning of lignite or subbituminous coal and is often referred to as "high lime" ash because it contains a high percentage of lime. Class C flyash is self-reactive or cementitious in the presence of water, in addition to being pozzolanic. Class F flyash is derived from the burning of anthracite or bituminous coal and is sometimes referred to as "low lime" ash. It requires the addition of lime to form a pozzolanic reaction. To be acceptable quality, flyash used for stabilization must meet the requirements indicated in ASTM C 593.

Design with LF is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, flyash, and lime), a number of factors can be varied in the mix design process, such as percentage of lime-flyash, the moisture content, and the ratio of lime to flyash. It is generally recognized that engineering characteristics such as strength and durability are directly related to the quality of the matrix material. The matrix material is that part consisting of flyash, lime, and minus No. 4 aggregate fines. Basically, higher strength and improved durability are achievable when the matrix material is able to "float" the coarse aggregate particles. In effect, the fine size particles overfill the void spaces between the coarse aggregate particles. For each coarse aggregate material, there is a quantity of matrix required to effectively fill the available void spaces and to "float" the coarse aggregate particles. The quantity of matrix required for maximum dry density of the total mixture is referred to as the optimum fines content. In LF mixtures, it is recommended that the quantity of matrix be approximately 2% above the optimum fines content. At the recommended fines content, the strength development is also influenced by the ratio of lime to flyash. Adjustment of the lime-flyash ratio will yield different values of strength and durability properties.

Asphalt Stabilization

Generally, asphalt-stabilized soils are used for base and subbase construction. Use of asphalt as a stabilizing agent produces different effects, depending on the soil, and may be divided into three major groups: 1) sand-bitumen, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-bitumen, which stabilizes the moisture content of cohesive fine-grained soils, and 3) sand-gravel bitumen, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of bitumen-stabilized mixtures generally can be assessed by measurement of their water absorption characteristics. Treatment of soils containing fines in excess of 20% is not recommended.

Stabilization of soils and aggregates with asphalt differs greatly from cement and lime stabilization. The basic mechanism involved in asphalt stabilization of fine-grained soils is a waterproofing phenomenon. Soil particles or soil agglomerates are coated with asphalt that prevents or slows the penetration of water that could normally result in a decrease in soil strength. In addition, asphalt stabilization can improve durability characteristics by making the soil resistant to the detrimental effects of water, such as volume. In noncohesive materials, such as sands and gravel, crushed gravel, and crushed stone, two basic mechanisms are active: waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane that prevents or hinders the penetration of water and thereby reduces the tendency of the material to lose strength in the presence of water. The second mechanism has been identified as adhesion. The aggregate particles adhere to the asphalt and the asphalt acts as a binder or cement. The cementing effect thus increases shear strength by increasing cohesion. Criteria for design of bituminous-stabilized soils and aggregates are based almost entirely on stability and gradation requirements. Freeze-thaw and wet-dry durability tests are not applicable for asphalt-stabilized mixtures.

There are three basic types of bituminous-stabilized soils, including

(1) *Sand bitumen.* A mixture of sand and bitumen in which the sand particles are cemented together to provide a material of increased stability.
(2) *Gravel or crushed aggregate bitumen.* A mixture of bitumen and a well-graded gravel or crushed aggregate that, after compaction, provides a highly stable waterproof mass of subbase or base course quality.
(3) *Bitumen lime.* A mixture of soil, lime, and bitumen that, after compaction, may exhibit the characteristics of any of the bitumen-treated materials indicated above. Lime is used with materials that have a high PI, *i.e.,* above 10.

Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions. In frost areas, the use of tar as a binder should be avoided because of its high temperature susceptibility. Asphalts are affected to a lesser extent by temperature changes, but a grade of asphalt suitable to the prevailing climate should be selected. As a general rule, the most satisfactory results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used. For higher quality mixes in which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in-place, with the bitumen being applied directly on the soil or soil aggregate system, and the mixing and compaction operations being conducted immediately thereafter. For this type of construction, liquid asphalts, *i.e.,* cutbacks and emulsions, are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, the type of construction equipment, and the climatic conditions. Generally, the following types of bituminous materials will be used for the soil gradation indicated:

(1) Open-graded aggregate.
   a. Rapid- and medium-curing liquid asphalts RC-250, RC-800, and MC-3000.
   b. Medium-setting asphalt emulsion MS-2 and CMS-2.

(2) Well-graded aggregate with little or no material passing the 0.075 mm (No. 200) sieve.
   a. Rapid and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800.
   b. Slow-curing liquid asphalts SC-250 and SC-800.
   c. Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1.

(3) Aggregate with a considerable percentage of fine aggregates and material passing the 0.075 mm (No. 200) sieve.
   a. Medium-curing liquid asphalt MC-250 and MC-800.
   b. Slow-curing liquid asphalts SC-250 and SC-800
   c. Slow-setting asphalt emulsions SS-1, SS-01h, CSS-1, and CSS-lh.

The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

The recommended soil gradations for subgrade materials and base or subbase course materials are shown in Tables 7-19 and 7-20, respectively.

