7.7 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 water table. 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, shoulders, and various other features in the subgrade 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.

7.7.1 Moisture Damage Acceleration

Excessive moisture within a pavement structure can adversely affect pavement performance. A pavement can be 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 (1995), 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).
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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 the effects of moisture, this approach is often costly and in many cases not feasible (e.g., may require replacing the subgrade). Drainage systems also add maintenance costs to the road. Therefore, it is necessary to consider drainage systems and sealing systems, for them to effectively perform over the life of the system. This 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 here. 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 through a drainage layer into a system of collector pipes. Generally, the actual process will be a combination of the two (AASHTO, 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

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

<table>
<thead>
<tr>
<th>Type</th>
<th>Moisture Problem</th>
<th>Climatic Problem</th>
<th>Structural Defect Begins in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Deformation</td>
<td>Excess Moisture</td>
<td>Frost Heave</td>
<td></td>
</tr>
<tr>
<td>Corrugation or Ripping</td>
<td>Slight</td>
<td>Moisture and Temperature</td>
<td>Unstable Mix</td>
</tr>
<tr>
<td>Rolling</td>
<td>Excess in Granular Layers or Subgrade</td>
<td>Moisture</td>
<td>Plastic Deformation, Slippage</td>
</tr>
<tr>
<td>Depression</td>
<td>Excess Moisture</td>
<td>Suction Materials</td>
<td>Settlement, Fill Material</td>
</tr>
<tr>
<td>Potholes</td>
<td>Excess Moisture</td>
<td>Moisture, Temperature</td>
<td>&lt; Strength, &gt; Moisture</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Type</th>
<th>Moisture Problem</th>
<th>Climatic Problem</th>
<th>Structural Defect Begins in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Defects</td>
<td>Possible</td>
<td>Freeze / Thaw Cycles</td>
<td>Mortar</td>
</tr>
<tr>
<td>Scaling</td>
<td>Yes</td>
<td>Freeze / Thaw Cycles</td>
<td>Chemical Influence</td>
</tr>
<tr>
<td>D-Cracking</td>
<td>Yes</td>
<td>Freeze / Thaw Cycles</td>
<td>Aggregate Expansion</td>
</tr>
<tr>
<td>Cracking</td>
<td>No</td>
<td>No</td>
<td>Rich Mortar</td>
</tr>
<tr>
<td>Surface Deformation</td>
<td>No</td>
<td>Temperature</td>
<td>Thermal Properties</td>
</tr>
<tr>
<td>Pumping and Erosion</td>
<td>Yes</td>
<td>Moisture</td>
<td>Inadequate Strength</td>
</tr>
<tr>
<td>Erosion</td>
<td>Yes</td>
<td>Moisture - Suction</td>
<td>Erosion - Settlement</td>
</tr>
<tr>
<td>Cutting / Warping</td>
<td>Yes</td>
<td>Moisture &amp; Temperature</td>
<td>Differential Moisture</td>
</tr>
<tr>
<td>Cracking</td>
<td>Yes</td>
<td>Moisture</td>
<td>Cracking Follows Erosion</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Climatic Condition</th>
<th>Greater than 12 million 20-yr design lane heavy trucks</th>
<th>Between 2.5 and 12 million 20-yr design lane heavy trucks</th>
<th>Less than 2.5 million 20-yr design lane heavy trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet Freeze</td>
<td>F</td>
<td>F</td>
<td>NR</td>
</tr>
<tr>
<td>Wet-No Freeze</td>
<td>R</td>
<td>R</td>
<td>F</td>
</tr>
<tr>
<td>Dry Freeze</td>
<td>F</td>
<td>F</td>
<td>NR</td>
</tr>
<tr>
<td>Dry-No Freeze</td>
<td>F</td>
<td>NR</td>
<td>NR</td>
</tr>
</tbody>
</table>

LEGEND:

- **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** = Annual precipitation > 508 mm (20 in.)
- **Freeze** = Annual freezing index > 83 °C-days (150 °F-days)
- **No Freeze** = Annual freezing index < 83 °C-days (150 °F-days)

<table>
<thead>
<tr>
<th>Climate</th>
<th>Annual precipitation &gt; 508 mm (20 in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Climate</td>
<td>&gt; 508 mm (20 in.)</td>
</tr>
<tr>
<td>Freeze</td>
<td>&gt; 83 °C-days (150 °F-days)</td>
</tr>
<tr>
<td>No Freeze</td>
<td>&lt; 83 °C-days (150 °F-days)</td>
</tr>
</tbody>
</table>

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
and the percent of time the pavement structure is near saturation. The influence of the drainage coefficient ($d$) 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 edgerains), 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 (10 ft/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 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). 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.

