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LONG-TERM PERFORMANCE OF GEOTEXTILE SEPARATORS
BUCODA TEST SITE—PHASE III

by

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16. ABSTRACT

This research was Phase III of field investigations carried out over 12 years at a test section in southwest Washington State in an effort to quantify the contribution of geotextile separators to the long-term performance of pavement sections. Five different geotextile separators, as well as a control (soil-only) section, were installed in a test section covering two lanes with different base course thicknesses on a low volume but heavily loaded rural highway west of Bucoda, Wash. Phase I evaluated the performance of the separators during construction. Phases II and III were conducted to evaluate the performance of the separators 5 and 12 years after construction, respectively.

Field and laboratory tests were conducted on the subgrade, granular base materials, and the geotextiles as part of the effort to correlate the performance of the pavement section to the presence of the geotextile separators. Falling weight deflectometer (FWD) testing was also performed at the site as part of the effort to quantify the performance of the pavement section.

The laboratory tests indicated that the geotextiles successfully performed their separation function over the 12-year period. However, the soil-only sections had a minimal amount of intermixing at the base course/subgrade interface, indicating that the separation benefits of geotextiles may not be realized under relatively thick pavement sections.

The FWD tests showed that the most significant increase in the subgrade moduli occurred in the first few months following construction. However, the soil-only sections exhibited behavior similar to the sections with geotextiles during the FWD testing, suggesting that for the relatively thick pavement sections, incorporation of geotextiles may not have provided a significant contribution to the overall performance of the section over the 12-year period. Some of the FWD results did suggest that geotextiles might contribute to an increase in the base course modulus over time.

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1. INTRODUCTION

1.1 PROBLEM STATEMENT

One of the oldest applications of geotextiles is their use as separators in roadways. In this application, the geotextile separates the pavement section from the underlying subgrade. However, in spite of many years of use, data to quantify the long-term performance of geotextile separators are lacking, especially their contribution to the long-term performance of the entire pavement section.

1.2 OBJECTIVES AND SCOPE OF WORK

To provide data to quantify the long-term performance of geotextile separators, a geotextile test section was constructed near Bucoda, Washington, in June of 1991.

1.2.1 Objectives

The objectives of the test section were as follows:

- evaluate the ability of different types of geotextile separators to stabilize a soft subgrade during construction
- examine the influence of different thicknesses of base course aggregate on geotextile performance during construction
- investigate the influence of different types of geotextiles on the long-term performance of the pavement section.

The first two objectives were achieved during construction of the test section, named Phase I, and a summary is presented in Chapter 2.

As part of the research, the Washington State Department of Transportation (WSDOT) periodically performed falling weight deflectometer (FWD) testing at the site.

In addition to the FWD testing, test pits were excavated 5 and 12 years after construction to observe the performance of the pavement section, perform in situ testing, and obtain aggregate base course, subgrade, and geotextile samples for laboratory testing.

The third objective was partially met in 1996 during the first series of test pit excavations, named Phase II. The focus of Phase II was the performance of the geotextile separators 5 years after installation, those results are also summarized in Chapter 2.

The second series of test pits, Phase III, were excavated in August of 2003, just over 12 years after installation of the geotextiles. The objective of this research was to investigate the influence of the different types of geotextiles on the long-term performance of the pavement section. In addition to the field investigation and laboratory testing, recently developed FWD backcalculation procedures were used to relate the long-term performance of the pavement section to the presence of the geotextiles.

1.2.2 Scope of Work

To accomplish the objectives, the project was divided into seven tasks:

Task 1 – Literature Review. The previous work conducted at the test section was reviewed (Savage, 1991; and Tsai and Savage, 1992; Tsai et el, 1993; Black, 1997; Black and Holtz, 1997, 1999) and is summarized in Chapter 2. Other research conducted for WSDOT related to the long-term performance of geotextile separators was also reviewed (Page, 1990; Holtz and Page, 1991; Metcalfe, 1993; Metcalfe and Holtz, 1994; Metcalfe, Holtz and Allen, 1995; and Holtz, 1996) and is summarized in Chapter 2. A literature review on research related to geotextile separators is also summarized in Chapter 2.

Task 2 – Review of FWD Results and Pavement Condition Surveys. WSDOT performed FWD testing along the test section before the reconstruction and has continued to perform FWD testing periodically over the 12 years since reconstruction. The Area Value and backcalculation procedures were used to analyze the historical FWD data. Pavement management surveys were also reviewed, and the pavement condition of the test section was surveyed during the field investigation.

Task 3 – Field Work Plans. The procedures used during previous test pit investigations in Washington State were reviewed. Specifically, the procedures used by Page (1990), Metcalfe (1993), and Black (1997), were reviewed in detail, and a detailed plan was developed to minimize the amount of time required for the test pit excavations (Task 4) while the quality of the research work was maintained.

Task 4 – Field Investigation. Test pit excavations were performed in each of the 12 areas of the test section. The asphalt pavement was removed, and samples were taken during removal of the base course. Geotextile and subgrade samples were also collected. More detailed descriptions of the test pit excavation procedures are presented in Chapter 3 and Appendix A.

Task 5 – Laboratory Tests. Moisture content and gradation tests were performed on the base course samples; these tests plus Atterberg limit and hydrometer tests were conducted on the subgrade samples. Permittivity tests and wide width tensile tests were performed on the geotextile samples. The laboratory testing procedures are discussed in Chapter 4.

Task 6 – Evaluation of Field and Laboratory Results. The results of both the field and laboratory testing programs were analyzed and evaluated in relation to the results developed during the literature review and the original objectives of the Bucoda Test Site research discussed above.

1.3 PROJECT DESCRIPTION

The test section was constructed in 1991 on SR 507, just west of Bucoda, Washington, as a change order to a reconstruction project along the route. The site is located in the southwestern part of the state, about 20 mi (32 km) south of Olympia. Figure 1.1 shows the project location. This section of SR 507 is oriented in a northwest-southeast direction, but the highway generally travels north-south, which is why the lanes are referred to as northbound and southbound.

This section of highway was selected for the study because it had been historically problematic, experiencing severe distress caused by logging truck traffic. Before reconstruction, the roadway surface contained significant ruts and alligator cracking, and the water table was within 1 to 2 ft (0.3 to 0.6 m) of the road surface during spring (Tsai et al., 1993). A review of records from a National Oceanic and Atmospheric Administration (NOAA) weather station about 8 mi from the site showed an average total precipitation of about 47 in., and average minimum and maximum temperatures of 42º and 62ºF (5º and 17ºC), respectively (Western Region Climate Center, 2004). Weather records indicated that the site rarely experienced any frost penetration.

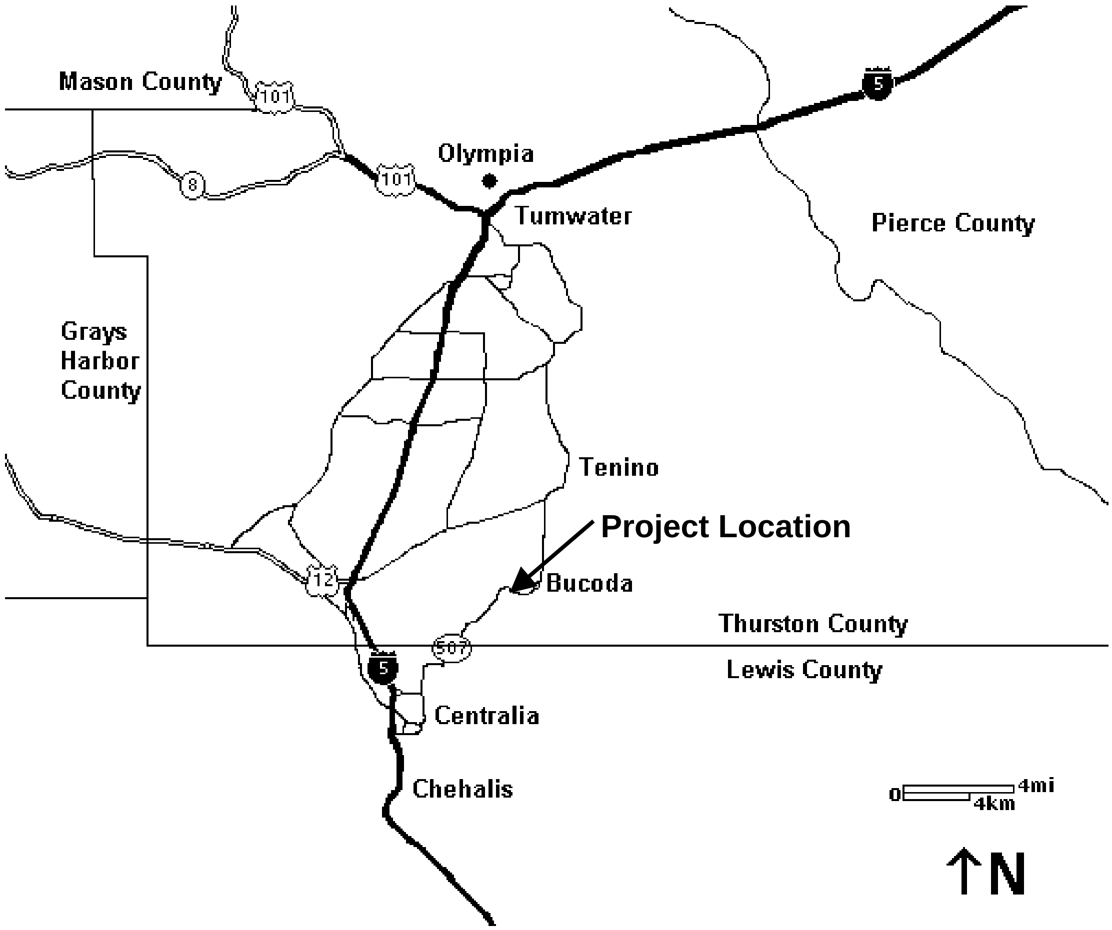

Figure 1.1. Project location map.

The test section was 150 ft (45.7 m) long and was divided into six 25-ft-long (7.6-m) sections in each travel lane. The project stationing was from 177+60 to 179+10 (ft). Each travel lane contained five different types of geotextile separators and one soil-only control section. The properties of the geotextiles are summarized in Table 1.1.

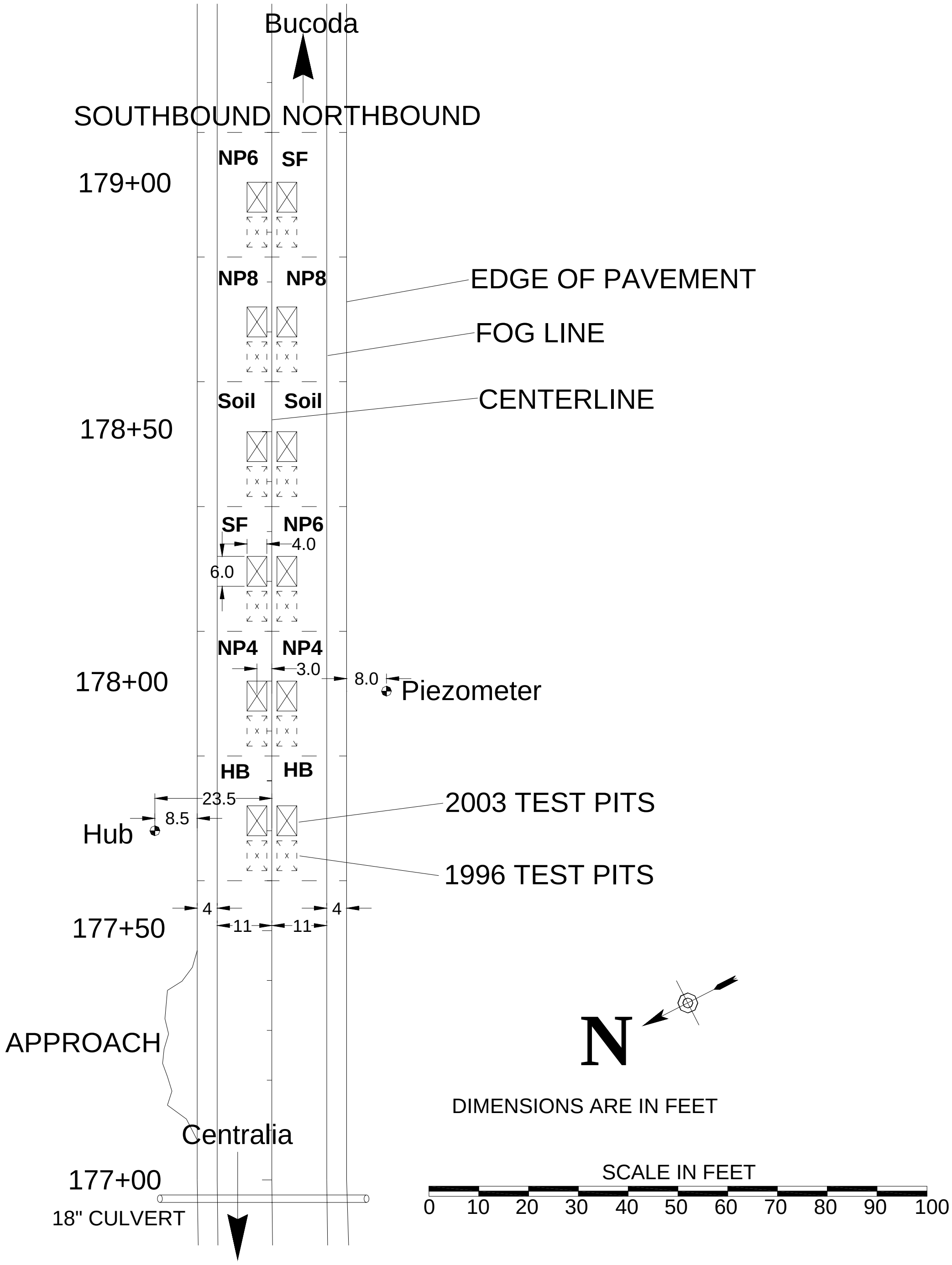

Figure 1.2. Test section layout.

Table 1.1. Summary of geotextile properties (after Black, 1997).

Symbol	Manufacturer	Structure	Polymer Type	Thickness, mils (mm)	Mass per Unit Area, oz/yd^2 (g/m^2)	Permittivity, sec^{-1}	AOS, US Std. Sieve No. (mm)
HB	Reemay Inc. 3401	NW	PP	17 (0.4)[91]	3.9 (132)	0.1	70 (0.21)
NP4	Polyfelt TS500	NW	PP	60 (1.5)*	4.5 (152)*	2.7	80-50 (0.18-0.30)
NP6	Polyfelt TS600	NW	PP	80 (2.0)*	6.3 (214)*	2.1	100-70 (0.15-0.21)
NP8	Polyfelt TS700	NW	PP	105 (2.6)*	8.3 (280)*	1.6	120-80 (0.125-0.18)
SF	Exxon GTF 300	W	PP	19.5 (0.5)*	7.1 (240)++	0.1*	50 (0.30)

Geotextile	Wide Width Strength /Elongation		Grab Tensile/ Elongation, lb (kN) /%	Puncture, lb (kN)	Trapezoidal Tear Strength, lb (kN)
	MD, lb/in. (kN/m) /%	XMD, lb/in. (kN/m) /%			
HB	35 (6.1) /45	40 (7.0) /50	130 (0.578) /60	40 (0.178)	60 (0.267)
NP4	50 (8.8) /80*	40 (7.0) /50*	110 (0.489) /50	60 (0.267)	50 (0.222)
NP6	70 (12.3) /95*	60 (10.5) /50*	150 (0.667) /50	75 (0.335)	70 (0.311)
NP8	90 (15.8) /95*	80 (14.0) /50*	205 (0.911) /50	100 (0.445)	85 (0.380)
SF	175 (30.6) /15[92]	175 (30.6) /15[92]	300 (1.334) /20	145 (0.645)	115 (0.511)

Geotextile properties are from Industrial Fabrics Association International (IFAI) (1990) unless noted otherwise. All values reported as minimum average roll values (MARV) unless noted by an asterisk (*) indicating typical values. HB = heat-bonded, NP = needle-punched, SF = slit film, NW = nonwoven, W= woven, PP = polypropylene, AOS = apparent opening size.
[91]Geotextile property from IFAI (1991)
[92]Geotextile property from IFAI (1992)
++From packaging label.

1.4 SITE TOPOGRAPHY AND GEOLOGY

This section of SR 507 traversed along the north end of the Skookumchuck River valley through the Bald Hills (see Figure 1.3). The roadway sloped downward to the southeast at about a 4.5 percent grade. The general topography was hilly, sloping downward to the south perpendicular to the test section toward the Skookumchuck River, which paralleled the roadway about 0.25 mi (0.4 km) south of the test section. The elevation of the roadway was about 70 ft (21 m) above the river elevation.

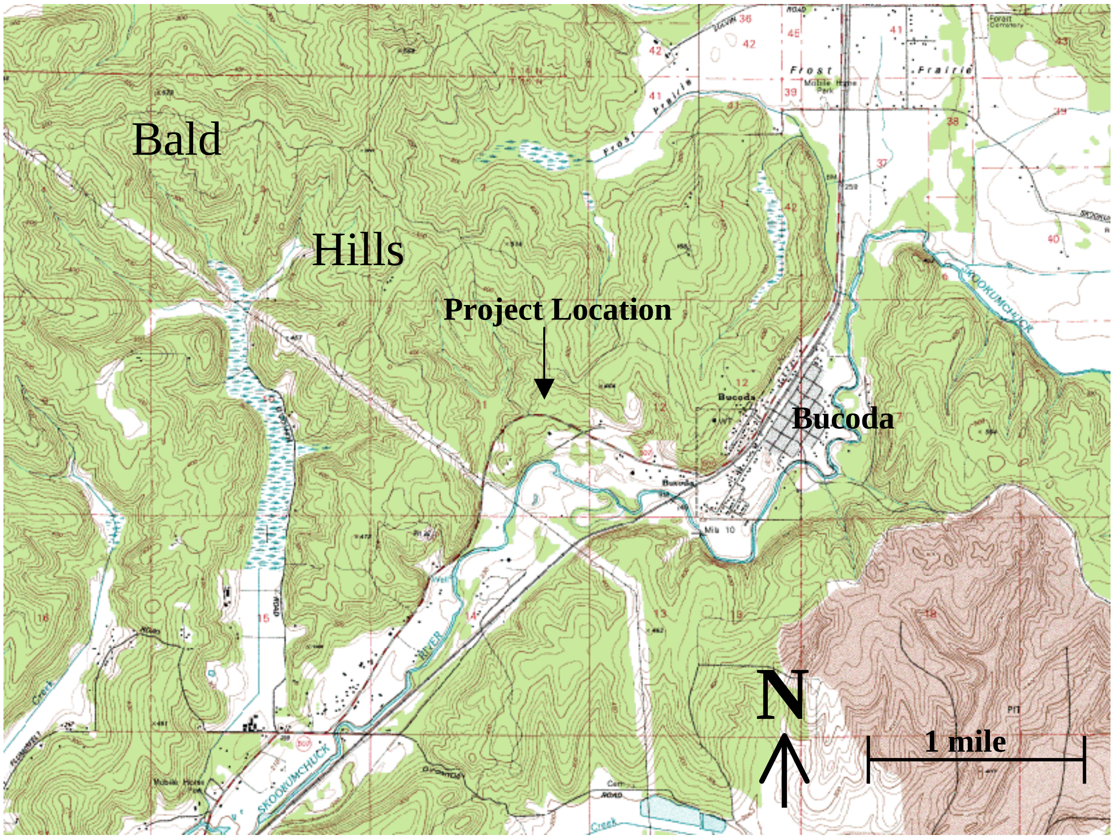

Figure 1.3. Topographic map of Bucoda and surrounding area (USGS, 1995).

During the last glaciation, the Bald Hills served as a barrier against which the ice mass terminated in many places (Wallace and Molenaar, 1961). The test section was located on a drainage path of the Vashon Glacier (Noble and Wallace, 1966). The hills near the test section were mapped as marine sedimentary rocks (Ts), and the lower areas as recessional outwash (Qvr) and alluvium (Qal) (Noble, 1966). The recessional outwash was described as glaciofluvial gravel and sand deposited during recession of the Vashon Glacier. The alluvium was described as predominately fine-grained floodplain deposits of detritus and peat.

2. LITERATURE REVIEW

This chapter summarizes recent research and publications on the topic of geotextile separators. Section 2.1 focuses on the separation/stabilization, filtration, drainage, reinforcement, survivability, and durability of geotextile separators. Section 2.2 summarizes two other investigations that evaluated the performance of geotextile separators funded by WSDOT in 1991 (eastern and central Washington) and 1994 (western Washington). Section 2.3 summarizes the results of the Phase I and Phase II investigations at the Bucoda test site conducted in 1991 and 1996, respectively. The current Federal Highway Administration (FHWA) design procedure is summarized in Section 2.4, and WSDOT design and specifications for geotextile separators are described in Section 2.5.