Table 7-19. Recommended gradations for bituminous-stabilized subgrade materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	Percent Passing
75-mm (3-in.)	100
4.75-mm (#4)	50-100
600-µm (#30)	38-100
75-µm (#200)	2-30

Table 7-20. Recommended gradations for bituminous-stabilized base and subbase materials (Joint Departments of the Army and Air Force, 1994).

Sieve Size	37.5 mm (1 ½ in.) Maximum	25 mm (1-in.) Maximum	19 mm (¾-in.) Maximum	12.7 mm (½-in.) Maximum
37.5-mm (1½-in.)	100	-	-	-
25-mm (l-in.)	8 4 ± 9	100	-	-
19-mm (¾-in.)	76 ± 9	83 ± 9	100	-
M-in	66 ± 9	73 ± 9	82 ± 9	100
9.5-mm (3/8-in.)	59 ± 9	64 ± 9	72 ± 9	83 ± 9
0.475-mm (#4)	45 ± 9	48 ± 9	54 ± 9	62 ± 9
2.36-mm (#8)	35 ± 9	36 ± 9	41 ± 9	47 ± 9
1.18-mm (#16)	27 ± 9	28 ± 9	32 ± 9	36 ± 9
600-µm (#30)	20 ± 9	21 ± 9	24 ± 9	28 ± 9
300-µm (#50)	14 ± 7	16 ± 7	17 ± 7	20 ± 7
150-µm (#100)	9 ± 5	11 ± 5	12 ± 5	14 ± 5
75-µm (#200)	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Stabilization with Lime-Cement and Lime-Bitumen

The advantage of using combination stabilizers is that one of the stabilizers in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas devoid of base material, lime has been used jointly with other stabilizers, notably Portland cement or asphalt, to provide acceptable base courses. Since Portland cement or asphalt cannot be mixed successfully with plastic clays, the lime is added first to reduce the plasticity of the clay. While such stabilization practice might be more costly than the conventional single stabilizer methods, it may still prove to be economical in areas where base aggregate costs are high. Two combination stabilizers are considered in this section: lime-cement and lime-asphalt.

a) *Lime-cement.* Lime can be used as an initial additive with Portland cement, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics, mainly by reducing the plasticity of the soil. The design approach is to add enough lime to improve workability and to reduce the plasticity index to acceptable levels. The design lime content is the minimum that achieves desired results. The design cement content is determined following procedures for cement-stabilized soils presented in Appendix F.

b) *Lime-asphalt.* Lime can be used as an initial additive with asphalt, or as the primary stabilizer. The main purpose of lime is to improve workability characteristics and to act as an anti-stripping agent. In the latter capacity, the lime acts to neutralize acidic chemicals in the soil or aggregate that tend to interfere with bonding of the asphalt. Generally, about 1 – 2% percent lime is all that is needed for this objective. Since asphalt is the primary stabilizer, the procedures for asphalt-stabilized materials, as presented Appendix F, should be followed.

Admixture Design

Design of admixtures takes on a similar process regardless of the admixture type. The following steps are generally followed and are generic to lime, cement, L-FA and L-C-FA, or asphalt admixtures.

Step 1. Classify soil to be stabilized.
(% < 0.075 mm – No. 200 sieve, % < 0.425 mm – No. 40 Sieve, PI, etc.)

Step 2. Prepare trial mixes with varying % content.
Lime: Select lowest % with pH = 12.4 in 1 hour
Cement: Use table to estimate cement content requirements
Asphalt: Use equation & table in Appendix F to estimate the quantity of cutback asphalt

Step 3. Develop moisture-density relationship for initial design.

Step 4. Prepare triplicate samples and cure specimens at target density.
Use optimum water content and % initial admixture, +2% and +4%

Step 5. Determine index strength.
Lime and Cement: Determine unconfined compressive strength (ASTM D 5102)
Asphalt: Determine Marshall stability

Step 6. Determine resilient modulus for optimum percent admixture.
Perform test or estimate using correlations (See Chapter 5)

Step 7. Conduct freeze-thaw tests (Regional as required).
(For Cement, CFA, L-C-FA)

Step 8. Select % to achieve minimum design strength and F-T durability.

Step 9. Add 0.5 – 1% to compensate for non-uniform mixing.

Appendix F provides specific design requirements and design step details for each type of admixture reviewed in this section. Additional design and construction information can also be obtained from industry publications including

- *Soil-Cement Construction Handbook*, Portland Cement Association, Skokie, Il, 1995.
- *Lime-Treated Soil Construction Manual: Lime Stabilization & Lime Modification,* National Lime Associaiton, Arlington, Virginia, 2004.
- *Flexible Pavement Manual*, American Coal Ash Association, Washington, D.C., 1991.
- *A Basic Emulsion Manual*, Asphalt Institute, Manual Series #19.
- http://www.cement.org/index.asp
- http://www.lime.org/

7.6.6 Soil Encapsulation

Soil encapsulation is a foundation improvement technique that has been used to protect moisture sensitive soils from large variations in moisture content. The concept of soil encapsulation is to keep the fine-grained soils at or slightly below optimum moisture content, where the strength of these soils can support heavier trucks and traffic. This technique has been used by a number of states (*e.g.,* Texas and Wyoming) on selected projects to improve the foundations of higher volume roadways. It is more commonly used as a technique in Europe and in foundation or subbase layers for low-volume roadways, where the import of higher quality paving materials is restricted from a cost standpoint. More than 100 projects have been identified around the world, usually reporting success in controlling expansive soils (Steinberg, 1998).