### 7.2.4 Types of Subsurface Drainage

In the past, pavement systems were designed without any subsurface drainage 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 is designed to remove 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 edgdrains with closely spaced outlets. An alternative to closely spaced outlets is dual drainage systems with parallel collector drains. An alternative to edgdrains 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.

### 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 edgdrains (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.

![Figure 7-3. Typical AC pavement with a daylighted base.](http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/07a.cfm)

### 7.2.6 Longitudinal Edgedrains

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

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

The most commonly used edgdrain 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 edgdrain that is often used in rehabilitation projects is a geocomposite drain in a sand filled trench with pipe outlets. Typical cross sections of edgdrains are illustrated in Figures 7-5 and 7-6.

![Figure 7-4. Typical AC pavement with pipe edgdrains (ERES, 1999).](http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/07a.cfm)
functioning properly, mostly due to improper construction (Daleiden 1998; Sawyer 1995), the mixed report is not surprising. In many others report significant improvement in pavement performance. Considering that only about 30% of all edgedrains in-service are 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.

pipes can lead to significant base erosion. As previously indicated, dense graded base with greater than 5% fines is not fines (fraction passing the 0.075 mm (No. 200 sieve)). Excessive fines can clog the drains, and the loss of fines through the

Outlet headwalls, typically precast concrete, are also an essential part of the edgedrain system to prevent displacement of the basin cleanout or repair. FHWA also recommends angled or radius outlet connections to facilitate clean out and video inspection. 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

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 clearing 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 edgeraind 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 edgerain 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 clearing 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 edgerain 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).
Figure 7-7. Dual pipe edgdrain systems showing alternate locations of the parallel collector pipe, either adjacent to or beneath the drain line (Christopher, 1998).

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 edgdrains 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.

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
Design of pavement drainage consists of determining: the infiltrated water with suitable outlets at reasonable outlet spacing or must be daylighted directly into the ditch. Finally, to ensure preventing any intermixing of the permeable base and separator layer. Permeable bases must also have pipe edgedrains to drain must have a separator layer capable of preventing the pumping of fines into the permeable base from underlying layers and from underlying layers and subgrade soils from infiltrating into the permeable base, thus maintaining the permeability and effective drainage paths through 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, and 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 mixes 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., 1999). 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.

7.2.10 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. Moisture-related damage to pavements has become more significant as traffic loadings have increased over the past 40 years. 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 expected.

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 expected.

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 to obtain 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 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 size and outlet spacing requirements.

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, 1998).
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 edgdrain).

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 edgdrain 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:

\[
t = T \times m \times 24
\]

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, T50, 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., edgdrain or daylighted section). First, the slope factor (\(S_1\)) must be calculated:

\[
S_1 = \frac{L_R S_R}{H}
\]

where,

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

Figure 7-9 is then entered with the \(S_1\), and the resulting T50 to be used in Eq. 7.1 is determined.

The “m” factor in Eq. 7.1 is determined by the equation:

\[
m = \frac{N_o L_R^2}{N_L L_R^2} = \frac{N_o L_R^2}{\psi}
\]

where,

- \(N_o = \text{the effective porosity of the drainage layer}\)
- \(k = \text{permeability of drainage layer in m/day (ft/day)}\)
- \(H = \text{thickness of drainage layer in m (ft)}\)
- \(\psi = \text{the transmissivity of the drainage layer in m}^2/\text{day (ft}^2\text{/day)}\)
- \(1 \text{ ft} = 0.3 \text{ m}\)

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

[Click here for text version of image](http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/07a.cfm)
The intrinsic factors that represent the drainage capabilities of drainage layer base are represented by the effective porosity ($N_e$) 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 volume 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 13125.