This review focuses on research published since the work of Black and Holtz (1997) and on similar investigations. The reader is referred to Black and Holtz (1997), Tsai (1995), and Metcalfe and Holtz (1994) for a more comprehensive review of earlier research. The purpose of this review was to compare the current research to other studies that evaluated the performance of geotextile separators in pavement sections.

2.1 RESULTS OF LITERATURE REVIEW

The primary function of geotextiles used in roadways is to separate the base course aggregate from the underlying subgrade. Secondary functions of geotextile separators may include filtration and drainage. Separators may also provide reinforcement to the pavement section. A key aspect of selecting a geotextile separator is determining the survivability criteria. The durability of the geotextile separator should also be considered during design and construction. The literature will be discussed in terms of these functions.

2.1.1 Separation and Stabilization

There is disagreement in the literature regarding the subgrade conditions under which the geotextile functions as a separator and as a stabilizer. The American Association of State Highway and Transportation Officials (AASHTO) Specification M 288 (2000) states that s*eparation* is appropriate where the subgrade is unsaturated and has a California Bearing Ratio (CBR) of ≥ 3, which correlates to a shear strength of greater than about 1900 psf (90 kPa). Where the subgrade consists of wet saturated conditions, and the CBR is between 1 and 3 (corresponding to shear strength of about 600 to 1900 psf [30 to 90 kPa]), M 288 states that *stabilization* is the geotextile function. Holtz et al. (1998) described separation as a geotextile function at CBR values of less than 3, and they were skeptical about the need for geotextiles in cases where the CBR > 3. Al-Qadi (2002) reported that intermixing at the base/subgrade interface has been noticed with CBR values of 8. Koerner (1998) listed separation as the primary function where the CBR value was greater than 8, speculating that the separation function of geotextiles placed on stiff subgrades could extend the pavement life by maintaining the base course thickness over the life of the pavement. Koerner (1998) suggested that a geotextile separator could double or triple the life of a pavement section.

To generalize, both the separation and stabilization functions prevent intermixing at the base course/subgrade interface. A geotextile functions as a stabilizer when placed below the initial lift of the base course where the subgrade is wet and saturated. Geotextiles used for stabilization have the secondary function of filtration and possibly reinforcement. However, a separator may also perform these functions. Therefore, from this point forward "separation" will be used to refer to both functions.

Before widespread use of separators, state department of transportation engineers commonly included sacrificial aggregate in their roadway design if the subgrade soils were soft (Al-Qadi, 2002). A relatively small amount of fines migrating into the base course from the underlying subgrade can reduce the strength and hydraulic conductivity of the base to that of the subgrade (Henry and Tingle, 2003). The additional percentage of fines that can result in significant strength loss is reported at values of between 10 and 20 percent in the literature. A separator prevents the base course aggregate from being pushed down into the underlying subgrade and the fine-grained subgrade soils from being pumped up into the aggregate. Penetration of aggregate into the underlying subgrade is due to localized bearing failures that commonly occur with wet, very soft, weak subgrades (Holtz et al., 1998). Dynamic loading conditions can cause fine-grained subgrade soils to be pumped up into the base course aggregate. Geotextiles provide subgrade drainage and dissipate excess pore pressure, which leads to consolidation and strength gain in the subgrade soils (Holtz et al., 1998; Black and Holtz, 1997, 1999).

Leu and Tasa (2001) presented observations from projects in northwestern Minnesota where geosynthetics were used to separate the base from poor subgrade soils. The projects had been in service for more than five years. They found that it was more cost effective to use stiffer geotextiles (with grab tensile strengths of 315 lb [1.4 kN] rather than 202 lb [0.9 kN]) because the additional benefits outweighed the small increase in material costs. Contractors were allowed to drive directly on the fabric to reduce installation costs (a stabilizing layer of aggregate was placed below the fabric). Long-term benefits included reduced maintenance, improved rutting resistance, improved spring breakup performance, and overall better performance, resulting in more economical maintenance programs.

2.1.2 Filtration

The geotextile separator acts as a filter and prevents fines in the subgrade from migrating into the overlying base course as a result of high pore water pressures induced by dynamic wheel loads (Holtz et al., 1998). Holtz et al. (1998) provided the following three filtration concepts used in the design process:

(1) If the largest opening in the geotextile is smaller than the largest aggregate particle placed above the geotextile, the aggregate will form a filter bridge and the soil will be retained.

(2) If the smallest openings in the geotextile are larger than the smallest particles in the subgrades soil, the small particles will pass through the geotextile and will not blind or clog it.

(3) The geotextile should have plenty of openings so that it will still have sufficient flow capacity if some blinding or clogging does occur.

Blinding occurs when the subgrade particles block the openings on the bottom surface of the geotextile. Clogging occurs when particles become trapped within the geotextile structure. Caking refers to particles deposited on the top surface of the geotextile, and it occurs when fines either filter down to the bottom of the base course or when the fines are deposited after migrating through the geotextile (Metcalfe, 1993). Figure 2.1 illustrates the concepts of blinding, clogging, and caking at the aggregate/geotextile/subgrade interface.

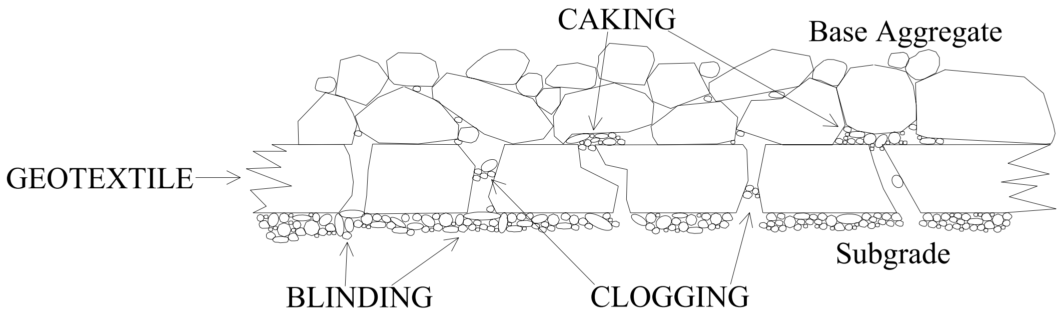

Figure 2.1. Illustration of blinding, clogging and caking (after Metcalfe et al., 1994).

Saathoff (1988) summarized the results of 81 field investigations conducted to evaluate the long-term filtration performance of geotextiles used in coastal engineering and shore protection, inland waterway construction, railroad track laying applications, and road construction. The geotextiles investigated consisted of 52 nonwoven and 29 woven fabrics that had been in service between 1 and 14 years. The ratio of the coefficients of permeability, *k*, for virgin geotextiles to soil-contaminated geotextiles averaged about 20 for nonwoven geotextiles and about 300 for woven geotextiles. The ratio of *k* of the soil-contaminated geotextiles to that of the filtered soil was greater than about 5 for the woven fabrics and about 2 for the nonwoven fabrics. The ratio of *k* of the virgin geotextiles to the soil was greater than about 80 for the nonwoven geotextiles and about 170 for the woven geotextiles.

Christopher and Valero (1999) documented the performance and durability of a woven monofilament polypropylene geotextile installed in 1969 in a filter application at the 79th Street Causeway Project in Miami, Florida. A similar study was conducted at the site in 1979 (Christopher, 1983). Permeability test results remained essentially unchanged from the time of installation in 1969. The percentage of open area also remained unchanged, except for a small decrease (about 10 percent) in the lower portion of the slope that was not subjected to tidal activity. This was attributed to the deposition of particles and incrustation. The paper is discussed further in section 2.1.6.

2.1.3 Drainage

Geotextile separators provide drainage by dissipating excess pore water pressure, which can lead to consolidation and strength in the subgrade, as discussed in Section 2.2.1.

Koerner (1998) defined geotextile drainage as, "the equilibrium soil-to-geotextile system that allows for adequate liquid flow with limited soil loss within the plane of the geotextile over a service lifetime compatible with the application under consideration." Koerner (1998) ranked the various manufacturing methods in the following order of **increasing** in-plane drainage capability:

- woven, slit film
- woven, monofilament
- nonwoven, heat-bonded
- nonwoven, resin-bonded – increasing with increasing weight and decreasing resin
- nonwoven, needle-punched – increasing with increasing weight.

Richardson (1997) reported a case in which a contractor had attempted to improve a submerged silt subgrade for a slab foundation by placing a woven slit film geotextile over the wet subgrade and placing 18 in. (450 mm) of clean gravel over the subgrade. The slit film fabric was not able to dissipate the increased pore pressures from the construction traffic, and severe rutting and pumping of the overlying gravel base occurred.

Alobaidi and Hoare (1998 and 1999) conducted laboratory cyclic loading tests to evaluate the mechanism of pumping at the base/subgrade interface in pavement sections without a geotextile and with a geotextile at the interface. On the basis of their results, they suggested that a gap forms during unloading when the entire load is transferred to the geotextile. They proposed that the rapid dissipation of cyclic pore pressures due to the presence of a geotextile causes erosion of the subgrade surface (pumping). Pumping occurs at the points where larger aggregates are in contact with the geotextile (LaFleur and Rollin, 1990). The aggregate forces the geotextile fibers apart, leaving an opening for fines to pass through. The mudcake that forms on top of the geotextile lowers the hydraulic conductivity of the system.

From laboratory in-plane hydraulic conductivity tests, Dembicki et al. (2002) found that for a given normal pressure and hydraulic gradient, the flow capacity is greater for the geotextile that has a greater mass per area. As the normal pressure increases, the flow capacity decreases.

2.1.4 Reinforcement

Geotextiles may provide reinforcement through lateral restraint, increased bearing capacity, and membrane tension support (Holtz et al., 1998). Soft saturated subgrades have relatively little resistance to the lateral forces caused by wheel loads that push the aggregate outward. Geotextiles with good interface friction and tensile strength will increase the lateral restraint of the aggregate. Geotextiles increase bearing capacity by providing a barrier that forces the failure surface along a path of higher strength up in the aggregate or along the interface between the subgrade and geotextile or the geotextile and the aggregate base. Membrane tension support occurs after the aggregate and underlying subgrade have rutted, and the resulting tension in the geosynthetic reduces the stress on the subgrade. This is only likely to occur after significant rutting has developed during construction or on gravel roads with a thin aggregate section.

Tsai and Holtz (1998) conducted 19 plate load tests using three different base course thicknesses, six geotextile separators and a non-reinforced section, two subgrade soils, and four different subgrade strengths (CBR between 0.5 and 2.0). Rut measurements were taken for all of the tests. Regression analyses were used to modify the Giroud and Noiray design formula to calculate rut depths based on aggregate thickness, number of equivalent single axle loads (ESALs), the undrained shear strength of the subgrade, and the type and weight of the geotextile separators. The rut prediction formula correlated well with the laboratory test results, and the formula generally over-predicted the results that were obtained from field tests.

Poor interface shear strengths can lead to lateral spreading at the base/geotextile or geotextile/subgrade interface. Bearden and Labuz (1998) presented results of large-scale direct shear tests on soil-fabric-aggregate systems. Three polypropylene geotextiles were tested: a lightweight slit film woven (4.5 oz/yd^2 [150 g/m^2]), a heavyweight woven (8 oz/yd^2 [267 g/m^2]), and a heavyweight nonwoven (10.2 oz/yd^2 [340 g/m^2]). The test results indicated that the nonwoven geotextile had interface friction angles that were about 20 percent higher on average than those of either the woven geotextiles. The tests were conducted at normal stresses of 1090 psf (52 kPa), 1610 psf (77 kPa), and 2150 psf (103 kPa). The stresses were selected to model a typical dual-wheel load on an unpaved road. The woven geotextiles had interface friction angles of between 32 and 38 deg., whereas the nonwoven geotextile had interface friction angles of between 41 and 53 deg.

Labuz and Bearden (1999, 2000) conducted plate model tests to simulate an unpaved road reinforced with a geotextile subjected to dynamic loading. A lightweight slit film woven (4.5 oz/yd^2 [150 g/m^2]) and a heavyweight nonwoven (10.2 oz/yd^2 [340 g/m^2]) geotextile were placed over a silty clay subgrade with shear strengths of between about 1000 and 2000 psf (50 and 100 kPa). The subgrade and geotextile were covered with 4 in., 6 in., or 8 in. (100 mm, 150 mm, or 200 mm) of gravel base course. The test results indicated that in terms of rut depths, the nonwoven reinforced system with 4 in. (100 mm) of gravel was equivalent to the slit film reinforced system with 6 in. (150 mm) of gravel and the unreinforced (soil-only) system with 8 in. (200 mm) of gravel. The results also suggested that reinforced roads could be designed (in terms of rut depths) using bearing capacity factors two times those of unreinforced roads. These findings agreed with the bearing capacity factors recommended in the FHWA Geosynthetic Design Manual (Holtz et al., 1998) that were originally developed for the U.S. Forest Service (Steward et al., 1977).

The Geosynthetic Material Association prepared the *GMA White Paper II* (Berg et al., 2000) to assist AASHTO in developing a specification for geosynthetic reinforcement of the aggregate base course in pavement sections. The three methods for quantifying the contribution of the geosynthetic to the pavement system are traffic benefit ratio (TBR), base course reduction ratio (BCR), and layer coefficient ratio (LCR). Use of geosynthetic reinforcement in pavement sections can lead to substantial life cycle cost savings by either extending the life of the pavement section or reducing the required structural section over an equivalent service life.

In situations where the subgrade is sufficiently stiff to prevent intermixing at the base/subgrade interface, geosynthetic (geotextile or geogrid) reinforcement may be incorporated into the pavement section to extend the life of the pavement (TBR) or reduce the thickness of the base course (BCR). TBR is defined as the ratio of permissible traffic loads for a pavement section with reinforcement in comparison to the same section without reinforcement. BCR is defined as the percentage of reduction in base course thickness for the equivalent service life.

Perkins and Ismeik (1997) reviewed earlier literature pertaining to the use of geotextiles as reinforcement in the base layer. The literature indicated common TBR values of between 3 and 10 and reductions in base thickness of between 22 and 55 percent. The literature indicated that improvement was seen at all levels of rut depths. In situations where the subgrade was soft enough for intermixing to occur, geogrids were found ineffective and geotextiles were superior. At the other extreme, where the subgrade was stiff enough that no intermixing occurred at the base/subgrade interface, geogrids generally provided better improvement to sections than geotextiles. The authors attributed the success of the geogrids to their ability to prevent spreading of the base course by interlocking with the base aggregate.

Perkins (1996) documented the ability to instrument test sections constructed to evaluate geosynthetic reinforcement of flexible pavements. A pavement loading system constructed at Montana State University used a circular plate to simulate traffic loading on pavement sections constructed inside a large concrete box and was instrumented with stress and strain cells (Perkins, 1999). Twenty-one tests were reported. The variables included geosynthetic type (two geogrids and one woven geotextile were evaluated), subgrade type and strength, locations of the geosynthetic within the base layer, and the base course layer thickness. The results showed a significant reinforcement effect that was attributed to the shear-resisting interface provided by the geosynthetics. On the basis of the laboratory traffic loading test results, Perkins (2001a, 2001b, and 2001c) developed 3-D finite element models to describe the rutting behavior of the sections when subjected to cyclic loads. The models were used to develop design equations that defined the reinforcement benefit in terms of TBR and/or BCR. The models were validated with the results of the test section and other results available in the literature. A Microsoft Excel spreadsheet was created to perform the design calculations and is available from the Montana Department of Transportation website at http://www.mdt.state.mt.us/departments/researchmgmt/grfp/grfp.html.

Perkins (2002) conducted additional tests to better define the influence of the type of traffic loading and the type of geosynthetic reinforcement for the previously developed finite element models. The test results indicated that the design calculations were conservative, except for cases where the base layer was relatively thin (less than 6 in. [150 mm]).

2.1.5 Survivability

Survivability is the ability of the geotextile to withstand construction conditions and perform its intended function. The most critical loading conditions on geotextile separators typically occur during construction. AASHTO Specification M 288 (2000) covers geotextiles used for separation in highway applications. It specifies minimum grab tensile strengths, sewn seam strengths, tear strengths, and puncture strengths based on the severity of installation conditions. The American Society for Testing and Materials (ASTM) has developed Standard Test Methods for conducting the above tests.

Factors that have the greatest influence on geotextile survivability include subgrade conditions (the amount of preparation and presence of any debris that may damage the geotextile), initial lift thickness, cover material (particle size and angularity), and loads from construction equipment. Tensile strength is mobilized in the geotextile when an upper piece of aggregate pushes downward on two lower pieces that are interlocked with the geotextile, forcing it apart in tension. A minimal amount of subgrade preparation is generally performed when geotextile separators are used. Therefore, the subgrade may contain rocks, sticks, tree stumps, and other debris that may puncture the geotextile after the base aggregate is placed and traffic loads are applied. Geotextile separators must also withstand impact (tear) resistance during placement of the base course aggregate. Aggregate dropped from a significant height directly onto the geotextile may puncture or tear the geotextile. This concern is easily avoided by following good construction practices, e.g., not allowing aggregate to be dumped directly onto the geotextile.

Watn et al. (1998) conducted a series of laboratory index tests and a field trial on nonwoven geotextiles in an attempt to correlate index properties used to classify geotextiles in Norway with resistance against construction damage. They found that the sum of the measured hole-diameters from the field samples correlated best to the mass per unit area and the laboratory failure strength; similarly, Naughton and Kempton (2002) found good correlation between mass per unit area and retained strength on the basis of laboratory simulations and field trials. They found that installation damage caused all of the geotextiles tested with a mass per unit area of less than 5.2 oz/yd^2 (175 g/m^2) to lose 50 percent of their initial tensile strength.

Previous investigations that were conducted in Washington to evaluate the survivability of geotextile separators are described in sections 2.2 and 2.3. Geotextile strength property requirements for survivability are presented in section 2.4.

2.1.6 Durability

Possible sources of geotextile degradation include exposure to ultraviolet light, changes in temperature, oxidation, hydrolysis, chemical attack, radioactive exposure, biological attack, aging, ozone attack, and rodent or termite attack (Koerner, 1998). Durability is a function of the polymer type. Studies have shown that oxidation is the primary cause of aging degradation in polypropylene and polyethylene geosynthetics, while hydrolysis is the primary cause of aging degradation in polyester (DiMillio, 2001).

The intensity and spectrum of sunlight, along with geographic location, temperature, cloud cover, wind, moisture, and atmospheric pollution are all factors that affect the amount of ultraviolet degradation (Koerner, 1998). The ASTM has developed several test methods to evaluate the effect of exposure to ultraviolet light on geotextiles. Ultraviolet degradation can be minimized by covering geotextiles stored on job sites and limiting the amount of exposure during placement.

Cazzuffi and Sacchetti (1999) conducted tensile creep tests on three geosynthetics to evaluate the effects of temperature on creep behavior. The geosynthetics included a high density polyethylene (HDPE) extruded geogrid, a polyester woven geogrid and a polypropylene/polyester woven/nonwoven geotextile. The tests were conducted at temperatures of 50°F, 68°F, and 104°F (10ºC, 20ºC, and 40ºC). The results indicated that in general the creep strain increased with temperature for all of the geosynthetics. The HDPE extruded geogrid was most affected by an increase in temperature, and the polyester woven geogrid showed small effects due to changes in room temperature.

Elias (2001) concluded that oxidation-induced strength loss did not occur in polypropylene geotextiles installed for periods of less than 20 years. He cited an earlier study (Elias et al., 1999) that showed that the typical antioxidants used in commercial geosynthetics protected polyolefin geosynthetics from oxidative related strength loss for 25 years to more than 100 years. Elias et al. (1999) found that the service life of polyolefin geosynthetics depends on the amount and type of antioxidant, as well as the type of fiber, manufacturing process, and oxygen concentration of the in situ soil. The molecular weight and pH of the in situ conditions were found to have the largest effect on the rate of degradation of polyester geosynthetics.

Polypropylene-based geosynthetics are subject to oxidation in soil or by oxygen dissolved in soil water, and polyesters are subject to hydrolysis by different aqueous solutions present in soil (Salman et al., 1997). Research by Salman et al. on polypropylene and polyester geotextiles concluded that polypropylene materials with effective antioxidants are not likely subject to thermo-oxidation during their design service life. For polyester materials, the dominant mechanism of hydrolytic reaction in neutral and acidic environments is molecular chain scission, resulting in a decrease in molecular weight and tensile strength. In an alkaline environment, reduction in molecular weight and fiber surface erosion contribute most to tensile strength loss.

The results of field retrievals and laboratory tests indicated that in neutral saturated soil conditions, a strength loss of between 0.25 and 0.5 percent per year should be anticipated for polyester geosynthetics (Elias, 2001). Polyester fibers with a molecular weight of above 25,000 and 30 or fewer carboxyl end groups are hydrolysis resistant (TC Mirafi, 2000). Limited data have suggested that installation damage may increase hydrolysis and antioxidant consumption rates (Elias, 2001).