Fine-grained soils can provide adequate bearing strengths for use as structural layers in pavements and embankments, as long as the moisture content remains below the optimum moisture content. However, increases in moisture content above the optimum value can cause a significant reduction in the stiffness (*i.e.,* resilient modulus) and strength of fine-grained materials and soils. Increased moisture content in fine-grained soils below pavements occurs over time, especially in areas subject to frost penetration and freeze-thaw cycles. Thus, fine-grained soils cannot be used as a base or subbase layer unless the soils are protected from any increase in moisture.

The soil encapsulation concept, sometimes referred to as membrane encapsulated soil layer (MESL), is a method for maintaining the moisture content of the soil at the desired level by encapsulating the soil in waterproof membranes. The waterproof membranes prevent water from infiltrating the moisture sensitive material. The resilient modulus measured at or below optimum conditions remains relatively constant over the design life of the pavement.

The prepared subgrade is normally sprayed with an asphalt emulsion before the bottom membrane of polyethylene is placed. This asphalt emulsion provides added waterproofing protection in the event the membrane is punctured during construction operations, and acts as an adhesive for the membrane to be placed in windy conditions. The first layer of soil is placed in sufficient thickness such that the construction equipment will not displace the underlying material. The completed soil embankment is also sprayed with an asphalt emulsion before placement of the top membrane. To form a complete encapsulation, the bottom membrane is brought up the sides and wrapped around the top, for an excavated section, or the top membrane is draped over the sides, for an embankment situation. The top of the membrane is sprayed with the same asphalt emulsion and covered with a thin layer of clean sand to blot the asphalt and to provide added protection against puncture by the construction equipment used to place the upper paving layers.

The reliability of this method to maintain the resilient modulus and strength of the foundation soil over long periods of time is unknown. More importantly, roadway maintenance and the installation of utilities in areas over time limit the use of this technique. Thus, this improvement technique is not suggested unless there is no other option available.
If this technique is used, the pavement designer should be cautioned regarding the use of the environmental effects model (EICM) to predict changes in moisture over time. Special design computations will be needed to restrict the change in moisture content of the MESL over time. The resilient modulus used in design for the MESL should be held constant over the design life of the pavement. The designer should also remember that any utilities placed after pavement construction could make that assumption invalid.

7.6.7 Lightweight Fill

When constructing pavements on soft soils, there is always a concern for settlement. For deeper deposits where shallow surface stabilization may not be effective, thicker granular aggregate as discussed in Section 7.3, may be effective for control deformation under wheel load, but would increase the concern for settlement. An alternate to replacement with aggregate would be to use lightweight fill.

The compacted unit density of most soil deposits consisting of sands, silts, or clays ranges from about 1,800 – 2,200 kg/m^3 (112 – 137 lbs/ft^3) Lightweight fill materials are available from the lower end of this range down to 12 kg/m^3 (0.75 lbs/ft^3). In many cases, the use of lighter weight materials on soft soils will likely result in both reduced settlement and increased stability. The worldwide interest and use of lightweight fill materials has led to the recent publication by the Permanent International Association of Road Congresses (PIARC) of an authoritative reference *"Lightweight Filling Materials"* in 1997.

Many types of lightweight fill materials have been used for roadway construction. Some of the more common lightweight fills are listed in Table 7-21. There is a wide range in density of the lightweight fill materials, but all have a density less than conventional soils. Addtional information on the composition and sources of the lightweight fill materials listed in Table 7-21 can be found in FHWA NHI-04-001 Ground Improvement Methods technical summaries.

Some lightweight fill materials have been used for decades, while others are relatively recent developments. Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low-lying land, as well as for repair of slide zones.

The steel-making companies have produced slag since the start of the iron and steel making industry. Initially, the slag were stockpiled as waste materials, but beginning around 1950, the slag were crushed, graded, and sold for fill materials.

Geofoam is a generic term used to describe any foam material used in a geotechnical application. Geofoam includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam (cellular glass). Geofoam was initially developed for insulation material to prevent frost from penetrating soils. The initial use for this purpose was in Scandinavia and North America in the early 1960s. In 1972, the use of geofoam was extended as a lightweight fill for a project in Norway.

The technique of using pumping equipment to inject foaming agents into concrete was developed in the late 1930s. Little is known about the early uses of this product. However, the U.S. Army Corps of Engineers used foamed concrete as a tunnel lining and annular fill. This product is generally job-produced as a cement/water slurry with preformed foam blended for accurate control and immediate placement.

Table 7-21. Densities and approximate costs for various lightweight fill materials.