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)):

\[
S_r = (S^2 + S_e^2)^{1/2} = (0.01^2 + 0.02^2)^{1/2} = 0.02
\]

\[
L_e = W\left(1 + \frac{S}{S_e}\right)^{1/2} = 24\ ft\left[1 + \left(0.01/0.02\right)^2\right]^{1/2} = 26.8\ ft
\]

\[
S_1 = \left(\frac{L_e}{S_e}\right)/H = \left(26.8\ ft \times 0.022\right)/0.33\ ft = 1.8
\]

\[
m = \left(N_e L_n e\right)/\left(kH\right) = \left(0.25 \times \left(26.8\ ft\right)^2\right)/\left(1000\ ft/day \times 0.33\ ft\right) = 0.54\ days
\]

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

Therefore, $t = T + m = 0.16 + 0.54\ days = 0.64\ hrs/day = 2.1\ hrs$

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 edgewash 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, edgewash trench widths (if applicable), and cross-section geometry crowned or supereleved. The software can be downloaded directly from the FHWA Web page [link] and is included with the NCHRP 1-37A pavement design software.

### 7.2.14 Edgewash 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 clearing 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$^3$/day (300 ft$^3$/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 m$^3$/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 edgewash 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:

\[
Q = \frac{53.01}{n} \cdot D^{1.5} S^{1/2}
\]  

(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 edgewash 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:

\[
I = \frac{R}{I_r}
\]  

where,

- $I$ = Infiltration rate, in/hr
- $R$ = Design rainfall, in/hr
- $I_r$ = Infiltration ratio

The inlet size and spacing should be sufficient to accommodate the anticipated flow rate.

The FHWA recommends a minimum pipe diameter of 100 mm (4 in.) and a maximum outlet spacing of 75 m (250 ft) to facilitate clearing 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.
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In order to provide support for construction traffic, the geotextile must also satisfy survivability and endurance criteria. Additional subgrade or subbase layer. Also as with aggregate separator layers, the geotextile layers will have to satisfy filtration criteria. In 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).

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.

![Figure 7-10. Rainfall Intensity in in./hr for a 2-year, 1-hour Storm Event (FHWA, 1992).](http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/07a.cfm)

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

$$ q_L = q_i L_R $$

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.

### 7.2.16 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 must have 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 0.47 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.
- 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 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 (Holz 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.

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

which can be simplified to:

$$ q_i = 2 C R $$

where,

- $q_i$ = Pavement infiltration, ft/day/hr² 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, 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.
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 (No. 200 mm) sieve.

Step 2. Determine the permeability of the base or subbase $k_{\text{base/subbase}}$, 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 ($\psi$) requirements for the geotextile (after Holtz et al., 1998)

- $\text{AOS} \leq D_{85 \text{base/subbase}}$ (For woven geotextile)
- $\text{AOS} \leq 1.8 D_{85 \text{subgrade}}$ (For nonwoven geotextile)*
- $k_{\text{geotextile}} \geq k_{\text{base/subbase}}$
- $\psi \geq 0.1 \text{ sec}^{-1}$

* 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 $\leq D_{85 \text{subgrade}}$ 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 edgetrains). 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).

<table>
<thead>
<tr>
<th>Test</th>
<th>Test Method</th>
<th>Units</th>
<th>Geotextile Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt; 50%*</td>
<td>≥ 50%*</td>
</tr>
<tr>
<td>Grab Strength</td>
<td>ASTM D 4632</td>
<td>N 1400</td>
<td>900</td>
</tr>
<tr>
<td>Seam Strength</td>
<td>ASTM D 4632</td>
<td>N 1200</td>
<td>810</td>
</tr>
<tr>
<td>Tear Strength</td>
<td>ASTM D 4533</td>
<td>N 500</td>
<td>350</td>
</tr>
<tr>
<td>Puncture Strength</td>
<td>ASTM D 4833</td>
<td>N 500</td>
<td>350</td>
</tr>
<tr>
<td>Burst Strength</td>
<td>ASTM D 3786</td>
<td>kPa 3500</td>
<td>1700</td>
</tr>
</tbody>
</table>

*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)

Notes

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. Return to Text