Geosynthetics can be tested in accordance with ASTM D 543 to evaluate the effect of exposure to different reagents. Information can usually be obtained from manufacturers that have conducted tests on their products with various reagents (Koerner, 1998).

Analyses were conducted to demonstrate that polyethylene materials would perform their intended function of containment in a low-level radioactive waste disposal landfill over a 500-year design period (Badu-Tweneboah et al., 1999). They concluded that radioactivity was the only potential source of energy that could have an impact on the performance of the containers, but the analysis showed that the effects were insignificant during the 500-year design period.

A five-year study was conducted at the Orange County Landfill in Florida to assess the biological clogging of geotextile filters (Mackey and Koerner, 1999). Two nonwoven geotextiles and two woven geotextiles were evaluated. Only limited clogging was found in the nonwoven geotextiles, and their use was recommended. A woven geotextile with a 7 percent open area exhibited a relatively rapid decrease in permittivity; consequently, it was not recommended for use in landfills. Mixed results were obtained for a woven geotextile with a 32 percent open area; thus the authors were unable to draw any conclusions regarding its performance.

Christopher and Valero (1999) presented the results of a field investigation of a filtration geotextile that had been in service for more than 30 years. The geotextile was a woven monofilament polypropylene geotextile installed in 1969 in a filtration application at the 79th Street Causeway Project in Miami, Florida. A similar investigation was conducted at the site in 1979 (Christopher, 1983). Three samples were obtained from three sections at the site. Section 1 was uncovered and thus exposed to UV rays, temperature extremes, and air. Section 2 was covered by soil in a tidal location where it was cyclically exposed to water and air. Section 3 was below the water level for the entire service life. Scanning electron microscope photos were used to evaluate aging due to oxidation. The photos showed no signs of significant oxidation. Tensile strength tests indicated that no apparent loss in strength occurred in the 20 years since the first investigation, except in Section 1. This was an indication that aging degradation occurred in the uncovered section. Some reduction in elongation was observed in the tensile tests, indicating that some aging may be occurring. The authors concluded that the strength loss in the past 30 years was primarily, if not entirely, due to the damage sustained during the original construction. Aging did not appear to be occurring in locations where the geotextile was properly installed and covered.

No literature was found citing cases where ozone attack or rodent/termite attack had been a problem in a geotextile application; however, these issues should be considered when exposure is expected.

2.1.7 Geotextile Separator Studies

Nine different types of geotextile separators were exhumed from a Swedish test road 5 and 10 years after installation (Brorsson and Eriksson, 1986). The separators consisted of five polyester and four polypropylene geotextiles. One of the polyester samples was woven; the rest of the geotextiles were nonwoven. Five were needle-punched, one was heat-bonded, and one was needle-punched and heat-bonded. In general, the polyester geotextiles experienced more strength loss over the test period. Although the geotextiles experienced some strength loss that varied from little to about 50 percent, the loss did not appear to affect the separation performance of the geotextile. Grain size distribution tests indicated that few or no fines migrated through the geotextile over the 10-year period. The authors stated that during construction, “it was impossible to walk on the subgrade material without sinking down half the height of one’s Wellingtons.” Conversely, during the test pit excavations, they noted that the subgrade was well consolidated, and hand vane shear strengths varied from between 1350 and 2000 psf (65 and 95 kPa).

The Virginia Department of Transportation constructed a test section in 1994 to evaluate the performance of geosynthetics between a granular base course and a fine-grained subgrade (Al-Qadi et al., 1997; Al-Qadi, 1998, 2002). The study included laboratory tests modeling a dynamically loaded secondary road. In the laboratory tests, two to three times more load cycles were required to cause failure when a geotextile was used as a separator than for an unreinforced section. On the basis of FWD test results from the field investigations, the researchers concluded that sections stabilized with geotextiles enhanced the contribution of the subgrade to the pavement section by increasing the subgrade resilient modulus more than 20 percent (Al-Qadi et al., 1997). Field testing, including ground penetrating radar, rut-depth measurements, and FWD testing, indicated that the geosynthetic reinforced sections experienced less distress than the soil-only control sections (Al-Qadi, 2002). The geosynthetic-reinforced sections were able to carry about 80 to 130 percent more ESALs than the unreinforced sections. Using the results of the field and laboratory tests, a design curve was developed that correlated design ESALs for a section with a geotextile to a section with no geotextile (Al-Qadi, 2002). A life cycle cost analysis can be performed using the chart to quantify the cost benefits of the geosynthetics.

Tsai and Holtz (1997) presented the results of cyclic plate load tests conducted in the laboratory to evaluate the performance of various types of geotextile separators with three thicknesses of base aggregate on silty and clayey subgrade soils with CBRs ranging from 0.5 to 7. The separators investigated included heat-bonded nonwovens, needlepunched nonwovens, a woven slit film woven, a graded granular filter, and a geomembrane. The objectives were to evaluate the geotextiles for survivability, susceptibility to fines migration, and subgrade pore pressure dissipation. The researchers found that the geotextile modulus had no significant influence on the magnitude of rut depth. In all of the tests, the pore pressures in the subgrade initially increased, and then they decreased with time. On the basis of the test results, the authors concluded that a geotextile that survives installation and service life appears to increase the bearing capacity of the soil. The geomembrane (used to model a completely clogged separator) resulted in the longest persisting elevated pore pressures, which led to a reduction in the subgrade modulus and deeper rut depths. The peak pore pressures did not appear to be influenced by the type of geotextile. The geotextiles that survived the dynamic loading appeared to adequately prevent fines migration.

Hayden et al. (1998) reported on the design, construction, instrumentation, and monitoring of a test section along US Route 1A located in the towns of Frankfort and Winterport, Maine. The 1.9-mile long (3-km) test section had historically poor pavement performance, which was attributed to the silty (AASHTO A-6 soil classification) subgrade soil conditions. Four types of geosynthetics were used to evaluate reinforcement (geogrid and geotextile), separation/stabilization (geotextile), and drainage and frost heave mitigation (geocomposite drainage net). Two soil-only control sections were constructed adjacent to the reinforcement and drainage test sections. An additional 24 in. (600 mm) of aggregate were placed in the control sections. The test section was constructed between May and November of 1997, which the authors noted was an unusually dry season. However, several problems related to soft, saturated soils were encountered. The separation geotextile was the easiest to work with during construction, but it became difficult to handle on windy days. It was difficult to get the geogrid to lay flat on the subgrade. The reinforcement geotextile was easy to install, but sewing of the seams and tensioning proved to be time consuming.

Hayden et al. (1999) presented the results from the first year of monitoring at the U.S. Route 1A project in Maine. Data obtained from the geogrid and geotextile reinforcement sections indicated that the greatest forces developed during placement of the initial aggregate lift. The forces developed were only a fraction of the ultimate tensile strengths of the geosynthetics. Forces induced in the geotextiles were highly variable over small distances, probably because of wrinkles in the geotextile, subgrade strength, and rutting from construction traffic. FWD tests were conducted before reconstruction and in April of 1998, following the spring thaw. Back-calculations indicated that the structural numbers for the test sections had increased, with the control sections showing the highest increase. This was attributed to the additional aggregate placed in the control sections during construction. Because of the short period of evaluation, no conclusions could be made from the FWD data.

Koerner (1997 and 1998) reported on a nationwide geotextile separation study to evaluate the performance of geotextile separators used over stiff subgrades. The goals of the study were to create a large database of sites with all types of traffic, subgrade, climatic and environmental conditions and to quantify the performance of the sections over their service lives. The Bucoda test site was one of 10 sites listed at the time the 2000 article was published. No results were presented in the article.

Suits and Koerner (2001) reported the results of an ongoing investigation in northern New York State where five different geotextile separators were installed in a rural two-lane road in late 1997. The geotextiles installed included four nonwovens and one woven. The subgrade soils were described as brown till (hardpan) and wet, spongy, gray sandy silt. A soil-only control section was also included in the test area. Site visits and falling weight deflectometer (FWD) tests were conducted at least annually in the first three years following construction. The results of the testing showed a decrease in the subgrade resilient modulus of between 5 and 48 percent in the geotextile sections and of 37 percent in the soil-only section between October of 1998 and July of 2000.

2.2 WSDOT GEOTEXTILE SEPARATOR STUDIES

Before the Bucoda investigation, WSDOT conducted a two-phase study to evaluate the performance of geotextile separators. Phase I (Holtz and Page, 1991; Page, 1990; and Holtz, 1996) was conducted in eastern and central Washington and Phase II (Metcalfe and Holtz, 1994; Metcalfe, 1993; Metcalfe et al., 1995; and Holtz, 1996) was conducted in western Washington. The following sections summarize the results of those investigations.

2.2.1 Phase I - Eastern and Central Washington

Holtz and Page (1991) evaluated the performance of both nonwoven and woven geotextiles at eight locations in eastern and central Washington. The geotextiles had been in service between 1 and 7 years at the time of the explorations. The geotextile separators included five slit film wovens, two needle-punched nonwovens, and one heat-bonded nonwoven. Samples of the base course, subgrade, and geotextile were retrieved for testing in the laboratory. Index tests (grain size distributions and moisture contents) were performed on the soil samples and the geotextile samples were tested for strength (grab tensile, trapezoidal tear, puncture resistance, and Mullen burst tests) and permittivity.

At three of the sites, the geotextiles were not installed directly against the subgrade as specified in the WSDOT Design Manual (see section 2.5). The damage sustained by the geotextiles due to the aggregate type and installations varied considerably, and although most of the geotextiles sustained some damage, it did not appear to affect their performance as separators. The lightweight geotextile (3.5 oz/yd^2 [120 g/m^2]) was severely damaged. No damage was observed in the heavyweight geotextile (8 oz/yd^2 [270 g/m^2]) geotextile placed in a high survivability installation. Puncture holes were observed in the slit film woven geotextiles that had been placed on subgrades with gravel-size particles. However, even at the locations where the geotextiles had sustained moderate damage, the pavements appeared to be performing adequately. The slit film woven geotextile had the highest increase in permittivity from the as-retrieved conditions to the washed condition.

Page drew the following conclusions: Lightweight geotextiles should not be used for separation applications. Use of relatively heavyweight geotextiles that meet high survivability criteria minimized installation damage, but at an increased cost. Geotextiles that sustained moderate installation damage still performed the separation function adequately, and the performance of the pavement section was not affected by the damage. Because of limited data, it was difficult to draw conclusions regarding the permeability of the geotextiles. However, visual observations and permeability tests suggested that the slit film woven geotextiles blinded more readily than nonwoven geotextiles. In spite of this, the blinding did not appear to affect the performance of the pavement section. The use of geotextiles also successfully expedited the construction process.

Page recommended that WSDOT specifications allow the field engineer to determine whether a geotextile is needed at the time of construction or that geotextiles be used only by change order. He also recommended that only nonwoven geotextile separators be used in situations where the subgrade consists of a clayey silt or sandy silt.

2.2.2 Phase II - Western Washington

Metcalfe and Holtz (1994) evaluated the survivability (short-term) and filtration/drainage (long-term) performance of 14 geotextile separators at sites in western

Washington. The geotextiles investigated included six slit film wovens, six needle-punched nonwovens, and two heat-bonded nonwovens. Samples of the base course, subgrade, and geotextile were retrieved for testing in the laboratory. Index testing (moisture content, grain size distribution and Atterberg limits) were performed on the soil samples and the geotextile samples were tested for strength (grab and wide width tensile tests) and permittivity.

Considerable variation in geotextile damage was observed. It appeared that the aggregate type had more influence on damage than the initial lift thickness. All of the geotextiles that were installed under an angular base course sustained some damage, including two heavier weight geotextiles (7 oz/yd^2 [230 g/m^2] and 6 oz/yd^2 [200 g/m^2]). Lighter weight geotextiles (4 oz/yd^2 [136 g/m^2]) onto which were placed rounded to subrounded aggregate sustained minor to no damage. There was evidence of in-service damage in one of the needle-punched nonwoven samples. The slit film wovens and needle-punched nonwovens experienced similar strength reductions. The heat-bonded nonwovens had higher strength reductions, but they were recovered from some of the higher installation survivability conditions.

The percentages of increases in permittivity after washing were similar for the slit film wovens and needle-punched nonwovens. While the heat-bonded nonwovens had the highest permittivity increase after being washed, observations suggested that the slit film wovens sustained more blinding than the other geotextiles. Caked fines and iron staining were also present on some of the slit film wovens. Only one of the slit film wovens met the calculated filtration requirements, and most did not meet WSDOT's permeability specification. The grain size distribution test results did not show any evidence of fines migration.

The pavement at one of the sites had evidence of premature failure, but it was not attributed to the performance of the geotextile. Otherwise, all of the pavements were in good condition, and it appeared that damage sustained by the geotextiles was not detrimental to the performance of the pavement sections.

The following conclusions were drawn from the Phase II investigation (Metcalfe et al., 1995; and Holtz, 1996). All of the geotextiles performed the separation function adequately. Blinding and caking appeared to be reducing the drainage capabilities of the slit film woven geotextiles. Therefore, slit film wovens should not be used at sites with soft, silty soils where the separator may be subjected to high groundwater conditions. The needle-punched nonwoven geotextiles had the best drainage performance. The lack of fines migration suggested that larger apparent opening size (AOS) values than those usually specified may be allowable. The consolidation and strength gain observed in the subgrades suggested that the long-term drainage and filtration performance of the geotextile separator may not be as important as the separation provided between the aggregate and subgrade.

2.3 SUMMARY OF PREVIOUS BUCODA TEST SITE INVESTIGATIONS

The Phase I investigation was conducted in 1991 during construction. Phase II was conducted in 1996, five years after construction.

2.3.1 Bucoda Test Site - Phase I

The Phase I research was described by Savage (1991), Tsai and Savage (1992), and Tsai et al. (1993). The objectives of Phase I were to compare the ability of five different geotextiles to stabilize a soft subgrade during construction and to examine the influence of different thicknesses of base course on geotextile performance. Field instrumentation and measurements included strain gauges, grids of rivet markers, moisture/temperature meters, and rut depth measurements.

Before the geotextile fabrics were placed, 1.5 ft (0.45 m) of material were subexcavated from the northbound lane and 2 ft (0.6 m) were subexcavated from the southbound lane. Nuclear density, torvane, and pocket penetrometer tests were performed on the subgrade at the subexcavation elevation. Samples of the subgrade were taken for moisture content, Atterberg limits, and hydrometer tests. Instrumentation for measuring moisture/temperature and geotextile strain was subsequently installed.

The initial lift of base course material over the geotextiles was 6 in. (150 mm) in the northbound lane and 12 in. (300 mm) in the southbound lane. The initial lift was compacted with a nonvibratory steel drum roller. Water was used to promote compaction. The design base course thickness was 12 in. (300 mm) in the northbound lane and 18 in. (460 mm) in the southbound lane. After completion of the first lift, a traffic test (referred to as Traffic 1) was performed with 10 passes of a loaded dump truck weighing 40 tons (350 kN) with rear tandem axles. After completion of Traffic 1, excavations (referred to as Excav 1) were made to the geotextile/subgrade elevation. Geotextile and subgrade samples were taken, and the instrumentation was read.

The second lift of base course material was placed and compacted. A second traffic test (Traffic 2) was performed, followed by the second set of excavations (Excav 2), which involved the same procedures used previously.

The following conclusions were drawn on the basis of the results of the Phase I investigation (Tsai et al., 1993):

- In all cases, the use of a geotextile prevented intermixing of the base course and subgrade if the geotextile survived construction.
- The presence of a geotextile resulted in more uniform rut depths, if the geotextile survived construction.
- In cases where the subgrade had a modest shear strength, rut depths were not reduced by the geotextiles.
- On the basis of rut depth measurements and visual observations, NP8 had the best overall performance.
- The SF geotextile appeared to reduce the strains in the underlying subgrade; however, pumping of the subgrade may have influenced these results.
- Observations indicated that during construction operations the needle-punched nonwoven geotextiles allowed unrestricted drainage of the subgrade while other types tended to retard drainage. The heavier weight needle-punched nonwoven geotextile appeared to enhance drainage.

2.3.2 Bucoda Test Site - Phase II

The results of the Phase II research were presented in Black (1997), Black and Holtz (1997), and Black and Holtz (1999). In 1996, excavations were made 5 years after installation of the geotextiles. Samples of geotextiles, subgrade, and base course materials were exhumed for visual observation and laboratory testing. In addition, in situ soil tests, including nuclear density, torvane and pocket penetrometer tests, were performed on the exposed subgrade soils. The field investigation procedures were generally the same for Phase II and Phase III. A more detailed description of the Phase III excavation procedures can be found in Chapter 3.

Atterberg limits and hydrometer tests were performed on the subgrade samples for classification and gradation. Sieve analysis tests were performed on the base course samples to quantify possible fines migration. Permittivity and wide width tensile tests were performed on the geotextile samples retrieved to characterize the filtration/drainage and strength characteristics of the geotextiles 5 years after installation.

Black and Holtz (1999) drew the following conclusions on the basis of the results of the Phase II investigation:

- At a site that historically had poor pavement performance, an assortment of geotextile separators had been effective at preserving the integrity of the pavement system since construction (5 years) even though some subgrade fines migrated through some of the separators into the base course.
- From the permittivity testing, it appeared that the heat-bonded geotextiles were significantly more susceptible to clogging than the needle-punched or slit film geotextiles.
- All of the geotextiles used in the test section survived the construction reasonably well, with the exception of the NP4 geotextile in the northbound lane, where severe rutting occurred during the trafficking tests for the Phase I investigation. More geotextile damage due to aggregate puncture generally appeared to occur under thinner initial base course lifts (northbound lane). Visual examinations indicated that the lighter-weight (HB and NP4) geotextiles sustained more

construction damage; however, this damage was not reflected in the results of the wide width tensile tests.

- The initial lift thickness of base course had a significant effect on the strength and elongation at failure of the geotextiles. Elongation at failure appeared to be more affected than the strength.
- The subgrade soils at the test site had consolidated since the geotextiles were installed. Density tests suggested that the subgrade in the sections containing geotextiles consolidated more than the subgrade in the sections without geotextiles.
- The long-term performance of geotextile separators may not be critical in many cases because of increased subgrade strength and reduced compressibility due to consolidation.

The results of the Phase I and Phase II investigations are compared to the results of the Phase III investigation in Chapter 5.

2.4 FHWA DESIGN PROCEDURE

The recommended design procedure for permanent roads developed by Christopher and Holtz (1991) is described in the FHWA Geosynthetic Design and Construction Guidelines Manual (Holtz et al., 1998) and is presented below.

The following concepts apply to the design procedure:

- A standard method (e.g., AASHTO) is used to design the pavement section, and any contribution that the geotextile makes to the structural support is neglected.
- Reductions in subexcavation (or dig-out) depths can be made, but the aggregate thickness required for structural support cannot be reduced.

- The recommended method will be used for installation of the first lift (stabilizer lift).

STEP 1. Determine the need for a geotextile. Estimate whether a geotextile is needed given the subgrade strength and past performance with similar soils. Maintenance records, nondestructive test results (e.g., FWD), or in situ test results may be useful in making this determination.

STEP 2. Design the pavement section without a geotextile. Design the pavement section with conventional methods (e.g., AASHTO). Do not consider any structural support from the geotextile.

STEP 3. Determine the need for additional aggregate. Use Figure 2.2 to determine whether additional aggregate is needed because of susceptibility of the subgrade soils to pumping and base course intrusion. If so, reduce that thickness and include a geotextile at the base/subgrade interface. A thickness reduction about one-half is normally cost effective.

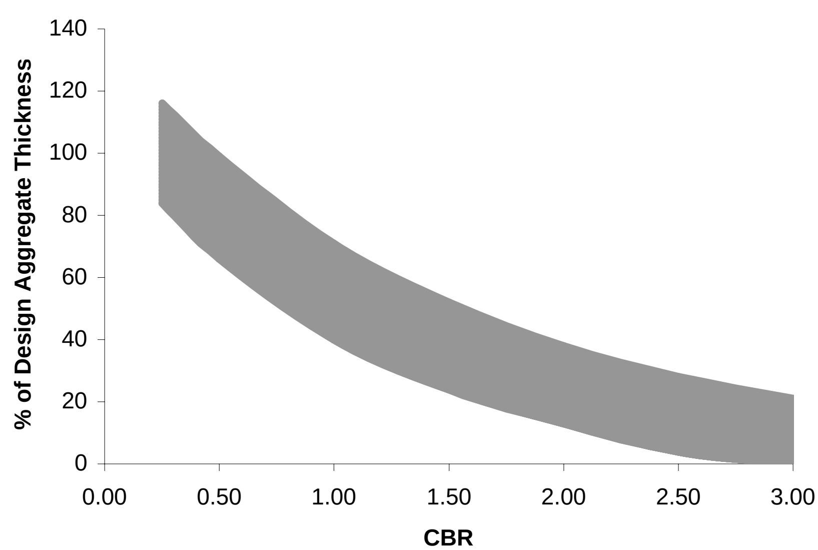

Figure 2.2. Aggregate loss to weak subgrades (after Holtz et al., 1998)

STEP 4. Determine the aggregate depth required to support construction equipment. Establish how much additional aggregate will be required to support the anticipated construction equipment. A rut criterion of 2 to 3 in. (50 to 75 mm) is typical. Using the rut criterion and anticipated traffic loading, the subgrade bearing capacity is estimated. Then, the bearing capacity and anticipated wheel loading are used to determine the required aggregate thickness from charts developed by the U.S. Forest service.