Fill Type	Range in Density kg/m^3	Range in Specific Gravity	Approximate Cost[1] \$/m^3
Geofoam (EPS)	12 to 32	0.01 to .03	40.00 to 85.00[2]
Foamed Concrete	320 to 970	0.3 to 0.8	40.00 to 55.00
Wood Fiber	550 to 960	0.6 to 1.0	12.00 to 20.00[2]
Shredded Tires	600 to 900	0.6 to 0.9	20.00 to 30.00[2]
Expanded Shale And Clay	600 to 1040	0.6 to 1.0	40.00 to 55.00[3]
Flyash	1120 to 1440	1.1 to 1.4	15.00 to 21.00[3]
Boiler Slag	1000 to 1750	1.0 to 1.8	3.00 to 4.00[3]
Air-Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9.00[3]

[1] See Chapter 6 for details on cost data

[2] Price includes transportation and placement cost

[3] FOB plant

Shredded tires and tire bales are a relatively recent source of lightweight fill materials. The availability of this material is increasing each year, and its use as a lightweight fill is further promoted by the need to dispose of tires. In most locations, the tires are stockpiled, but they are unsightly and present a serious fire and health hazard. Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. More than 85 fills using shredded tires as a lightweight fill have been constructed in the United States. In 1995, three tire shred fills with a thickness greater than 8 m (26 ft) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (9.8 ft).

Expanded shale lightweight aggregate has been used for decades to produce aggregate for concrete and masonry units. Beginning in about 1980, lightweight aggregates have also been used for geotechnical purposes. Completed projects include the Port of Albany, New York marine terminal, where lightweight fill was used behind a bulkhead to reduce the lateral pressures on the steel sheeting. Other projects include construction of roadways over soft ground. The existing high-density soils were partially removed and replaced with lightweight aggregate to reduce settlement. Other projects have included improvement of slope stability by reduction of the gravitational driving force of the soil in the slope and replacement with a lightweight fill.

Waste products from coal burning include flyash and boiler slag. Both of these materials have been used in roadway construction. One of the first documented uses of flyash in an engineered highway embankment occurred in England in 1950. Trial embankments led to the acceptance of flyash fills, and other roadway projects were constructed in other European countries. In 1965, a flyash roadway embankment was constructed in Illinois. In 1984, a project survey found that flyash was used in the construction of 33 embankments and 31 area fills. Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.

The FHWA NHI-04-001 provides an overview of the more common lightweight fill materials that have been used for geotechnical applications in highway construction. Typical geotechnical engineering parameters that are important for design are provided. In addition, design and construction considerations unique to each of these lightweight fill materials are presented. This information can be used for preliminary planning purposes. The technical summary also presents guidelines for preparation of specifications along with suggested construction control procedures. Four case histories are also presented to demonstrate the effectiveness of lightweight fills for specific situations. Approximate costs for the various lightweight fill materials are also presented.

With regard to pavement design, if a minimum of 1 m (3 ft) of good quality gravel type fill is placed between the pavement structure and the lightweight materials as a cover, then the lightweight material will have little impact on pavement design, even for the more compressible tire and geofoam materials. However, if a thinner cover must be used, the support value for these materials must be determined. Lab tests can be used, as discussed in Chapter 5, especially for the granular type materials. The ideal method is to perform field resilient modulus tests on placed material (*i.e.,* on cover soils after placement over the lightweight material(s)), especially for the bulkier materials, such as tires and geofoam.

7.6.8 Deep Foundations and Other Foundation Improvement Methods (from Elias et al., 2004)

In some cases, the extent (area and depth) of poor subgrade conditions are too large for surface stabilization or removal. In extreme cases, the soils may be too week to support the roadway embankment (even for embankments that only consist of the pavement structure). In these cases, other deep ground improvement methods, such as deep foundations, may be required. Ground improvement technologies are geotechnical construction methods used to alter and improve poor ground conditions so that embankment and structure construction can meet project performance requirements where soil replacement is not feasible for environmental or technical reasons, or it is too costly.

Ground improvement has one or more than one of the following main functions:

- to increase bearing capacity, shear or frictional strength,
- to increase density,
- to control deformations,
- to accelerate consolidation,
- to decrease imposed loads,
- to provide lateral stability,
- to form seepage cutoffs or fill voids,
- to increase resistance to liquefaction and,
- to transfer embankment loads to more competent layers

There are three strategies available to accomplish the above functions representing different approaches. The first method is to increase the shear strength, density, and/or decrease the compressibility of the foundation soil. The second method is to utilize a lightweight fill embankment to reduce significantly the applied load to the foundation, and the third method is to transfer loads to a more competent deeper layer.

The selection of candidate ground improvement methods for any specific project follows a sequential process. The steps in the process include a sequence of evaluations that proceed from simple to more detailed, allowing a best method to emerge. The process is described as follows:

1) *Identify potential poor ground conditions, their extent, and type of negative impact.* Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause long-term settlement, or cause construction or post-construction instability.
2) *Identify or establish performance requirements.* Performance requirements generally consist of deformation limits (horizontal and vertical), as well as some minimum factors of safety for stability. The available time for construction is also a performance requirement.
3) *Identify and assess any space or environmental constraints.* Space constraints typically refer to accessibility for construction equipment to operate safely, and environmental constraints may include the disposal of spoil (hazardous or not hazardous) and the effect of construction vibrations or noise.
4) *Assessment of subsurface conditions.* The type, depth, and extent of the poor soils must be considered, as well as the location of the ground-water table. It is further valuable to have at least a preliminary assessment of the shear strength and compressibility of the identified poor soils.
5) *Preliminary selection.* Preliminary selection of potentially applicable method(s) is generally made on a qualitative basis, taking into consideration the performance

criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and the level of improvement that is required. Table 7-22, which groups the available methods in six broad categories, can be used as a guide in this process to identify possible methods and eliminate those that by themselves, or in conjunction with other methods, cannot produce the desired performance.

6) *Preliminary design*. A preliminary design is developed for each method identified under "Preliminary selection" and a cost estimate prepared on the basis of data in Table 7-23. The guidance in developing preliminary designs is contained within each Technical Summary.
7) *Comparison and selection*. The selected methods are then compared, and a selection made by considering performance, constructability, cost, and other relevant project factors.

State-of-the-art design and construction methods and/or references are provided in each of the FHWA NHI-04-001 Ground Improvement Methods technical summaries to form the basis of a final design. The success of any ground improvement method is predicated on the implementation of a QA/QC program to verify that the desired foundation improvement level has been reached. These programs incorporate a combination of construction observations, in-situ testing and laboratory testing to evaluate the treated soil in the field. Details are provided in each technical summary contained in the FHWA NHI-04-001.

Table 7-22. Ground improvement categories, functions, methods and applications (Elias et al., 2004).

Category	Function	Methods	Comment
Consolidation	Accelerate consolidation, increase shear strength	(1) Wick drains (2) Vacuum consolidation	Viable for normally consolidated clays. Vacuum consolidation viable for very soft clays. Can achieve up to 90% consolidation in a few months.
Load Reduction	Reduce load on foundation, reduce settlement	(1) Geofoam, (2) Foamed concrete (3) Lightweight granular fills, tire chips, etc.	Density varies from 1 – 12 kN/m^3 (6 – 76 lb/ft^3). Granular fills usage subject to local availability.
Densification	Increase density, bearing capacity and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction.	(1) Vibro-compaction using vibrators (2) Dynamic compaction by falling weight impact	Vibrocompaction viable for clean sands with <15% fines. Dynamic compaction limited to depths of about 10 m (33 ft), but is applicable for a wider range of soils. Both methods can densify granular soils up to 80% Relative Density. Dynamic compaction generates vibrations for a considerable lateral distance.
Reinforcement	Internally reinforces fills and/or cuts. In soft foundation soils, increases shear strength, resistance to liquefaction and decreases compressibility.	(1) MSE retaining walls (2) Soil Nailing walls (3) Stone column to reinforce foundations	Soil Nailing may not applicable in soft clays or loose fills. Stone columns applicable in soft clay profiles to increase global shear strength and reduce settlement.
Chemical Stabilization by Deep Mixing Methods	Physio-chemical alteration of foundation soils to increase their tensile, compressive and shear strength, and to decrease settlement and/or provide lateral stability and or confinement.	(1) Wet mixing methods using primarily cement (2) Dry mixing methods using lime-cement	Applicable in soft to medium stiff clays for excavation support where the groundwater table must be maintained, or for foundation support where lateral restraint must be provided, or to increase global stability and decrease settlement. Requires significant QA/QC program for verification.

Table 7-22. Ground improvement categories, functions, methods and applications (continued).

Category	Function	Methods	Comment
Chemical Stabilization by Grouting	To form seepage cutoffs, fill voids, increase density, increase tensile and compressive strength	(1) Permeation grouting with particulate or chemical grouts (2) Compaction grouting (3) Jet grouting, and (4) Bulk filling	(1) Permeation grouting to increase shear strength or for seepage control, (2) compaction grouting for densification and (3) jet grouting to increase tensile and/or compressive strength of foundations, and (4) bulk filling of any subsurface voids.
Load Transfer	Transfer load to deeper bearing layer	Column (Pile) supported embankments on flexible geosynthetic mats	Applicable for deep soft soil profiles or where a tight schedule must be maintained. A variety of stiff or semi-stiff piles can be used.

Table 7-23a. Comparative Costs (SI units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/m^3
Wick Drains	$ 1.50 - 4.00/m	$ 0.80 - 1.60
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 $ 3.00 - 21.00/m^3 $ 12.00 - 30.00/m^3 $ 35.00 - 65.00/m^3 $ 45.00 - 65.00/m^3	
Vibrocompaction	$ 15.00 - 25.00/m	$ 1.00 - 4.00
Dynamic Compaction	$ 6.00 - 11.00/m^2	$ 1.00 - 2.00
MSE Walls	$ 160.00 - 300.00/m^2	
RSS Slopes	$ 110.00 - 260.00/m^2	
Soil Nail Walls	$ 400.00 - 600.00/m^2	
Stone Columns	$ 40.00 - 60.00/m	$ 50 - 75
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 $30.00/m 	 $ 60 $ 85 -150
Grouting Permeation Compaction Jet	 $ 65.00/m + $ 0.70/Liter 	 $ 30 - 200 $ 200 - 275
Column-Supported Embankments	$ 95/m^2 + cost of column	n/a

Table 7-23b. Comparative Costs (U.S. customary units) (Elias et al., 2004).