STEP 5. Compare thicknesses. Select the section with the greater thickness from Steps 3 and 4.

STEP 6. Check geotextile filtration. Use the properties of the subgrade soil to check that the geotextile meets the following criteria:

$$AOS \leq D_{85} \text{ (wovens)}$$
$$AOS \leq 1.8D_{85} \text{ (nonwovens)}$$
$$k_{geotextile} \geq k_{soil}$$
$$\Psi \geq 0.1 \text{ sec}^{-1}$$

STEP 7. Determine survivability requirements. The specified geotextile should meet the survivability requirements given in tables 2.1 and 2.2.

STEP 8. Specify geotextiles to meet or exceed the criteria established in Step 7.

STEP 9. Follow recommended construction procedures.

Table 2.1. Geotextile strength property requirements[1,2,3] (from AASHTO M 288, 2000)

Property	**ASTM Test Method**	**Units**	[2] **Class 1[4]**		**Class 2**	
			Elongation < 50%[5]	**Elongation ≥ 50%[5]**	**Elongation < 50%[5]**	**Elongation ≥ 50%[5]**
Grab Strength	D 4632	lb (N)	315 (1400)	200 (900)	250 (1100)	160 (700)
Sewn Seam Strength[6]	D 4632	lb (N)	280 (1260)	180 (810)	225 (990)	140 (630)
Tear Strength	D 4533	lb (N)	110 (500)	80 (350)	90 (400)	55 (250)
Puncture Strength	D 4833	lb (N)	110 (500)	80 (350)	90 (400)	55 (250)
Permittivity[7]	D 4991	sec^{-1}	0.5 for < 15% passing 0.075 mm (No. 200) sieve 0.2 for 15 to 50% passing 0.075 mm (No. 200) sieve 0.1 for > 50% passing 0.075 mm (No. 200) sieve			
Apparent Opening Size[8,9]	D 4751	mm	0.43 (No. 40 sieve) for < 15% passing No. 200 (0.075 mm) sieve 0.25 (No. 60 sieve) for 15 to 50% passing No. 200 (0.075 mm) sieve 0.22 (No. 70 sieve) for > 50% passing No. 200 (0.075 mm) sieve			
Ultraviolet Stability	D 4355	%	50% after 500 hours of exposure			

NOTES:
1. Acceptance of geotextile shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum, use value in weaker principal direction. All numerical values represent minimum average roll value. Lot samples according to ASTM D 4354.
4. Default geotextile selection is Class 1. Class 2 geotextile may be specified by the engineer for moderate survivability conditions (see Table 2.2).
5. As measured in accordance with ASTM D 4632.
6. When seams are required. Values apply to both field and manufactured seams.
7. Also, the geotextile permeability should be greater than the soil permeability.
8. Subsurface drainage geotextile requirements.
9. For cohesive soils with a plasticity index greater than seven, geotextile maximum average roll value for apparent opening size is 0.30 mm (No. 50 sieve).

Table 2.2. Construction survivability ratings (from Holtz et al., 1998; after Task Force 25, 1990).

Site Soil CBR at Installation[1]	< 1		1 to 2		> 3	
Equipment Ground Contact Pressure, psi (kPa)	**> 50 (350)**	**< 50 (350)**	**> 50 (350)**	**< 50 (350)**	**> 50 (350)**	**< 50 (350)**
Cover Thickness,[2] compacted, in. (mm)						
4[3,4] (100)	NR[5]	NR	1[5]	1	2[5]	2
6 (150)	NR	NR	1	1	2	2
12 (300)	NR	1	2	2	2	2
18 (450)	1	2	2	2	2	2

NOTES:
1. Assume saturated CBR unless construction scheduling can be controlled.
2. Maximum aggregate size not to exceed one-half the compacted cover thickness.
3. For low-volume, unpaved roads (ADT < 200 vehicles).
4. The 4 in (100 mm) minimum cover is limited to existing road bases and is not intended for use in new construction.
5. NR = NOT RECOMMENDED. High survivability relates to a Class 1 geotextile and moderate survivability relates to a Class 2 geotextile, per AASHTO M288 (1997).

2.5 WSDOT DESIGN AND SPECIFICATIONS.

Chapter 530 of the WSDOT Design Manual covers the design of geosynthetics. Sections 2-12 and 9-33 cover installation, material property requirements, and testing. The WSDOT procedures for the design and specification of geotextiles for separation and stabilization applications is summarized below.

2.5.1 Design

Chapter 530 of the WSDOT Design Manual (2002) covers geosynthetics. For separation and stabilization, the Manual refers to the properties listed in the Standard Specifications for most applications. Separation is defined as prevention of the mixing of two dissimilar materials. According to the Manual, separation geotextiles should only be used when the subgrade can be compacted in accordance with the Standard Specifications without removal and replacement of the subgrade soil. Separators can be considered if the subgrade resilient modulus is greater than 5800 psi (40 MPa) (CBR ~3.9) and if the subgrade does not consist of a saturated fine sandy, silty, or clayey soil. Separators are not recommended for situations where the subgrade resilient modulus is greater than 15,000 psi (100 MPa) (CBR ~10) and the subgrade consists of a dense granular soil.

A stabilization geotextile should be used where the subgrade consists of a saturated fine-grained soil with a resilient modulus of less than 5800 psi (40 MPa). In these situations, the subgrade generally cannot be compacted in accordance with the Standard Specifications. A site-specific design is required where the fill will be more than 5 ft high, or if the subgrade is an extremely soft saturated wet silt, clay, or organic (peat) soil.

In both the separation and stabilization applications, the geotextile should be placed directly on the subgrade soil rather than on aggregate, which defeats the purpose of the geotextile.

2.5.2 Specifications

Sections 2-12 and 9-33 of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (2004) cover the installation, material property requirements, and testing of geosynthetics.

Section 2-12, Construction Geotextile, covers geotextile installation. Roll identification, storage, and handling must conform with ASTM D 4873. The geotextile should be stored off the ground and in a manner that will prevent damage from sunlight, construction, precipitation, chemicals, flames, extreme temperatures, and anything else that may affect the performance properties of the geotextile. The subgrade must be relatively free of protruding objects before placement of the geotextile. Exposure of the geotextile during installation is limited to 14 days. The rut tolerance for construction equipment is 3 in. (75 mm) on the initial lift of aggregate, and no turning of vehicles is allowed. The contractor is responsible for replacing any damaged geotextile at his own expense. Seams, stitch type, and equipment used for stitching should meet the manufacturer's recommendations. Separation geotextiles require overlaps of 2 ft (600 mm) at all joints if stitching is not performed, and the initial lift thickness must be 6 in. (150 mm) or more.

Stabilization geotextiles also require a minimum overlap of 2 ft (600 mm) at all joints unless stitching is used. The initial lift thickness must be 12 in. (300 mm) or more, and vibratory compaction is not allowed on the first lift.

The geotextile property requirements specified in Section 9-33 of the WSDOT Standard Specifications are presented in Table 2.3.

Table 2.3. WSDOT Standard Specification - geotextile for separation and soil stabilization.

		Geotextile Property Requirements[1]	
		Separation	Soil Stabilization
Geotextile Property	Test Method[2]	Woven/Nonwoven	Woven/Nonwoven
AOS	ASTM D 4751	0.60 mm max. (#30 sieve)	0.43 mm max. (#40 sieve)
Water Permittivity	ASTM D 4491	0.02 sec^{-1}	0.10 sec^{-1}
Grab Tensile Strength, min. in machine and x-machine direction	ASTM D 4632	250/160 lbs. min. (1110/710 N min.)	315/200 lbs. min. (1400/890 N min.)
Grab Failure Strain, in machine and x-machine direction	ASTM D 4632	< 50% / ≥ 50%	< 50% / ≥ 50%
Seam Breaking Strength	ASTM D 4632[3]	220/140 lbs. min (980/620 N min.)	270/180 lbs. min. (1200/800 N min.)
Puncture Resistance	ASTM D 4833	80/50 lbs. min. (355/220 N min.)	112/79 lbs. min. (500/350 N min.)
Tear Strength, min. in machine and x-machine direction	ASTM D 4533	80/50 lbs. min. (355/220 N min.)	112/79 lbs. min. (500/350 N min.)
Ultraviolet (UV) Radiation Stability	ASTM D 4355	50% strength retained min., after 500 hrs. in weatherometer	50% strength retained min., after 500 hrs. in weatherometer

NOTES:
1. The properties listed are minimum average roll values. (i.e., the test result for any sampled roll in the lot shall meet or exceed the values shown in the table.)
2. The test procedures used are essentially in conformance with the most recently approved ASTM geotextile procedures, except for geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 and 915, respectively.
3. With seam located in the center of 8-inch long specimen oriented parallel to grip faces.

2.6. SUMMARY OF LITERATURE REVIEW

The use of geotextiles in roadway applications has increased significantly in the past decade, partly because of their relatively low cost and partly because of their acceptance in the transportation engineering community as an engineering material. A great deal of research has been conducted to quantify relationships between geotextile index properties and their performance in pavement sections, but much has yet to be learned. Geotextiles that are able to survive construction conditions generally perform beyond the life of the other pavement section materials. Limited research has been conducted to quantify the long-term drainage and filtration performance of geotextile separators in the field. Most of the literature reviewed did not compare any design criteria to the results of the permittivity tests used to quantify the filtration and drainage properties of exhumed samples. The current research compares the results of the field and laboratory investigations to filtration and drainage criteria recommended in the FHWA design procedure (Holtz et al., 1998).

In the past five years, the reinforcement benefits of geotextiles and geogrids have been better quantified, and design procedures have been developed. The design procedures can be used to determine the TBR, which quantifies the additional design life gained by adding a geotextile to the pavement section. Alternatively, the BCR can be determined, which quantifies the possible reduction in the base course thickness by including a geosynthetic into the section.

The Bucoda test site is one of the earliest test sections constructed to evaluate the field performance of geotextile separators. Currently, multiple test sites across the nation are being used to evaluate the performance of geotextile separators.

3. FIELD EXPLORATIONS AND TESTS

This chapter summarizes the procedures used to conduct the test pit excavations for the Phase III field investigations at the Bucoda test site. The field observations are presented, and the in situ test procedures and results are summarized.

3.1. FIELD PROCEDURES

Falling weight deflectometer (FWD) testing was conducted by WSDOT on August 12, 2003. Twelve test pit excavations were conducted within the test section between August 12 and 14, 2003 (Phase III). During construction of the test section (Phase I), test pit investigations were conducted in the outside wheel path, and instrumentation was also placed in the outside wheel path. The test pits in 1996 (Phase II) were excavated in the inside wheel path. To avoid placing the instrumentation in the outside wheel paths, the test pits were dug in the inside wheel path, and the center of the test pits were offset about 8 ft (2.4 m) east (toward Bucoda) from the center of the 1996 (Phase II) test pit locations. The general site layout is shown in Figure 3.1, and the test section layout is shown in Figure 3.2. The centers of the Phase III test pits were 3 ft (0.9 m) from the centerline. The test pits were about 4 ft (1.2 m) wide and about 6 ft (1.8 m) long parallel to the centerline.

The University of Washington (UW) field crew consisted of three graduate students. The UW field crew was responsible for marking the test pit locations; performing the site survey; assisting with base course removal; taking all samples of the base course, geotextiles and subgrade; performing in situ tests; and record keeping. WSDOT provided the equipment, materials, and personnel to perform traffic control,

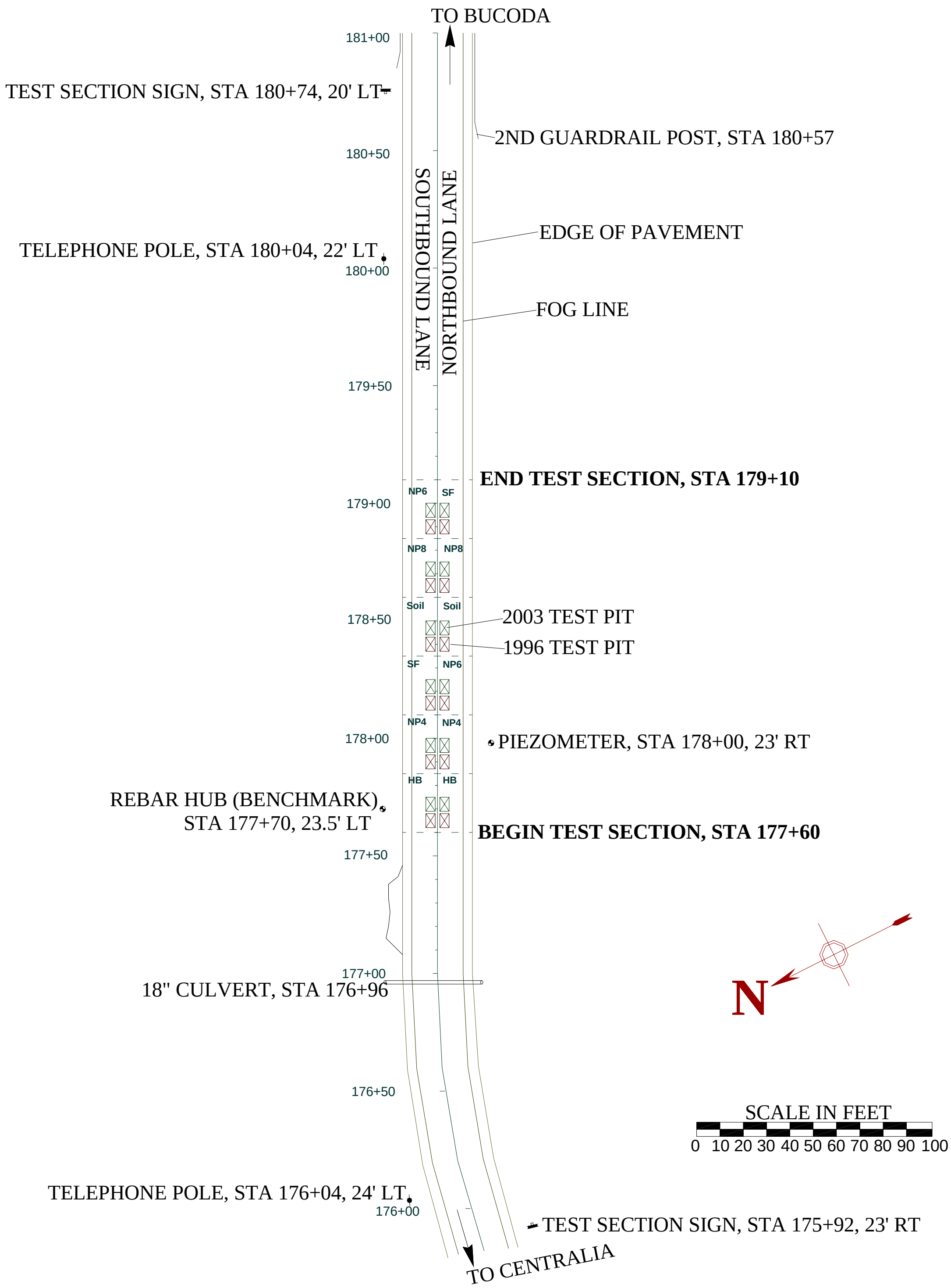

Figure 3.1. General site layout.

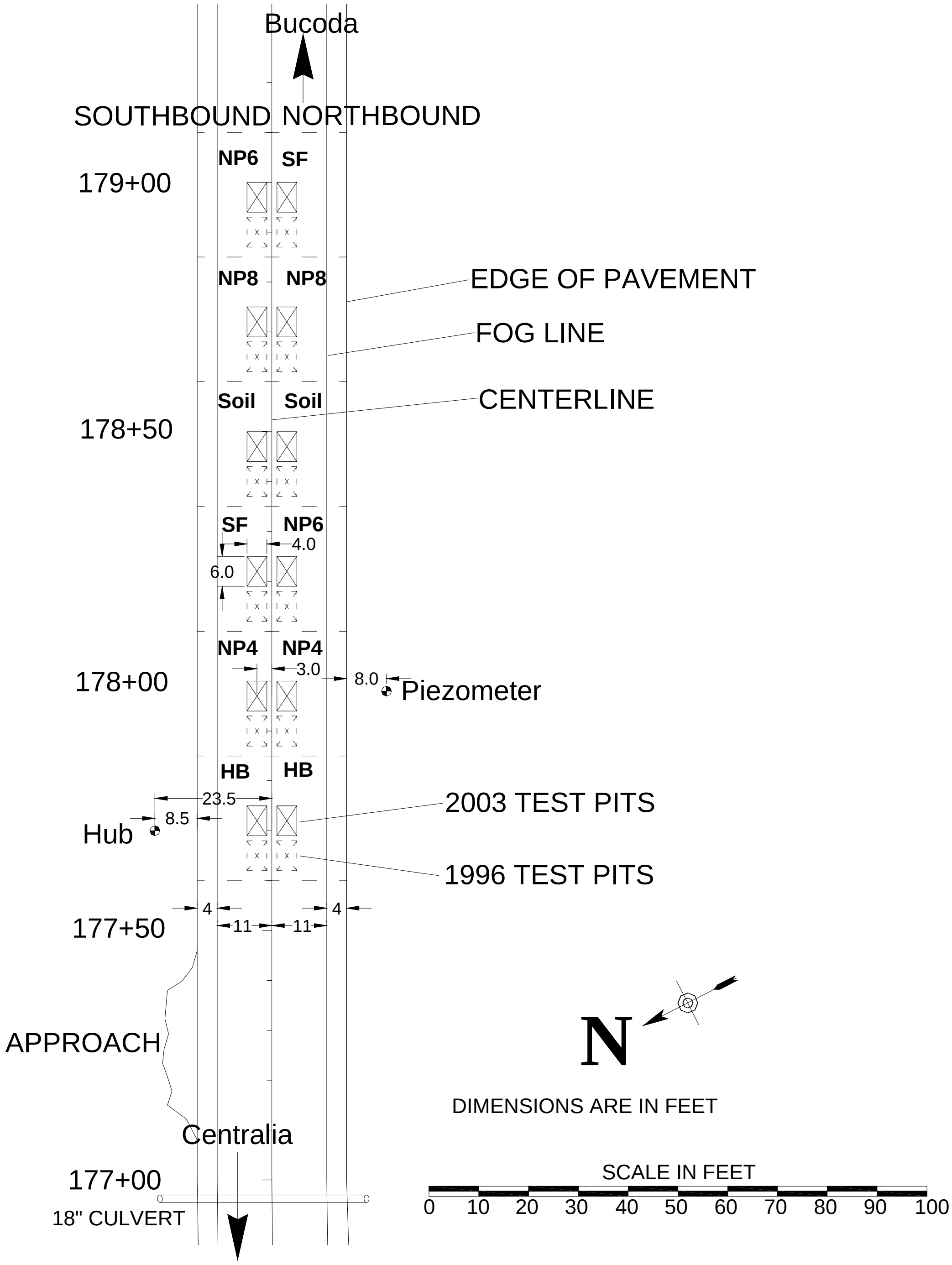

Figure 3.2. Test section layout.

FWD testing, pavement grinding and removal, base course removal, nuclear density gauge testing of the subgrade, and test pit backfilling and patching.

The UW field crew visited the site on August 11, 2003, to determine the location of the benchmark from the previous investigation and mark the test pit locations. The benchmark consisted of No. 4 rebar that was driven into the ground 24 ft (7.3 m) left of the centerline at Station 177+70. The rebar appeared to have been damaged by a mower; however, the bottom of the rebar appeared unharmed, so it was used as the benchmark. The damaged rebar was replaced with a section of No. 8 rebar at the same location. Measurements were taken to tie the test pit locations in with other landmarks. All stationing was calculated using the rebar hub at Station 177+70 as a benchmark. The UW field crew used a tripod-mounted level to determine ground surface elevations at the rebar hub, the piezometer, and at the center of all of the test pit locations. The 18-in. culvert invert north of the roadway at Station 176+96 was used as an elevation benchmark. This benchmark was assigned an elevation of 100.00 ft (30.48 m).