Method	Unit Cost	Cost of Treated Volume $/yd^3
Wick Drains	$ 0.46 - 1.22/ft	$ 0.60 - 1.20
Lightweight Fill Granular Tires-Wood Geofoam Foamed Concrete	 $ 2.30 - 16.10/yd^3 $ 9.20 - 23.00/yd^3 $ 26.75 - 50.00/yd^3 $ 34.50 - 50.00/yd^3	
Vibrocompaction	$ 4.60 - 7.60/ft	$ 0.75 – 3.00
Dynamic Compaction	$ 5.00 - 9.20/yd^2	$ 0.75 - 1.50
MSE Walls	$ 15.00 - 28.00/ft^2	
RSS Slopes	$ 10.00 - 24.00/ft^2	
Soil Nail Walls	$ 37.00 - 56.00/ft^2	
Stone Columns	$ 12.20 - 18.30/ft	$ 38 - 57
Deep Soil Mixing Dry w/lime-cement Wet w/cement	 $9.15/ft 	 $ 46 $ 65 -115
Grouting Permeation Compaction Jet	 $ 20/ft + $ 2.65/Gallon 	 $ 23 - 153 $ 150 - 210
Column Supported Embankments	$ 81.50/yd^2 + cost of column	n/a

7.7 RECYCLE

Recycling, in principal, is a very powerful and often political concept. While the benefits of recycling including conservation of aggregate and binders and preservation of the environment, it requires serious consideration. The long-term performance of recycled materials in pavements and, in come cases the environmental impact, must be carefully evaluated to avoid costly performance and maintenance issues. In this section, the evaluation requirements for recycled materials will be reviewed. There are two forms of recycling in pavements: 1) reuse of the pavement materials themselves and 2) the use of recycled waste materials for subgrade stabilization or as a substitute for aggregate.

7.7.1 Pavement Recycling

The method of recycling the pavement will, in most cases, depend on whether the surface pavement has an AC or PCC surface pavement. In either case, the material could be rubblized, or, in some cases, processed (*e.g.,* sieving, stockpiling, and reusing the reclaimed asphalt pavement (RCP) materials or recycled concrete materials (RCM) plus the aggregate base). Both pavement types can also be rubblized in place and compacted. This procedure is known as rubblize and roll for PCC pavements and full-depth reclamation for AC pavements. For AC pavement materials, there are also several other methods, including hot mix asphalt recycling, hot in-place recycling, and cold in-place recycling, all of which produce a bound product, which is beyond the scope of this manual.

Recycled Asphalt

The design requirements for RCP aggregates are essentially the same as natural aggregates. The strength of the material must be determined using the methods outlined in Chapter 5 and Section 7.3, and an assessment must be made of the drainage characteristics, as discussed in Section 7.2. With full-depth reclamation, all of the asphalt pavement sections and a predetemined amount of underlying materials are treated with recyling agents to produce a stabilized base course, and is well covered in FHWA-SA-98-042 (Kandhal, and Mallick, 1997) . The advantages of this process are establishing high production rate and maintaining the geometry of the pavement or shoulder reconstruction. The primary drawbacks are aggregate size, depth limitation and depth control, and need for specialized equipment. With the sizing, RAP can often only be effectively screened down to a maximum size of 50 mm (2 in.). If a significant amount of contaminated base course (*i.e.,* containing significant amount of fines) is removed with the asphalt, the hydraulic properties of the aggregate could also be poor.

Recycled Concrete

Again, the design requirements for RCM aggregates are essentially the same as natural aggregates. Recycled concrete has been used by a number of states as base materials since the 1980s. However, several states have identified three significant issues, including

- the formation of tufa (calcium deposits) clogging drains and filter materials;
- alkaline (high pH) run-off; and,
- freeze thaw degradation.

As a result, these states are now primarily using the recycled concrete, mixed with natural soils, as embankment fill.

7.7.2 Recycled Waste Materials

A number of recylced waste materials have been used in permanent construction, practically all of which where covered in Section 7.6.7 since they have a lighter weight than conventional aggregate. Other applications not reviewed in Section 7.6.7 include the use of recycled materials as a replacement for base materials (*e.g.,* slag and bottom ash) and, in some cases (*e.g.,* glass and tire shreds) drainage aggregate. As indicated in Section 7.6.7, the materials must be evaluated with respect to the same property requirements as the material they will replace. The pavement support value (*e.g.,* resilient modulus or CBR) should be determine based on lab tests reviewed in Chapter 5. Field trails using FWD tests to confirm the as constructed properties are also recommended. Durability is a critical issue with many of these materials, and, obviously, an assessment of environmental issues must be made.

7.8 REFERENCES

AASHTO (1993). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2004). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing (24rd ed.)*, American Association of State Highway and Transportation Officials, Washington, D.C.

American Society for Testing & Materials (2000). *ASTM Book of Standards, Vol. 4, Section 01*, Construction Materials: Cement; Lime; Gypsum, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards, Vol. 4, Section 02*, Construction Materials: Concrete and Aggregates, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards, Vol. 4, Section 08 and 09*, Construction Materials: Soils & Rocks, ASTM, Philadelphia, PA.

American Society for Testing & Materials (2000). *ASTM Book of Standards, Vol. 4, Section 03*, Construction Materials: Road and Paving Materials; Vehicle-Pavement Systems, ASTM, Philadelphia, PA.