Before the excavations, photographs were taken of the site, and the pavement condition was observed within the limits of the test section. FWD testing was conducted at each test pit location. WSDOT used a trailer-mounted Dynatest FWD pulled behind a van. The test section is shown in Figure 3.3, and the WSDOT FWD is shown in Figure 3.4. The FWD testing procedures are further discussed in Chapter 6. After completion of FWD testing, the northbound lane was closed to traffic. The SF-NB location was the first test pit excavated. Test pits SF-NB, NP8-NB, Soil-NB, and NP6-NB were excavated on August 12th. All six of the test pits located in the southbound lane were excavated on

Figure 3.3. View of test section looking east (toward Bucoda).

Figure 3.4. Falling weight deflectometer (FWD) trailer and van.

August 13th in the following order: NP6-SB, NP8-SB, Soil-SB, SF-SB, NP4-SB, HB-SB. The NP4-NB and HB-NB test pits were completed on August 14th.

An effort was made to closely follow the procedures of the previous investigation (Black and Holtz, 1997). Procedures from similar investigations in Washington by Holtz and Page (1991) and Metcalfe and Holtz (1994) were also reviewed.

The test pit excavations began by saw-cutting the perimeter. Then an Alitec CP16BH pavement grinder (shown in Figure 3.5) attached to a Case 580 backhoe was used to remove the pavement. To limit damage to the geotextiles, the base course material was removed from the northwest corner of the test pit until the geotextile or subgrade was located. The base course material was loosened with pry bars and removed with a Vactor 2100 Series vacuum truck (Figure 3.6). After the location of the geotextile was determined, the excavation continued outward from the corner. Three samples of base course material were taken: (1) in the upper 12 in. (300 mm), (2) about 6 in. (150 mm) above the geotextile/subgrade, and (3) immediately on top of the geotextile/subgrade. Visual observations were made of the condition of the base course. The final 4 to 6 in. (100 to 150 mm) of material were removed with hand tools to avoid damaging the geotextile.

A photograph of the test pit was taken, and the depth to the geotextile/subgrade was recorded, along with visual observations of the geotextile condition. If standing in the excavation was required, the crew only stood along the perimeter of the test pit to avoid damage to the geotextile. The geotextile was cut with a utility knife about 2 in. (50 mm) from the perimeter of the test pit. One corner of the geotextile was peeled back, and another photograph was taken. The geotextile was removed, and photographs were taken

Figure 3.5. Pavement grinding operation.

Figure 3.6. Removing the base course with a vacuum truck.

of the top and bottom surfaces. The geotextile sample was sealed in two heavy-duty yard bags and covered to protect it from sunlight.

A photograph of the test pit was taken after removal of the geotextile. Subgrade observations were recorded. Pocket penetrometer, torvane and nuclear density tests were performed on the subgrade. A subgrade sample was then taken. The pavement and base course thicknesses were recorded.

A geotextile similar to the sample removed was installed in each geotextile test pit. The new geotextile was overlapped at least one inch (30 mm). Backfill was first placed along the edges to hold the geotextile in place. The backfill was compacted in lifts with a walk behind plate compactor up to the bottom of the pavement elevation. A photograph of backfill compaction is shown in Figure 3.7. Hot mix material was placed over the backfill and compacted with a roller (Figure 3.8).

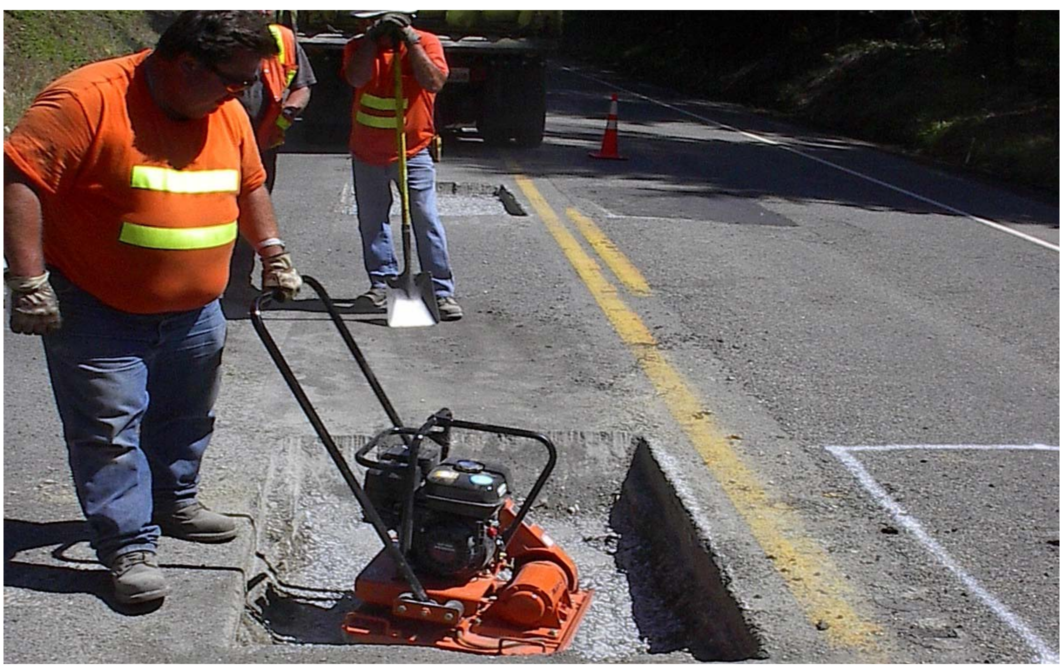

Figure 3.7. Backfill compaction.

Figure 3.8. Placing a patch over the test pit.

3.2. FIELD OBSERVATIONS

3.2.1 Site Observations

On August 11, 2003, the weather was mostly cloudy with intermittent rain showers and some periods of heavy rain. A Record of Climatological Observations was obtained for a National Oceanic and Atmospheric Administration (NOAA) weather station located in Centralia, Washington, about 9 mi (14 km) southwest of the test section (NCDC, 2003). The station received 0.28 in. (7.1 mm) of rain on August 11, 2003. Between August 5 and 9, 2003, the same station received 0.34 in. (8.6 mm) of rain. Between July 1 and August 4, 2003, the station received only 0.08 in. (2.0 mm) of rain. The mean daily temperatures during the investigation were between 65 and 70 deg F (18 and 21 deg C) (NCDC, 2003). The weather was generally humid. It the mornings it was usually foggy with temperatures in the mid-50ºF. In the afternoons it was generally sunny and humid, with temperatures in the upper 70ºF to mid 80ºF (about 25ºC to 30º C).

The 150-ft (45.7-m) long test section was located on a straight stretch of SR 507. (The site topography and geology are described in Section 1.3.) The test section ran northwest to southeast. This roadway slopes downward to the southeast at about a 4.5 percent grade. Thick vegetation and trees line both sides of the roadway. This section of SR 507 traverses the north end of the Skookumchuck River valley through the Bald Hills. The general topography is hilly, sloping downward to the south through the test section toward the Skookumchuck River, which parallels the roadway about ¼ mi (0.4 km) south of the test section. The elevation of the roadway is about 70 ft (21 m) above the river elevation.

Utilities were located before the test pit excavations. Two underground telephone utilities were marked parallel to the centerline near the northbound edge of the roadway (south side of road). An overhead power line ran parallel to the centerline north of the road.

WSDOT maintenance personnel indicated that an aggregate (chip) seal was placed on the adjacent section of SR 507 in 2002. A chip seal is a nonstructural form of routine maintenance used to seal the pavement surface and/or provide additional skid resistance. It consists of a layer of uniformly graded aggregate placed over a coating of liquid asphalt. The 2002 chip seal ended at the limits of the test section. However, some time between 1996 and 2002 a chip seal had been placed that covered up the patches from the previous investigation. Others observed some minor rutting in the wheel paths before placement of the chip seal. At the time of the Phase III investigation, no rutting or cracking was observed within the limits of the test section. Some minor longitudinal cracking was observed just outside the limits of the test section where no geotextiles were present. In general, the pavement in the test section appeared to be performing well.

3.2.2 Water Level Observations

Table 3.1 summarizes the water levels observed in the piezometer during the field investigation. The piezometer was located at Station 177+98, 23 ft (7.0 m) right of the centerline. The ground surface elevation at the piezometer was 100.89 ft (30.8 m) using the project datum. The piezometer was installed on May 16, 1991. The piezometer was installed to a depth of 11 ft (3.4 m).

Table 3.1. Water level observations.

Date	Water Level Depth, ft (m)	Water Level Elevation, ft (m)
8/11/03	5.3 (1.6)	95.6 (29.1)
8/12/03	4.7 (1.4)	96.2 (29.3)
8/14/03	5.1 (1.55)	95.8 (29.2)

3.2.3 Base Course Observations

The base course material consisted of crushed basalt that was specified to meet the requirements for crushed surfacing base course (CSBC). The CSBC material was well-graded, with 100 percent of the material passing the 1 ½-in. sieve and less than 10 percent passing the No. 200 sieve. The base course thickness ranged from 12.7 to 18.6 in. (320 to 470 mm) in the northbound lane and from 20.0 to 26.4 in. (510 to 670 mm) in the southbound lane. The design base course thickness was 12 in. (300 mm) for the northbound lane and 18 in. (460 mm) for the southbound lane. Figures 3.9 and 3.10 show

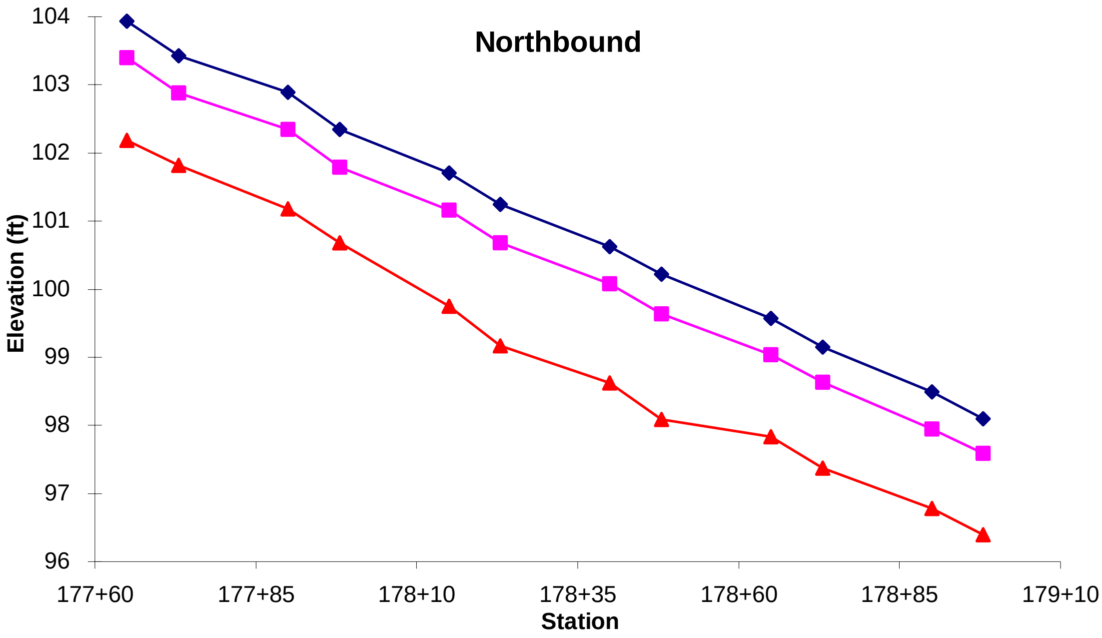

Figure 3.9. Profiles through the center of the test sections, northbound lane.

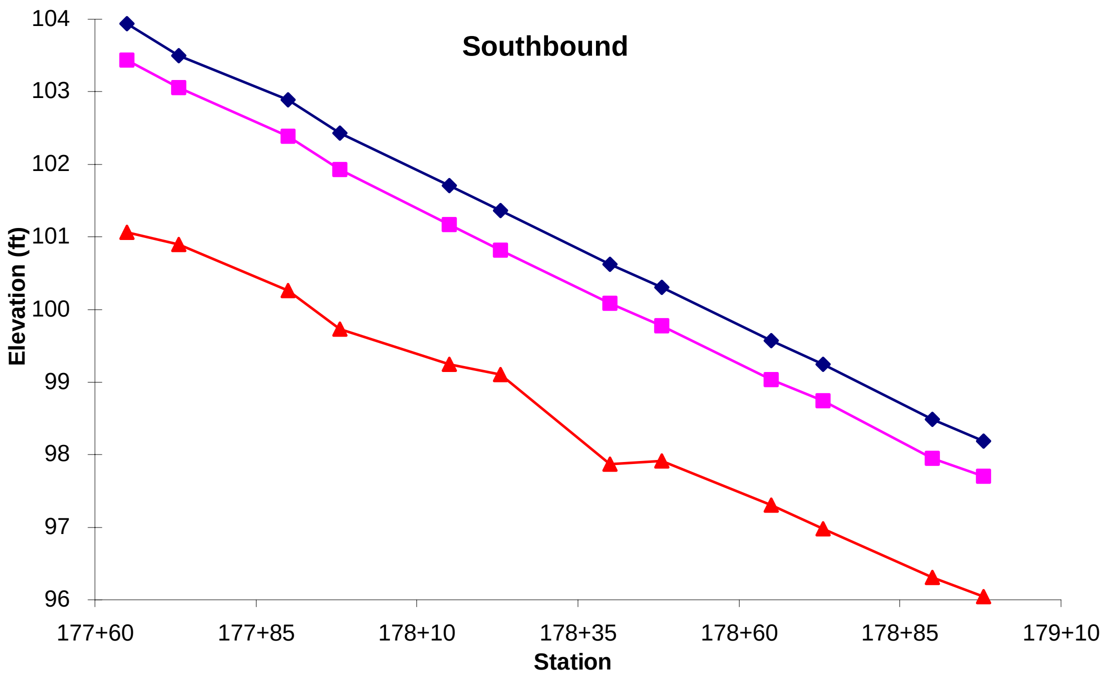

Figure 3.10. Profiles through the center of the test sections, southbound lane.

the profiles through the center of the test section in the northbound and southbound lanes, respectively.

In general, the base course was dense, and steel pry bars were necessary to loosen the material before it was removed by the vacuum truck.

The base course material in test pit NP4-NB was notably denser than that in the other test pits. The section thickness (pavement + base course) in the NP4-NB test pit ranged from 13.5 to 26 in. (340 to 610 mm). The average thickness of the pavement was 6.7 in. (170 mm); thus there was as little as 6.8 in. (170 mm) of cover over the geotextile and subgrade in that test pit. Tables 3.2 and 3.3 summarize the test pit observations.

Table 3.2. Summary of test pit observations, northbound lane.

Section	Bituminous, in. (mm)	Base Course, in (mm)	Mudcake, in. (mm)	Other Observations
HB-NB	6.3-7 (160-178)	11-14.3 (279-363)	0.3 (8)	Geotextile was heavily iron-stained. Subgrade surface heavily iron-stained.
NP4-NB	6.5-6.8 (165-173)	6.8-19.5 (165-495)	0.3-0.5 (8-13)	Base course denser than other test pits. Mudcaking thicker in lower areas. Tear near NE corner, parallel to centerline. Rut, 5.5-12.5 in. (140-318 mm) along S edge. Hump thru middle, parallel to centerline. Some organics (roots) present in subgrade. North half of test pit heavily iron-stained.
NP6-NB	6.5-7 (165-178)	17-18.5 (432-470)	0-0.5 (0-13)	Up to 1.5 in. (38 mm) of fines migration observed. Geotextile heavily iron-stained. Some iron-staining present on subgrade surface. Humps, 0.5-1.0 in. (13-25 mm) high near center.
Soil-NB	7 (178)	18.5-18.8 (470-478)	NA	Up to 0.5 in. (13 mm) of base/subgrade intermixing. Some iron-staining present on subgrade.
NP8-NB	6-6.5 (152-165)	14.5-15.5 (368-394)	Trace	Up to 0.5 in. (13 mm) of fines migration observed.
SF-NB	6-6.3 (152-160)	14.5 (368)	0-0.5 (0-13)	0.5-1.0 in. (13-25 mm) of fines migration observed. Hump 0.5-1.0 in. (13 –25 mm) higher near SE corner.

Table 3.3. Summary of test pit observations, southbound lane.

Section	Bituminous, in. (mm)	Base Course, in. (mm)	Mudcake, in. (mm)	Other Observations
HB-SB	5-5.8 (127-147)	24.5-27 (622-686)	0-0.5 (0-13)	Geotextile heavily iron-stained. Subgrade surface heavily iron-stained. Subgrade surface very hummocky.
NP4-SB	5.8-6.3 (147-160)	24.8-28.3 (630-719)	0-0.5 (0-13)	Lowest 1-2 in. (25-51 mm) of base course was wet. Up to 3 in. (76 mm) of observed fines migration. Subgrade surface was wet and shiny. Subgrade surface very hummocky, with lots of humps and low areas.
SF-SB	6.5 (165)	19.8-21.3 (503-541)	Trace	Some blinding observed on bottom of geotextile. Random iron-staining on subgrade surface.
Soil-SB	6-6.5 (152-165)	22.3-22.5 (566-572)	NA	2-3 in. (51-76 mm) fines migration observed. 0.5 in. (13 mm) of base/subgrade intermixing. Rut along N edge.
NP8-SB	6.0-6.5 (152-165)	20-21.5 (508-546)	0-0.5 (0-13)	Mudcaking observed in lower areas. Subgrade heavily iron-stained. Rut, 1.5 in. (38 mm) along N edge (parallel to CL). Hump in middle.
NP6-SB	5.3-6.3 (135-160)	19.8-20.3 (503-516)	0-0.5 (0-13)	Fold in geotextile ran from NW to SE corner. Subgrade heavily iron-stained on surface near NW and SE corners. Rut, 2 in (51 mm) along N edge (parallel to CL). Hump, 0.5 in (13 mm) adjacent to rut. Up to 1.5 in (38 mm) fines migration observed. Subgrade soils different on E/W halves. (2 subgrade samples taken).

3.2.4. Geotextile Observations

Observations made during exhumation of the geotextiles included noting any holes, folds, indentations, staining, and any evidence of blinding, clogging, or mud caking. Some of the observations are included in tables 3.2 and 3.3. The geotextiles were more closely examined in the laboratory, and those observations are discussed in Chapter 4.

Only one failure of a geotextile was encountered in the NP4-NB test pit. The tear was more than a foot long and paralleled the centerline. The extent of the tear is unknown because it continued beyond the limits of the excavation. As discussed previously, test section NP4-NB had the highest rut depths and experienced the largest strains in the geotextile during the traffic tests during construction of the section (Tsai and Savage, 1992).

A limited amount of damage was sustained by the geotextiles during the excavation procedures. Generally, damage consisted of punctures from the steel pry bar that were caused by nonuniform depths to the geotextile within individual test pits.

3.2.5. Subgrade Observations

Subgrade observations included soil classification, color, iron-staining, moisture, rutting, and any other notable conditions. The observations are included in tables 3.2 and 3.3, above. In the NP6-SB test pit, two different soil types were observed in the subgrade. The west half of the test pit appeared to be native soil, and the east half was noted as possible fill.

3.2.6. Photographs

Digital photographs were taken during each test pit excavation with all of the overlying base course material removed, with one corner of the geotextile peeled back from the subgrade, and after removal of the geotextile. Photos were also taken of both the top and bottom of the geotextile after removal. Figures 3.11 through 3.40 are photographs taken during the test pit excavations. Note that in the photos of the test pits, the identification card was always placed in the northwest corner.

Figure 3.11. First encounter of geotextile in SF-NB test pit.

Figure 3.12. SF-NB test pit with geotextile peeled back.

Figure 3.13. Bottom of SF-SB geotextile after removal.


Figure 3.14. Nuclear density test in SF-NB test pit.

Figure 3.15. NP8-NB test pit with geotextile peeled back.

Figure 3.16. NP8-NB test pit after removal of geotextile.

Figure 3.17. Bottom of NP8-NB geotextile after removal.

Figure 3.18. Top of NP8-NB geotextile after removal.

Figure 3.19. NP6-SB test pit with geotextile peeled back.

Figure 3.20. Top of NP6-SB after removal.

Figure 3.21. Bottom of NP6-SB geotextile after removal.

Figure 3.22. NP8-SB test pit with geotextile peeled back.

Figure 3.23. Top of NP8-SB geotextile after removal.

Figure 3.24. Soil-SB test pit after exposure of subgrade.

Figure 3.25. Geotextile peeled back in SF-SB test pit.

Figure 3.26. Bottom of SF-SB geotextile after removal.

Figure 3.27. Top of SF-SB geotextile after removal.

Figure 3.28. NP4-SB test pit with geotextile peeled back.

Figure 3.29. Bottom of NP4-SB geotextile after removal.

Figure 3.30. Top of NP4-SB geotextile after removal.

Figure 3.31. HB-SB test pit with geotextile peeled back.

Figure 3.32. Top of HB-SB geotextile.

Figure 3.33. Bottom of HB-SB geotextile after removal.

Figure 3.34. NP4-NB test pit prior to geotextile removal.

Figure 3.35. NP4-NB test pit with geotextile peeled back.

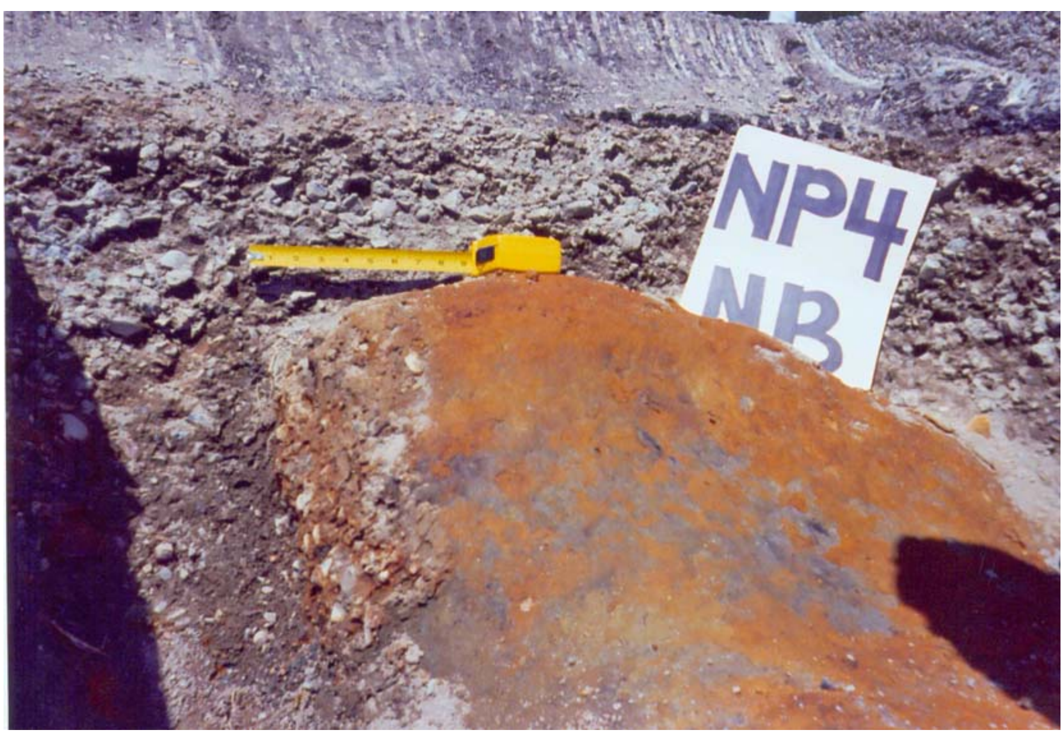

Figure 3.36. Photo of wave in NP4-NB subgrade. Tear was below end of tape measure.

Figure 3.37. Top of NP4-NB geotextile after removal.

Figure 3.38. Bottom of NP4-NB geotextile after removal.

Figure 3.39. HB-NB test pit with geotextile peeled back.

Figure 3.40. Top of HB-NB geotextile after removal.

3.3. IN SITU TESTING

After removal of the geotextiles, pocket penetrometer, torvane, and nuclear density tests were performed on the underlying subgrade.

3.3.1. Pocket Penetrometer Tests

The pocket penetrometer is a hand-held device that measures the unconfined compressive strength of very soft to stiff clayey soils. A piston attached to a calibrated spring is pushed into the soil, and the resistance of the soil is a measure of the unconfined compressive strength. Assuming the soil is fully saturated and fully drained, the unconfined compressive strength, q_u, can be related to the undrained shear strength, c_u, through the following relationship:

$$c_u = \frac{q_u}{2}$$

A summary of the results of the pocket penetrometer tests are in Table 3.4, below. The results of the individual tests conducted at each location are included in the Appendix. The unconfined compressive strengths determined from pocket penetrometer tests ranged from 0.25 to 3.75 tsf (24 to 359 kPa), with an average value of 1.73 tsf (165 kPa). Correlations between consistency and unconfined compressive strength of clays presented in Das (1998) indicated that the subgrade soils had a medium to very stiff consistency.

As previously discussed, two soil types were observed in the subgrade of the NP6-SB test pit. The west half of the test pit appeared to be native soil, and the east half was noted as possible fill. Therefore, the results of the strength tests performed on the west half were recorded separately from those on the east half.

Table 3.4. Summary of pocket penetrometer testing.

	NORTHBOUND LANE					
	HB	**NP4**	**NP6**	**Soil**	**NP8**	**SF**
q_u, tsf (kPa)	3.35 (321)	1.70 (163)	1.05 (101)	1.90 (182)	2.05 (196)	1.88 (180)
c_u, tsf (kPa)	1.68 (160)	0.85 (81)	0.53 (50)	0.95 (91)	1.03 (98)	0.94 (90)

	SOUTHBOUND LANE						
	HB	**NP4**	**SF**	**Soil**	**NP8**	**NP6-W**	**NP6-E**
q_u, tsf (kPa)	3.75 (359)	1.40 (134)	0.75 (72)	1.15 (110)	1.00 (96)	0.25 (24)	2.20 (211)
c_u, tsf (kPa)	1.88 (180)	0.70 (67)	0.38 (36)	0.58 (55)	0.50 (48)	0.13 (12)	1.10 (105)

3.3.2. Torvane Tests

The torvane is a hand held device used to directly estimate the undrained shear strength of very soft to stiff clayey soils. The device has multiple vanes that are pushed into the soil and rotated under the pressure of a calibrated spring until the soil fails. The results of the torvane tests are summarized in Table 3.5. The results of the individual tests conducted at each location are included in the Appendix.

Table 3.5. Summary of torvane testing.

	NORTHBOUND LANE					
	HB	**NP4**	**NP6**	**Soil**	**NP8**	**SF**
c_u, tsf (kPa)	0.68 (65)	0.68 (65)	0.33 (32)	0.78 (75)	0.39 (37)	0.31 (30)

	SOUTHBOUND LANE						
	HB	**NP4**	**SF**	**Soil**	**NP8**	**NP6-W**	**NP6-E**
c_u, tsf (kPa)	0.82 (78)	0.63 (61)	0.48 (45)	0.58 (56)	0.33 (32)	0.25 (24)	0.65 (62)

Comparisons of the shear strength determined from the pocket penetrometer tests and torvane tests for the southbound and northbound lanes are shown in figures 3.41 and 3.42, respectively. Comparisons among the NP4-SB, NP4-SB, Soil-SB, NP8-SB, NP6-SB-W, NP4-NB, NP6-NB, and Soil-NB sections were good. The HB-SB, NP6-SB-E, HB-NB, NP8-NB, and SF-NB sections were significantly different. The significant difference was likely due to several factors, but the highly variable subgrade soil conditions that were encountered in the test section probably contributed most to the differences. The pocket penetrometer and torvane test only a very small amount of soil, thus yielding variable results when conditions are not uniform.

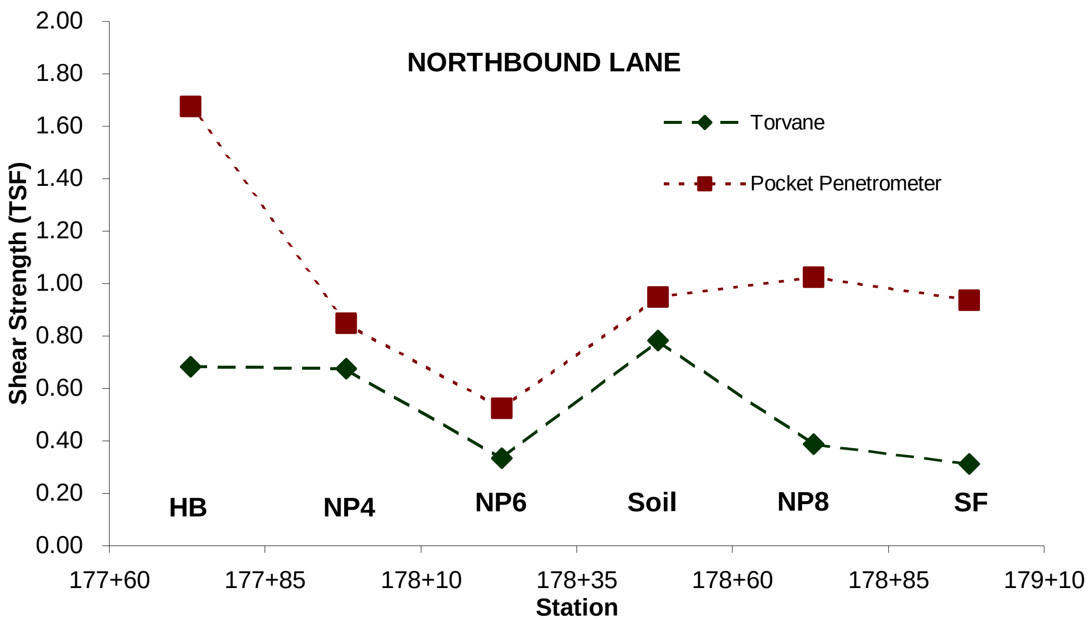

Figure 3.41. Comparison of in situ shear strength tests, northbound lane.

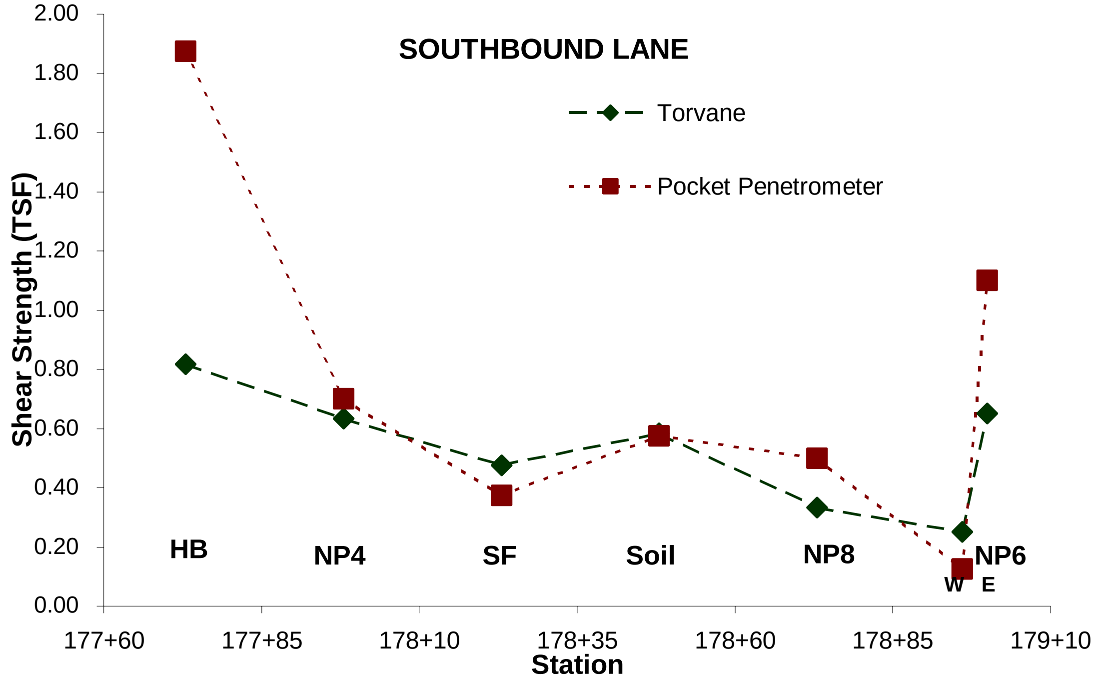

Figure 3.42. Comparison of in situ shear strength tests, southbound lane.

3.3.3. Density Tests

Density tests were performed on the subgrade of each test pit using a Trolxer Model 3430 nuclear density gauge. Two tests were performed with the nuclear density gauge probe set at a depth of 8 in. (203 mm). After the first test was completed, the gauge was rotated 90 deg., and a second test was performed in the same probe hole as the first test. The average of the two values was reported as the density for that test pit. The wet density, WD_F, dry density, DD_F, moisture density, MD_F, and moisture content, MC_F, were recorded for each test. Laboratory moisture content tests, w, were performed on samples taken from each of the test pits. Corrected values for wet density, WD, dry density, DD, and moisture density, MD, were determined by using the following relationships:

$$WD = WD_F$$

$$DD = \frac{WD}{1 + w}$$

$$MD = WD - DD$$

The results of the density tests are summarized in Table 3.6.

In the NP6-SB test pit, two different soil types were observed in the subgrade. The west half of the test pit appeared to be native soil, and the east half was noted as possible fill. Therefore, two sets of density tests were performed in the NP6-SB test pit, and their results varied significantly. The NP6-SB (east) density test had unreasonably high values. It is likely that a large cobble or piece of construction debris near the probe during the test caused the unreasonably high results.

Table 3.6. Summary of density test results.

	SOUTHBOUND LANE						
	HB	**NP4**	**SF**	**Soil**	**NP8**	**NP6-W**	**NP6-E**
***WD*, pcf (Mg/m^3)**	114.7 (1.84)	108.8 (1.74)	112.6 (1.80)	104.8 (1.68)	104.0 (1.67)	99.0 (1.59)	146.0 (2.34)
***DD*, pcf (Mg/m^3)**	87.0 (1.39)	75.4 (1.21)	78.4 (1.26)	71.1 (1.14)	64.2 (1.03)	56.7 (0.91)	103.9 (1.66)
***w*, %**	31.8	44.3	43.5	47.3	61.9	74.5	40.5

	NORTHBOUND LANE					
	HB	**NP4**	**NP6**	**Soil**	**NP8**	**SF**
***WD*, pcf (Mg/m^3)**	118.8 (1.90)	111.1 (1.78)	113.8 (1.82)	110.9 (1.78)	109.5 (1.75)	110.9 (1.78)
***DD*, pcf (Mg/m^3)**	90.0 (1.44)	77.9 (1.25)	81.0 (1.30)	83.6 (1.34)	76.3 (1.22)	74.4 (1.19)
***w*, %**	31.9	42.5	40.5	32.6	43.4	49.1

4. LABORATORY OBSERVATIONS AND TESTS

Visual observations were made of and laboratory tests were performed on the exhumed soil and geotextile at the University of Washington Geotechnical Laboratory. Strength and permittivity tests were conducted on the geotextile samples, and index tests were performed on the exhumed soil samples. Comparisons between the Phase III test results and the Phase I and Phase II test results are presented in Chapter 5.

4.1 GEOTEXTILE VISUAL EXAMINATION

The condition of the geotextiles was evaluated, including a damage assessment and a visual examination for iron staining, blinding, clogging, and caking.

4.1.1 Examination Procedures

The exhumed geotextile samples were placed on a light table. Holes caused by aggregate puncture larger than 40 mil (2 mm) were measured to quantify geotextile damage. Tears or rips in the geotextile that occurred during construction were also documented. Only the exhumed NP4-NB sample had a significant tear that occurred during construction.

The concepts of blinding, clogging, and caking are described in Section 2.1.3 and shown in Figure 2.1. Nonwoven geotextiles are generally more susceptible to clogging, and woven geotextiles are more susceptible to blinding (Metcalfe and Holtz, 1994; Black and Holtz, 1997). Therefore, the percentage of blinding was estimated for the woven samples, and the percentage of clogging was estimated for the nonwoven samples.

The following percentages were used to determine the degree of blinding/clogging (after Metcalfe and Holtz, 1994):

Percentage of Area Blinded/Clogged	Degree of Blinding/Clogging
0	None
1-10	Minimal
10-25	Minor
25-50	Moderate
50-75	Heavy
75-100	Severe

The percentage of area that had caking was also estimated, and the same arbitrary percentages (above) that had been used to describe the degree of blinding/clogging were used to describe the degree of caking.

Many of the samples also had rust-colored staining that apparently results from minerals carried by groundwater and deposited within the geotextile structure. A mineralogical analysis was not performed, but the coloring was assumed to be iron oxide. It will be referred to as iron oxide staining in this report. The percentage of iron oxide staining on each of the geotextile samples was also estimated. The following arbitrary percentages were assigned to determine the degree of iron oxide staining (after Metcalfe and Holtz, 1994):

Percentage of Area with Iron Oxide Staining	Degree of Iron Oxide Staining
0	None
< 1	Negligible
0-10	Minimal
10-25	Minor
25-50	Moderate
50-100	Heavy

4.1.2 Visual Examination Summary

Table 4.1 summarizes the laboratory geotextile observations. Only a few aggregate puncture holes were observed in the exhumed geotextile samples. The lightest weight nonwoven geotextiles (NP4 and HB) had the highest number of aggregate puncture holes. The number and size of holes were larger for the samples from the northbound lane than for the southbound lane, probably because the northbound lane had a smaller initial lift and smaller total thickness than the southbound lane.

Table 4.1. Summary of laboratory geotextile observations.

Geotextile	Sample Size, in. x in. (cm x cm)	Measured Damage[4,5]	Degree of Blinding/ Clogging	Degree of Caking	Degree of Iron Staining
HB-NB[1]	39 x 24 (100 x 62)	240, 120, 160 mil holes.	Heavy	Moderate	Heavy
NP4-NB[2]	39 x 31 (100 x 80)	120, 320, 400, 520, 640 mil holes.	Heavy	Moderate	Heavy
NP6-NB	33 x 28 (85 x 72)	160, 120 mil holes.	Severe	Moderate	Moderate
NP8-NB[3]	40 x 26 (102 x 66)	None	Moderate	Minimal	Minimal
SF-NB	49 x 22 (129 x 57)	None	Heavy	Heavy	Minimal
HB-SB[1]	37 x 22 (94 x 56)	None	Severe	Moderate	Heavy
NP4-SB	35 x 29 (90 x 73)	120, 120, 120, 120, 120 mil holes.	Severe	Heavy	Moderate
NP6-SB	39 x 33 (100 x 85)	None	Heavy	Moderate	Minor
NP8-SB[3]	39 x 22 (98 x 55)	None	Severe	Minimal	Moderate
SF-SB	41 x 28 (104 x 72)	None	Moderate	Severe	Moderate

Notes:
1. Many visible "pinholes" (< 10 mil), too numerous to count.
2. Large tear in southwest corner, approximately 10 in x 10 in (25 cm x 25 cm).
3. Some black staining present at aggregate indentations.
4. Damage was measured during wide width tensile tests.
5. 1 mil = 0.001 in. = .0254 mm.

The NP8-NB samples contained only moderate clogging and the SF-SB samples had moderate blinding; on the other hand, the NP6-NB, HB-SB, NP4-SB and NP8-SB samples contained severe clogging. The NP8-NB and NP8-SB specimens exhibited only minimal caking. The SF-SB sample exhibited severe caking. The NP8-NB and SF-NB samples showed only minimal iron staining, but the HB-NB, NP4-NB, HB-SB samples showed heavy iron staining.

Both of the HB samples had pin-sized holes across the geotextile that were too numerous to count. The NP4-NB sample had a large tear in the southwest corner that most likely occurred during construction.

4.2 LABORATORY TESTS

The geotextile samples and soil samples were brought back to the University of Washington Geotechnical Laboratory for testing. One geotextile sample was obtained from each test pit. The soil samples consisted of three base course and one subgrade sample from each test pit excavation, as described in Chapter 3.

4.2.1 Water Content

Water content tests were conducted on all of the base course and subgrade samples. The tests were conducted in accordance with ASTM D 2216, "Standard Test Method for Laboratory Determination of Water (Water) Content of Soil and Rock." Two tests were performed on each of the soil samples, and the results were averaged. The results of the water content tests are summarized in Table 4.2.

In the base course, the water contents ranged from 3.1 percent to 10.6 percent, with an overall average value of 5.3 percent. The water content generally increased with depth below the pavement. The NP6-NB, SF-SB, Soil-SB, and NP6-SB base course samples from immediately above the geotextile had the highest water contents (all were above 9 percent). The higher water content is an indication of a greater percentage of fines present in the base course immediately above the geotextile than at 6 in. above the geotextile. The grain size analyses results corroborated these findings; however, the difference in fines content from immediately above the geotextile to 6 in. above the geotextile was less than 5 percent at all four locations.

Table 4.2. Summary of water content tests.