Austroads (1998). Guide to Stabilisation in Roadworks, Austroads Publication No. AP-60/98, Austroads Inc.

Berg, R.R, Christopher, B.R. and Perkins, S.W. (2000). *Geosynthetic Reinforcement of the Aggregate Base/Subbase Courses of Pavement Structures, GMA White Paper II*, Geosynthetic Materials Association, Roseville, MN, USA, 176 p.

Bhatti, M.A., Barlow, J.A., and Stoner, J.W. (1996). "Modeling Damage to Rigid Pavements Caused by Subgrade Pumping," ASCE, *Journal of Transportation Engineering*, Vol. 122, No. 1, pp. 12-21.

Blanco, A.M., Bowders, J.J., Donahue, J.P. (2003). "Laboratory and In-Situ Hydraulic Conductivity of Pavement Bases in Missouri," presented at the Subsurface Drainage Session, TRB Annual Conference, TRB, Washington, D.C.

CALTRANS (1995) *Design Criteria for Portland Cement Concrete Pavements (PCCP).* Design Information Bulletin No. 80., California Department of Transportation, Sacramento, CA.

Carpenter, S. H., M. I. Darter, and B. J. Dempsey (1979) "Evaluation of Pavement Systems for Moisture-Accelerated Distress." *Transportation Research Record 705*, Transportation Research Board, Washington, D.C.

Cedergren, H.R. (1987) *Drainage of Highway and Airfield Pavements*, Robert E. Krieger Publishing Co. Inc. Malabar, FL

Cedergren, H. R. (1988) "Why All Important Pavements Should be Well Drained." *Transportation Research Record 1188*, Transportation Research Board, Washington, D.C.

Christopher, B.R. and V.C. McGuffey (1997). *Synthesis of Highway Practice 239: Pavement Subsurface Drainage Systems*, National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington, D.C.

Christopher, B.R. (2000) *Maintenance of Highway Edgedrains*, National Cooperative Highway Research Program, Synthesis of Highway Practice 285, Transportation Research Board, National Academy Press, Washington, D.C., 2000, 62 p.

Christopher, B.R., Zhao, A., Hayden, S.A. (2002) "Geocomposite for Pavement Subsurface Drainage," Presentation at TRB Committee A2KO6 on Subsurface Drainage. 2002 Annual Transportation Research Board Meeting, Washington, DC; also see: Christopher, B.R., Hayden, S.A., and Zhao, A., (2000). "Roadway Base and Subgrade Geocomposite Drainage Layers," *Testing and Performance of Geosynthetics in Subsurface Drainage, ASTM STP 1390*, J.S. Baldwin and L.D. Suits, Eds., American Society for Testing and Materials, West Conshohocken, PA.

Christory, J.P. (1990). "Assessment of PIARC Recommendations on the Combating of Pumping in Concrete Pavements," Proceedings of the Second International Workshop on the Design and the Evaluation of Concrete Pavements, Siguenza, Spain.

Coduto, D.P. (1999) *Geotechnical Engineering: Principles and Practices*, Prentice Hall, Upper Saddle River, New Jersey.

Daleiden, J. (1998) *Video Inspection of Highway Edgedrain Systems*, Final Report, Brent Rauhut Engineering.

Dawson A R and Hill A R. (1998) "Prediction and implication of water régimes in granular bases and sub-bases", Proc. Int. Symp. On Subdrainage in Roadway Pavements and Subgrades, Granada, Spain, pp 121-128.

De Beer, M. (1990). "Erodibility of Cementitious Subbase Layers in Flexible and Rigid Pavements," 2nd International Workshop on the Theoretical Design of Concrete Pavements, Siguenza, Spain.

Dempsey, B.J. (1982). "Laboratory and Field Studies of Channeling and Pumping," Transportation Research Board, *Transportation Research* Record 849, pp. 1-12.

Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J.G., and Berg, R.R. (2004). *Ground Improvement Methods*, Participant Notebook, NHI Course 132034, FHWA NHI-04-001, National Highway Institute, Federal Highway Administration, Washington, D.C., 1022 pp.

ERES Consultants, Inc. (1999). *Pavement Subsurface Drainage Design*, Participants Reference Manual for NHI Course Number 131026, National Highway Institute, Federal Highway Administration.

Fehsenfeld, F. M. (1988) *Performance of Open-Graded "Big Rock Mixes" in Tennessee and Indiana*, National Asphalt Pavement Association, Riverdale, MD.

FHWA HI-95-038 (1998 revision), *Geosynthetic Design and Construction Guidelines,* Participant Notebook, NHI Course No. 13213, FHWA Publication No. FHWA HI-95-038 (revised), authors: Holtz, R.D., Christopher, B.R., and Berg, R.R., Federal Highway Administration, Washington, DC.

FHWA (1992) *Drainable Pavement Systems—Participant Notebook.* (Demonstration Project 87). Publication No. FHWA-SA-92-008. Federal Highway Administration, Washington, DC.

FHWA (1994) *Drainable Pavement Systems — Instructor's Guide*. (Demonstration Project 87). Publication No. FHWA-SA-94-062. Office of Technology Applications and Office of Engineering, Federal Highway Administration, Washington, DC.