Section	Average Water Content (%)			
	Base Course - Upper 12 in. (30 cm)	Base Course – 6 in. (15 cm) Above Geotextile	Base Course - Immediately Above Geotextile	Subgrade
HB-NB	3.1	4.0	6.3	31.9
NP4-NB	3.7	3.8	5.6	42.5
NP6-NB	4.4	4.3	9.3	40.5
Soil-NB	4.5	4.5	4.9	32.6
NP8-NB	4.8	4.6	5.6	43.4
SF-NB	4.3	4.2	5.9	49.1
HB-SB	3.9	4.4	6.6	31.8
NP4-SB	3.9	4.4	7.0	44.3
SF-SB	3.9	4.3	9.6	43.5
Soil-SB	3.3	6.3	9.5	47.3
NP8-SB	4.0	5.3	7.7	61.9
NP6-SB[1] (east)	3.5	4.6	10.6	40.5
NP6-SB[1] (west)				74.5
Average	**3.9**	**4.6**	**7.4**	**44.9**

Comments:
1. The NP6-SB subgrade had two distinctly different subgrade soil types on each half of the excavation bottom. One sample was obtained from each half (east and west) of the test pit. The east half refers to the end with higher stationing, towards Bucoda; and the west half refers to the end with lower stationing, towards Centralia.

The water content of the subgrade ranged from 31.8 percent to 74.5 percent, with an average value of about 45 percent. The highest subgrade water contents were at the adjacent NP6-SB (west) and NP8-SB test pits, with values of 74.5 percent and 61.9 percent, respectively. These samples also had the highest plasticity indexes.

4.2.2 Grain Size Analysis

Sieve analyses were conducted on all of the base course samples, and sieve-hydrometer analyses were conducted on all of the subgrade samples. The sieve analyses were conducted in accordance with ASTM C 136, "Standard Test Method for Sieve Analysis of Fine and Course Aggregate," and ASTM C 117, "Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing." The sieve-hydrometer analyses were conducted in accordance with ASTM D 422, "Standard Test Method for Particle-Size Analysis of Soils."

The purpose of the particle size analysis was to determine whether fines were migrating through the geotextiles into the base course. The HB-NB, NP4-NB, NP6-NB, SF-NB, SF-SB, and NP6-SB sections all had a higher percentage of fines present immediately above the geotextile than at higher elevations in the base course. In the Phase II investigations, only the SF-NB and NP6-NB sections had higher percentages of fines immediately above the geotextile. The difference in percentage was in all cases less than about 5 percent, which indicates that when fines migrate through the geotextile, the amount is almost negligible. The difference may be due to changes in the gradation of the base course material during construction operations. The acceptable range of two results on the same sample is 1.8 percent in the range of 2 to 10 percent passing (ASTM C 136, 2003).

Graphical results of the grain size distribution tests are contained in Appendix B.

4.2.3 Atterberg Limits

Atterberg limits were determined for each of the subgrade samples in accordance with ASTM D 4318, “Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.” The wet method of sample preparation was used, i.e., the samples were wet sieved to remove particles coarser than the No. 40 (0.420 mm) sieve.

The results of the Atterberg limit tests are summarized in Table 4.3. The Unified Soil Classification System (USCS) and American Association for State Highway and Transportation Officials (AASHTO) classifications are also presented in Table 4.3.

Table 4.3. Summary of Atterberg limit tests and soil classifications.

Section	Liquid Limit	Plastic Limit	Plasticity Index	Water Content	USCS Soil Classification	AASHTO Soil Classification
HB-NB	69	26	43	32	CH	A-7-6
NP4-NB	89	30	59	43	CH	A-7-5
NP6-NB	73	26	47	41	CH	A-7-6
Soil-NB	62	23	39	33	CH	A-7-6
NP8-NB	71	34	37	43	MH	A-7-5
SF-NB	72	34	38	49	CH	A-7-5
HB-SB	49	22	27	32	CL	A-7-6
NP4-SB	86	27	59	44	CH	A-7-5
SF-SB	83	28	55	44	CH	A-7-6
Soil-SB	80	25	55	47	CH	A-7-6
NP8-SB	134	33	101	62	CH	A-7-5
NP6-SB[1] (east)	49	34	15	41	ML	A-7-5
NP6-SB[1] (west)	150	40	110	75	CH	A-7-5

[1]See Table 4.2 Footnote.

The Atterberg limit test results indicated that the subgrade along the test section consisted of mostly high plasticity clayey soils. The majority of the tests conducted during the Phase I and Phase II investigations also plotted above the A-line. The natural water contents were all much higher than the plastic limits, indicating that the soil was in a plastic state. It is likely that the silty soil encountered in the east half of the NP6-SB test pit was fill that was not removed during construction before the geotextilewas placed. The test results are plotted on Figure 4.1.

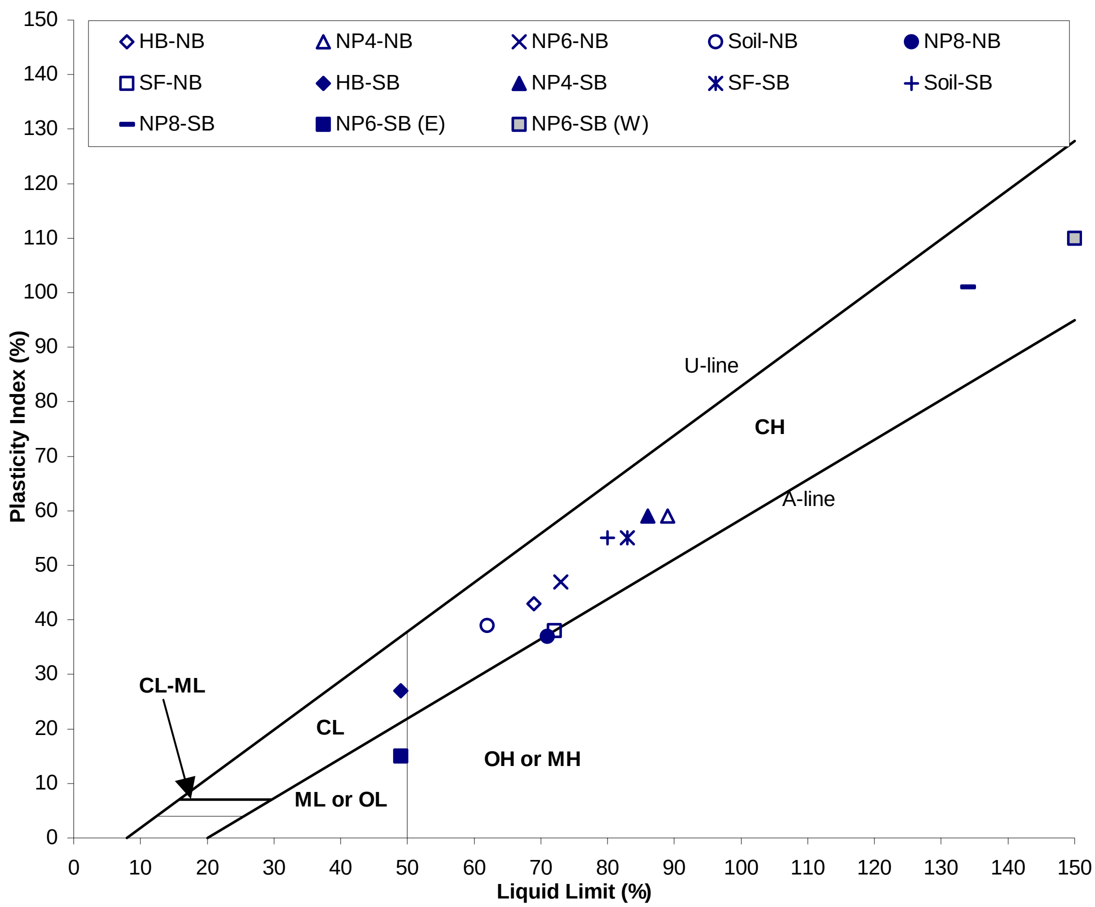

Figure 4.1. Plot of Atterberg limit test results on Casagrande's plasticity chart.

4.2.4 Permittivity

Permittivity tests were conducted on each of the exhumed geotextiles in general accordance with ASTM D 4491, "Standard Test Methods for Water Permeability of Geotextiles by Permittivity." Permittivity tests were also conducted on virgin control samples similar to the geotextiles obtained from the test pit excavations. Four specimens were cut and tested from each geotextile sample, and five tests were run on each specimen. Then each specimen was "washed" by gently massaging it under running water, and five more tests were conducted on the washed specimen.

The permeameter used was similar to the "STS geotextile permeameter" described by Christopher (1983), Holtz and Page (1991), and Metcalfe and Holtz (1994). Test specimens were soaked in de-aired water for a minimum of 24 hours before testing. The constant head test was conducted at a head of 2 in. (5 cm). The quantity of flow was measured directly from the standpipe, and the time was determined by using a stopwatch. The complete permittivity test procedure and the individual tests results are in Appendix C.

The purpose of the permittivity tests was to quantify the amount of clogging that had occurred since installation of the geotextiles. No tests were performed on the geotextiles at the time of construction, and the remaining field samples were lost before any tests were conducted.

The NP4, NP6 and NP8 virgin specimens were taken from samples of the same manufacturer and model number. Whether the samples were from the same lot is unknown because the lot number for the samples placed at the test site was unknown. No identification was available on the HB and SF control samples to identify the manufacturer and model number. The samples selected from the University of Washington Geosynthetics Laboratory had element, polymer, manufactured structure, color and physical properties that were similar to those installed below the test section.

The mean average roll values (MARV) or typical values from Table 1.1 were used for comparison. The virgin specimens were washed and tested in the same manner as the exhumed specimens to quantify the effects of washing. The results showed that washing had a negligible effect on the permittivity of the geotextiles.

Permeability, k, is the flow rate of a liquid through a material under differential head. Geotextile permittivity, ψ, is defined in ASTM D 4491 as the volumetric flow rate of water per unit cross-sectional area per unit head under laminar flow conditions. Permittivity is defined by the following formula:

$$\psi = QR_t/hAt$$

where:

ψ = permittivity, s^{-1},
Q = quantity of flow, mm^3,
h = head of water on the specimen, mm,
A = cross-sectional area of specimen, mm^2,
t = time for flow (Q), s,
R_t = temperature correction factor determined by $R_t = u_t/u_{20C}$,
u_t = water viscosity at test temperature, mP, and
u_{20C} = water viscosity at 20ºC, mP.

The results of the permittivity tests are summarized in tables 4.4, 4.5, and 4.6.

Table 4.4. Summary of permittivity tests results, northbound lane.

Section	Specimen	Unwashed – First Run ψ (s^{-1})	Washed – Average of 5 Runs ψ (s^{-1})	% Increase in ψ After Washing	Manufac-turers' Reported Value ψ (s^{-1})
HB-NB	A	0.047	0.27	474	0.1
	B	0.026	0.087	235	
	C	0.31	0.90	190	
	D	0.28	0.93	232	
	Averages -	0.17	0.55	283	
NP4-NB	A	0.69	2.26	228	2.7
	B	0.29	0.90	210	
	C	0.18	2.15	1094	
	D	0.74	2.54	243	
	Averages -	0.48	1.96	313	
NP6-NB	A	0.31	1.60	416	2.1
	B	0.27	1.87	593	
	C	0.55	2.30	318	
	D	0.25	1.81	624	
	Averages -	0.35	1.90	449	
NP8-NB	A	0.86	1.65	92	1.6
	B	0.25	1.24	396	
	C	0.63	1.52	141	
	D	0.93	2.05	120	
	Averages -	0.67	1.62	187	
SF-NB	A	0.096	0.15	58	0.1
	B	0.061	0.11	73	
	C	0.059	0.11	83	
	D	0.125	0.15	20	
	Averages -	0.085	0.13	59	

Table 4.5. Summary of permittivity tests results, southbound lane.

Section	Specimen	Unwashed – First Run ψ (s^{-1})	Washed – Average of 5 Runs ψ (s^{-1})	% Increase in ψ After Washing	Manufac-turers' Reported Value ψ (s^{-1})
HB-SB	A	0.45	1.08	142	0.1
	B	0.15	0.45	203	
	C	0.51	0.95	85	
	D	0.034	0.14	306	
	Averages -	0.29	0.66	184	
NP4-SB	A	0.63	1.25	98	2.7
	B	0.71	1.81	155	
	C	0.97	1.95	101	
	D	1.14	2.00	75	
	Averages -	0.86	1.75	107	
NP6-SB	A	1.12	2.59	131	2.1
	B	0.48	2.33	389	
	C	0.52	1.87	260	
	D	0.55	2.35	320	
	Averages -	0.67	2.29	275	
NP8-SB	A	0.47	1.40	199	1.6
	B	0.69	1.59	131	
	C	0.62	1.66	169	
	D	0.73	1.75	141	
	Averages -	0.62	1.60	160	
SF-SB	A	0.090	0.15	62	0.1
	B	0.078	0.16	99	
	C	0.036	0.10	169	
	D	0.115	0.16	41	
	Averages -	0.080	0.14	93	

Table 4.6. Summary of permittivity tests results, virgin samples.

Section	Specimen	Unwashed – Average of 5 Runs ψ (s^{-1})	Washed – Average of 5 Runs ψ (s^{-1})	% Increase in ψ After Washing	Manufac-turers' Reported Value ψ (s^{-1})
HB	A	0.144	0.150	4.2	0.1
	B	0.123	0.120	-2.4	
	C	0.156	0.154	-1.3	
	D	0.153	0.153	0.0	
	Averages -	0.144	0.144	0.2	
NP4	A	2.79	2.81	0.7	2.7
	B	2.76	2.78	0.7	
	C	2.57	2.63	2.3	
	D	2.74	2.60	-5.1	
	Averages -	2.72	2.71	-0.4	
NP6	A	2.52	2.51	-0.4	2.1
	B	2.30	2.46	7.0	
	C	1.97	1.97	0.0	
	D	2.11	2.20	4.3	
	Averages -	2.23	2.29	2.7	
NP8	A	1.86	1.87	0.5	1.6
	B	1.70	1.72	1.2	
	C	1.21	1.24	2.5	
	D	1.14	1.12	-1.8	
	Averages -	1.48	1.49	0.7	
SF	A	0.343	0.345	0.6	0.1
	B	0.160	0.162	1.3	
	C	0.481	0.462	-4.0	
	D	0.467	0.479	2.6	
	Averages -	0.363	0.362	-0.2	

The HB-NB and HB-SB specimens were the only exhumed geotextiles whose average unwashed permittivity exceeded the manufacturer's reported value (given in Table 1.1). The NP4, NP6 and NP8 nonwoven specimens (from both lanes) all showed average unwashed permittivity values about one order of magnitude lower than the manufacturer's value. The SF-NB and SF-SB specimens exhibited average unwashed permittivities just below the manufacturer's reported values.

The virgin HB, NP4, and NP6 samples had permittivity results close to the manufacturer's reported values. The virgin NP8 sample showed permittivity results about 10 percent lower than the manufacturer's reported value. The virgin SF sample had a permittivity that was more than three times the manufacturer's reported value.

The nonwoven specimens had the highest increase in permittivity after washing. The average percentage increase in permittivity after washing ranged from 107 percent to 449 percent for the nonwoven specimens. The SF (woven) specimens had an average increase in permittivity of 59 percent in the northbound lanes and 93 percent in the southbound lanes. After washing, only the NP4-NB, NP6-NB, and NP4-SB specimens' average permittivity did not meet or exceed the manufacturer's reported value.

4.2.5 Wide Width Tensile Strength

Wide width tensile strength tests were conducted on exhumed and virgin geotextile specimens in accordance with ASTM D 4595, "Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method." All tests were conducted in the machine direction of the geotextile, which was parallel to the centerline for the exhumed specimens. The specimens were 8 in. (200 mm) long and 8 in. (200 mm) wide. The distance between the clamps (gauge distance) was 4 in. (100 mm). ASTM does not specify procedures for preparing exhumed test specimens; therefore, the procedures recommended in Elias (2001) for preparing geosynthetics exhumed from construction projects were used. The specimens were hand washed gently under tap water before testing. The wide width specimens were soaked in water for a period of at least 24 hours before testing. The specimens were tested by using a constant rate of extension of 10 percent/min. Six test specimens were cut from each exhumed geotextile sample.

The purpose of the wide width tensile tests was to quantify the loss of strength, if any, of the geotextiles over their service lives. No tests were performed on the geotextiles at the time of construction, and the remaining field samples were lost before any tests could be conducted on the virgin companion samples from the test site. Therefore, it was necessary to use the mean average roll values (MARV) or typical values from Table 1.1 for comparisons.

Many different clamping methods are possible for holding the specimens in the testing machine. The ideal method prevents slippage of the geotextile between the clamps without the geotextile tearing at the clamps during the test. Tests were performed on virgin samples in order to determine an appropriate method of clamping the specimens. Two types of clamps were used, knurled and roughened. The knurled clamps have diamond shaped teeth, and the roughened clamps have a 60-grit sandpaper-like texture created by coarse sandblasting. In addition to using no protection, two methods of protecting the geotextile from the clamps were also evaluated. Tests were conducted with duct tape placed on the geotextile between the clamps; alternatively, thin particleboard strips were placed between the geotextile specimens and the clamps. The pressure applied to the clamps was also varied.

These tests determined that the procedures used by Black (1996) during the Phase II investigation were appropriate for testing the exhumed geotextile specimens. The knurled clamps were used for testing all of the exhumed geotextile specimens. No additional protection was provided between the clamps for the nonwoven specimens.

Duct tape was applied for additional protection to the slit film woven specimens between the knurled clamps. The nonwoven specimens were tested with a clamping pressure of 2000 psi (13.8 MPa), and the slit film woven specimens were tested at a clamping pressure of 3000 psi (20.7 MPa) because of their higher tensile strength. The number and size of holes between the clamps was measured for each of the exhumed specimens.

The NP4, NP6 and NP8 virgin specimens were taken from samples of the same manufacturer and model number. Whether the samples were from the same lot is unknown because the lot number for the samples placed at the test site was unknown. No identification was available on the HB and SF virgin samples to identify the manufacturer and model number. The samples selected from University of Washington Geosynthetics Laboratory had element, polymer, manufactured structure, color, and physical properties that were similar to those installed below the test section. The results of the tensile strength tests revealed that the virgin HB samples did not have the same strength properties as the HB geotextile installed at the test section.

The results of the wide width tensile strength tests are summarized in tables 4.7, 4.8, and 4.9. Results of each individual wide width tensile test are contained in Appendix D.

Table 4.7. Summary of wide width tensile tests, virgin samples.

Geotextile	Clamp Type/ Protection	Clamping Pressure, psi (MPa)	No. of Tests	Average Wide Width Strength, lb/in (kN/m)	Average Elongation at Failure, %
HB	Roughened	2000 (13.8)	3	100 (17.6)	80
HB	Knurled	3000 (20.7)	4	99 (17.2)	64
HB	Knurled	2000 (13.8)	3	110 (19.2)	77
NP4	Roughened	2000 (13.8)	3	55 (9.6)	83
NP4	Roughened	3000 (20.7)	3	60 (10.5)	81
NP4	Knurled	3000 (20.7)	2	54 (9.4)	71
NP4	Roughened w/ strips	3000 (20.7)	2	61 (10.7)	74
NP6	Roughened w/ strips	3000 (20.7)	4	78 (13.6)	78
NP6	Roughened	3000 (20.7)	3	73 (12.7)	86
NP6	Knurled w/ duct tape	3000 (20.7)	3	77 (13.5)	90
NP8	Knurled w/ duct tape	3000 (20.7)	4	90 (15.8)	86
NP8	Knurled	3000 (20.7)	3	79 (13.8)	85
NP8	Roughened w/ strips	3000 (20.7)	3	88 (15.3)	95
SF	Knurled	3000 (20.7)	5	187 (32.8)	16
SF	Knurled w/ duct tape	3000 (20.7)	5	205 (35.8)	19

Table 4.8. Summary of wide width tensile tests, ultimate strength.

Section	Average Ultimate Tensile Strength, lb/in (kN/m)	Manufacturers' Reported Tensile Strength, lb/in (kN/m)	Percent Retained Strength
HB-NB	23 (4.0)	35 (6.1)	66
NP4-NB	33 (5.6)	50 (8.8)	66
NP6-NB	50 (8.8)	70 (12.3)	71
NP8-NB	57 (10.0)	90 (15.8)	63
SF-NB	146 (25.5)	175 (30.6)	83
HB-SB	27 (4.7)	35 (6.1)	77
NP4-SB	32 (5.5)	50 (8.8)	64
NP6-SB	56 (9.9)	70 (12.3)	80
NP8-SB	68 (11.9)	90 (15.8)	76
SF-SB	137 (24.0)	175 (30.6)	78

Table 4.9. Summary of wide width tests, elongation at ultimate tensile strength.

Section	Average Percent Elongation at Maximum Tensile Strength	Manufacturers' Reported Elongation at Failure	Percent Elongation Retained[1]
HB-NB	26	45	58
NP4-NB	60	80	75
NP6-NB	49	95	52
NP8-NB	46	95	48
SF-NB	13	15	87
HB-SB	26	45	58
NP4-SB	46	80	58
NP6-SB	59	95	62
NP8-SB	54	95	57
SF-SB	13	15	87

[1]Percent Elongation Retained = (Average Percent Elongation ÷ Manufacturer's Reported Elongation) x 100

4.2.6 Four-Inch Strip Tensile Strength

Archived geotextile samples exhumed during the Phase II investigation were kept in the University of Washington Geosynthetic Laboratory. For comparison purposes, and to eliminate operator and procedural differences in testing methods, four-inch strip tensile tests were performed on samples exhumed from the northbound lane during the Phase II investigation in 1996 and during the Phase III investigation in 2003. Four-inch strip tests were also performed on some virgin samples. Eight-inch wide width strip tests could not be conducted because the remaining exhumed samples were not large enough.