Hagen, M.G., and G.R. Cochran (1995) "Comparison of Pavement Drainage Systems, Final Report," Minnesota Department of Transportation, Minneapolis, MN.

Hindermann, W. L. (1968) *The Swing to Full-Depth*. Information Series No. 146, The Asphalt Institute, Lexington, KY.

Holtz, R.D., Christopher, B.R., and Berg, R.R. (1998 revision). *Geosynthetic Design and Construction Guidelines,* Participant Notebook, NHI Course No. 132013, FHWA Publication No. FHWA HI-95-038 (revised), Federal Highway Administration, Washington, D.C.

Holtz, R. D., and Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering*, Prentice-Hall, Inc., Englewood Cliffs, NJ.

Kandhal, P.S. and R.B. Mallick (1997) *Pavement Recycling Guidelines for State and Local Governments*. Federal Highway Administration Publication No. FHWA-SA-98-042, Washington, D.C.

Kaplar, C.W. (1974). "A Laboratory Freezing Test to Determine the Relative Frost Susceptibility of Soils," *Technical Report TR 250*, Cold Regions Research and Engineering Laboratory (CRREL), U.S. Army Corps of Engineers, 1974.

Koerner, R.M. (1998). *Designing with Geosynthetics*, 4th Edition, Upper Saddle River, NJ, Prentice Hall.

McCallister, L.D. and Petry, T.M. (1990). *Property Changes in Lime-Treated Expansive Clay Under Continuous Leaching*, U.S. Army Corps of Engineers, Washington, D.C., No. GL-90-17.

McKeen, R.G. (1976). Design and Construction of Airport Pavements on Expansive Soils. Report No. FAA-RD-76-66.

NCHRP 1-37A (2004). *Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, Final Report, National Cooperative Highway Research Program NCHRP Project 1-37A, Transportation Research Board, National Research Council, Washington, D.C.

NHI 131033 **Hot Mix Asphalt Pavement Evaluation and Rehabilitation**

NHI 131060A **Concrete Pavement Design Details and Construction Practices**

NHI 13126 (Currently 131026) (1999) *Pavement Subsurface Drainage Design*, Participants Reference Manual, prepared by ERES Consultants, Inc, National Highway Institute, Federal Highway Administration, Washington, D.C.

PIARC Technical Committee on Concrete Roads (1987). "Combating Concrete Pavement Slab Pumping."

Rollings, M.P., and Rollings, R.S. Jr. (1996). *Geotechnical Materials in Construction*, McGraw-Hill, New York, NY.

Sawyer, S. (1995) *Establishment of Underdrain Maintenance Procedure*. Report FHWA/OK95(04), Federal Highway Administration, Oklahoma City, OK.

Seed, H.B. and C.K. Chan (1959) "Structure and Strength Characteristics of Compacted Clays," *ASCE Journal of Soil Mechanics and Foundation Engineering*, Vol. 85, No. 5, pp. 87-128.

Snethen, D.R., L.D. Johnson, and D.M. Patrick (1977). *An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils*, Report No. FHWA-RD-77-94, Federal Highway Administration, Washington, D.C.

Sowers, G.F. (1979) *Introductory Soil Mechanics and Foundations: Geotechnical Engineering* (4th edition), MacMillan, New York, NY.

Steinberg. M.L. (1998). *Geomembranes and the Control of Expansive Soils in Construction*, McGraw Hill, Inc., New York, NY.

Thompson, M.R. (1970). *Soil Stabilization for Pavement Systems – State of the Art, Technical Report*, Construction Engineering Research Laboratory, Champaign, IL.

Ulmeyer, J.S., Pierce, L.M., Lovejoy, J.S., Gribner, M.R., Mahoney, J P, and Olson, G.D. (2002) "Design and Construction of Rock Cap Roadways: Case Study in Northeast Washington State" Prepared for presentation at the 2003 TRB Annual Meeting, Transportation Research Board, Washington D.C. Also see *Transportation Research Record 1821*.

U.S. Army Corps of Engineers (1992). *Engineering and Design Drainage Layers for Pavements*, Engineer Technical Letter 1110-3-435, U.S. Army Corps of Engineers, Washington, D.C.

USFS (1977) *Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads*, authors: Steward, J., Williamson, R. and Mohney, J., USDA, Forest Service, Portland, OR. Also reprinted as Report No FHWA-TS-78-205.

Witczak, M.W. (1972). "Relationships Between Physiographic Units and Highway Design Factors," Report 132, National Cooperative Highway Research Program, Highway Research Board, Washington, D.C.

Yu, H. T., K. D. Smith, M. I. Darter, J. Jiang, L. Khazanovich (1998a). *Performance of Concrete Pavements, Volume III—Improved Concrete Pavement Performance*. FHWA-RD-95-111, Federal Highway Administration, McLean, VA.

Yu, H. T., L. Khazanovich, S. P. Rao, M. I. Darter, H. Von Quintus (1998b) *Guidelines for Subsurface Drainage Based on Performance*. NCHRP 1-34 Final Report, National Cooperative Highway Research Program, Washington DC.