The results of the virgin tests indicated that the four-inch strip tensile test had a higher standard deviation than the wide width tensile tests (see test results in Appendix D). However, the tests results showed that, in general, minimal strength loss had occurred since the last investigation in 1996. The left-over Phase II geotextile material was sealed in plastic bags and stored inside a cardboard box.

The same procedures that were used for the wide width tensile tests were used for the four-inch tensile tests, the only exception being that duct tape was applied to the NP4 specimens as additional protection from the knurled clamps. As discussed in the preceding section, the virgin HB geotextile had different strength properties than the HB geotextile installed at the Bucoda test site. Summaries of the four-inch strip tensile tests are presented in tables 4.10 and 4.11.

Table 4.10. Summary of four-inch strip tensile tests, virgin geotextile samples.

Geotextile	No. of Tests	Average Ultimate Tensile Strength, lb/in (kN/m)	Manufacturers' Reported Tensile Strength, lb/in (kN/m)	Average Percent Elongation at Maximum Tensile Strength	Manufac-turers' Reported Elongation at Failure
HB	4	62 (10.8)	35 (6.1)	50	45
NP4	6	45 (7.9)	50 (8.8)	80	80
NP6	6	56 (9.8)	70 (12.3)	86	95
NP8	8	65 (11.4)	90 (15.8)	91	95
SF	4	206 (36.1)	175 (30.6)	19	15

Table 4.11. Summary of four-inch strip tensile tests, exhumed geotextile samples.

Geotextile	Number of Tests		Average Ultimate Tensile Strength, lb/in (kN/m)		Average % Elongation at Maximum Tensile Strength	
	Phase II (1996) Specimens	Phase III (2003) Specimens	Phase II (1996) Specimens	Phase III (2003) Specimens	Phase II (1996) Specimens	Phase III (2003) Specimens
HB-NB	3	6	21 (3.6)	21 (3.6)	21	24
NP4-NB	3	2	34 (5.9)	18 (3.1)	65	43
NP6-NB	2	3	54 (9.5)	51 (8.9)	54	62
NP8-NB	3	3	50 (8.7)	45 (7.9)	49	49
SF-NB	3	3	138 (24.2)	144 (25.2)	13	14

Unfortunately, the size of the sample was very limited and only broad generalizations can be made from the data shown. The results of the four-inch strip tests are discussed further in Chapter 5.

4.3. SUMMARY

Atterberg limit test results indicated that the subgrade along the test section consisted of mostly high plasticity clayey soils with water contents about 15 percent higher than the plastic limit of the soil. The permittivity of the exhumed geotextile samples increased between 59 percent and 449 percent after washing, indicating clogging of the geotextiles. Wide width tensile test results indicated that the geotextiles retained between 63 percent and 83 percent of the manufacturer's reported tensile strength since installation in 1991.

5. ANALYSIS OF FIELD AND LABORATORY TESTS

In this chapter, the results of the field and laboratory investigations conducted at the Bucoda test site and at the University of Washington Geotechnical and Geosynthetics Laboratories are analyzed. Comparisons are made between the results of this investigation and the results of the Phase I investigation performed during construction in 1991 and the Phase II investigation conducted in 1996. Analysis of the falling weight deflectometer test results is presented in Chapter 6.

5.1 SUBGRADE CONDITIONS

Laboratory tests conducted on the subgrade samples included Atterberg limits and water content tests. In situ tests conducted on the subgrade included pocket penetrometer, torvane, and nuclear density tests.

5.1.1 Index and Classification Properties.

The water content and Atterberg limit tests were performed during the Phase I, Phase II, and Phase III investigations. Figures 5.1 and 5.2 show the variation of water content in the subgrade during the three phases of investigation.

The figures show that the water contents generally increased with stationing (toward the town of Bucoda), which is downhill. In general, the water contents were lowest during construction (Phase I) and highest during the Phase III investigation. This observation seems counterintuitive, since installation of the geotextiles should improve drainage, thereby lowering the water content. The inconsistency in the water content findings may be attributed to the variability of the soils along the test section and seasonal fluctuations due to changes in weather patterns, temperature, and precipitation.

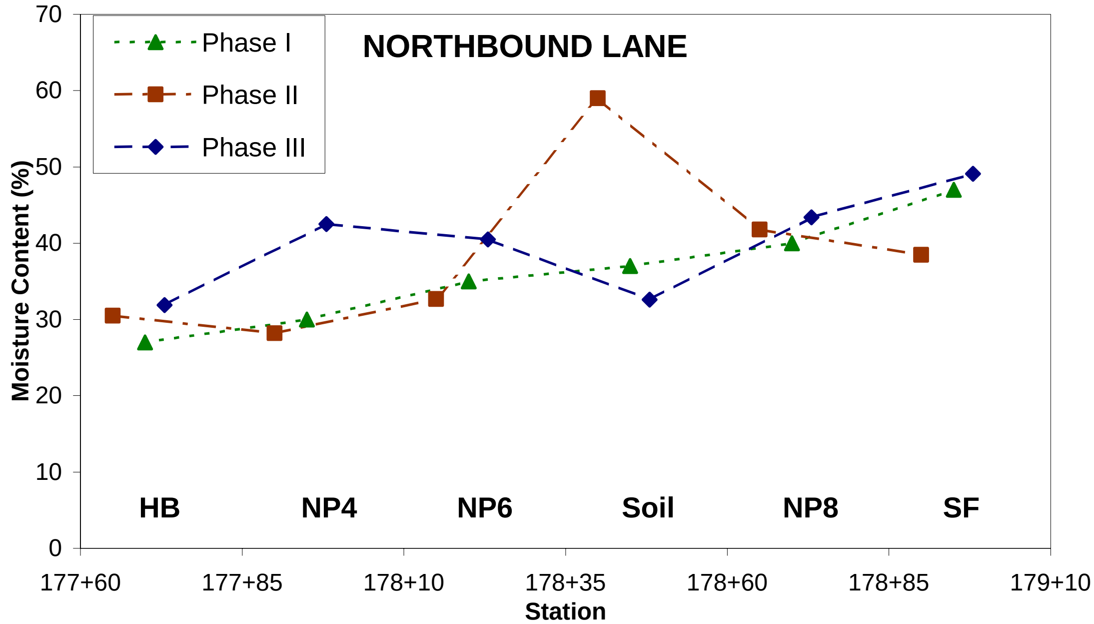

Figure 5.1. Summary of subgrade water content tests, northbound lane.

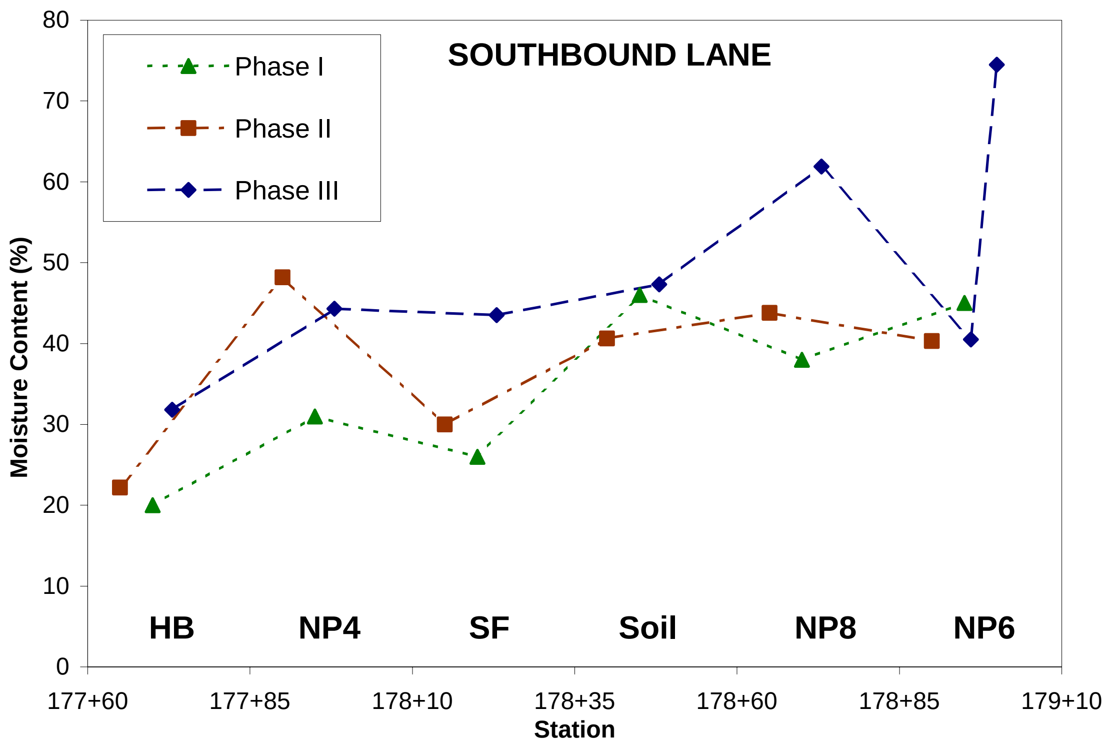

Figure 5.2. Summary of subgrade water content tests, southbound lane.

The significant difference in water contents in the NP6-SB section found during the Phase III investigation is evidence that subgrade soil conditions can change in a very short distance.

Atterberg limit tests were performed on subgrade samples during each phase of investigation. Figure 5.3 shows the variation of the Atterberg limit test results from the test section.

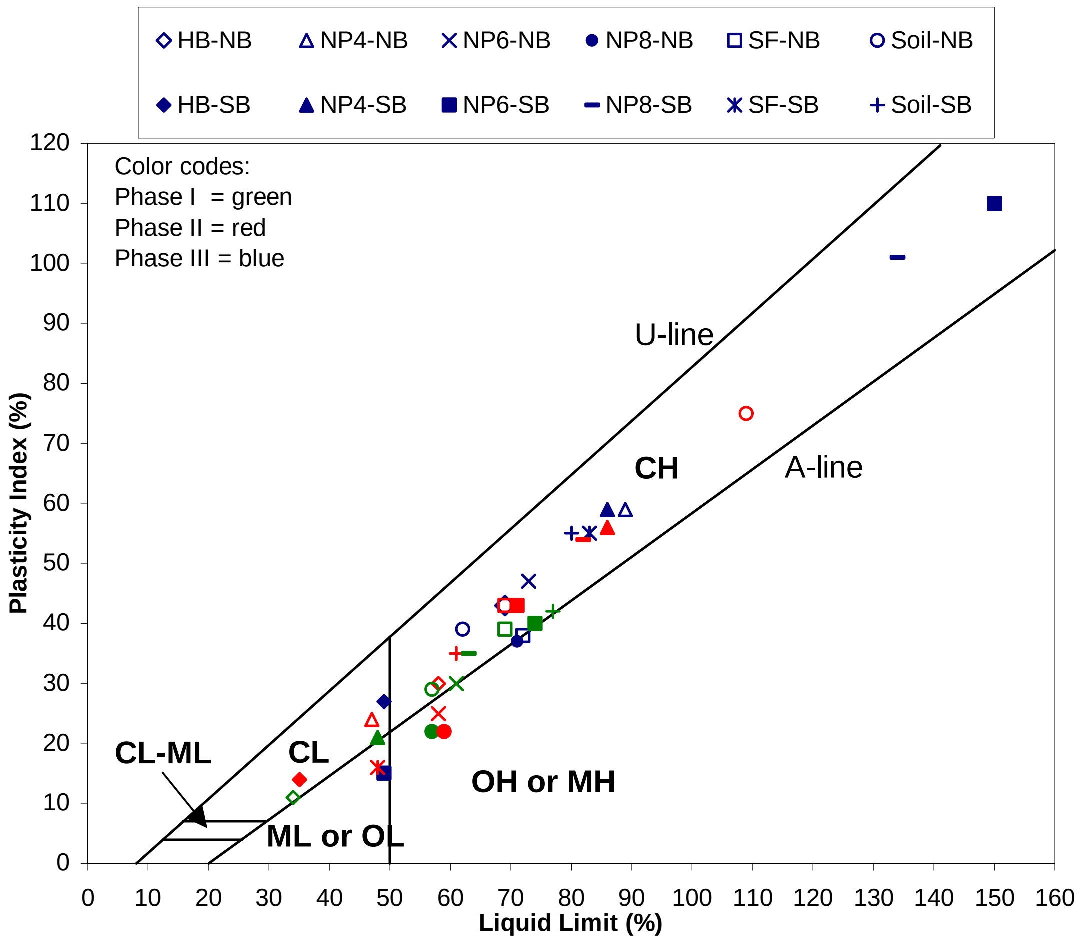

Figure 5.3. Summary of Atterberg limit tests.

The results of the Atterberg limit tests also illustrate the variability of the soils along the test section. The Phase I Atterberg limit test results generally showed lower plasticity indexes and liquid limits than the Phase II and Phase III test results. The Phase I samples were oven-dried and pulverized before testing. In contrast, the Phase II and Phase III samples were not oven-dried before testing, which may have contributed to this difference. Oven drying can alter the test results if organics or certain clay minerals (e.g., halloysite or montmorillonite) are present in the soil (Holtz, Advanced Geotechnical Laboratory class notes, Fall Quarter, 2003).

5.1.2 Shear Strength

Subgrade shear strengths were determined from pocket penetrometer tests and torvane tests during each phase of site investigation. Figures 5.4 and 5.5 show the results of the pocket penetrometer tests, and figures 5.6 and 5.7 show the results of the torvane tests.

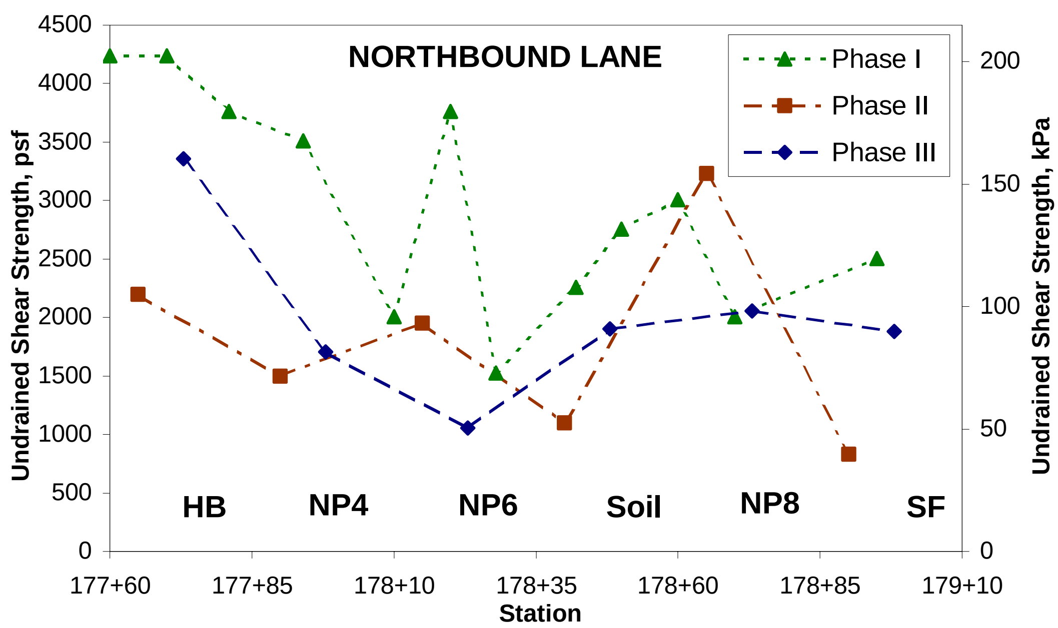

Figure 5.4. Pocket penetrometer results, subgrade soils, northbound lane.

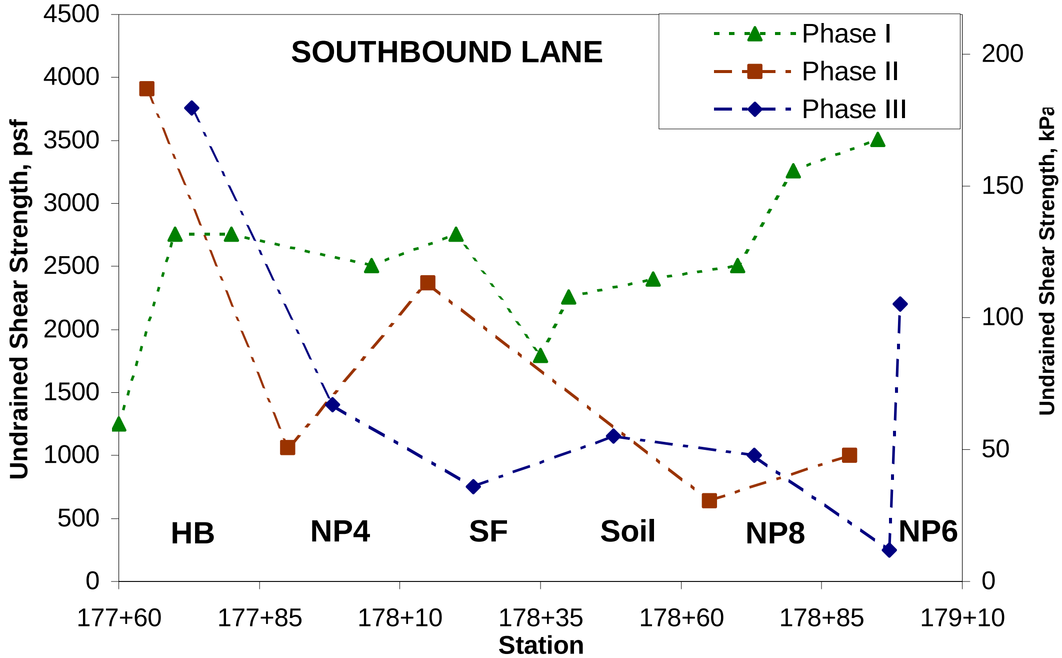

Figure 5.5. Pocket penetrometer results, subgrade soils, southbound lane.

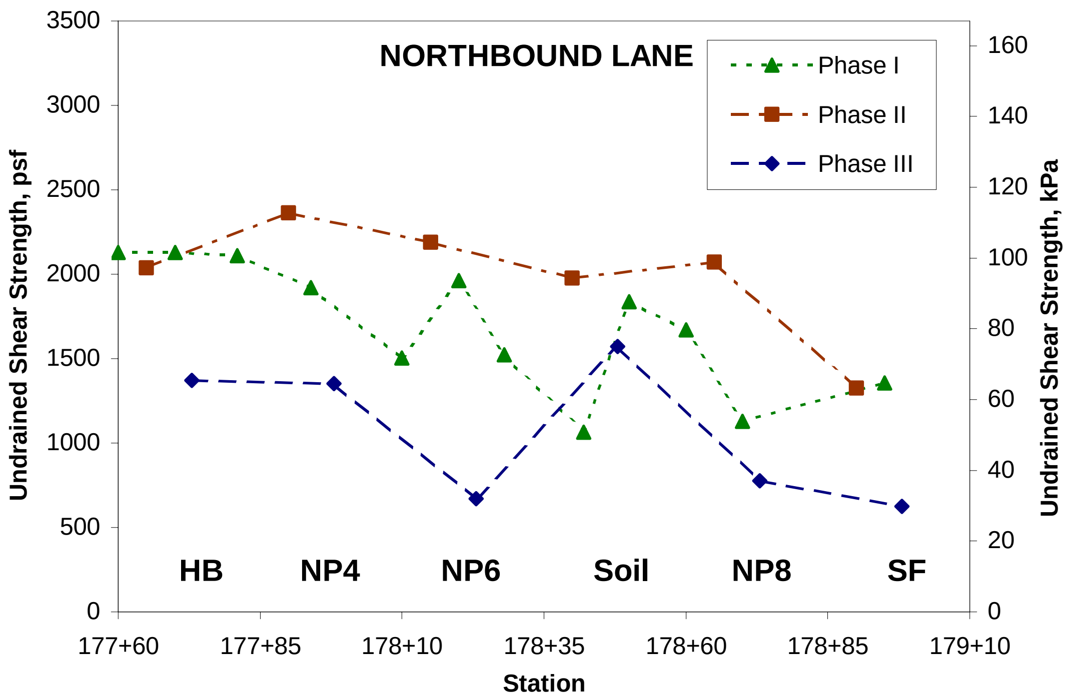

Figure 5.6. Torvane results, subgrade soils, northbound lane.

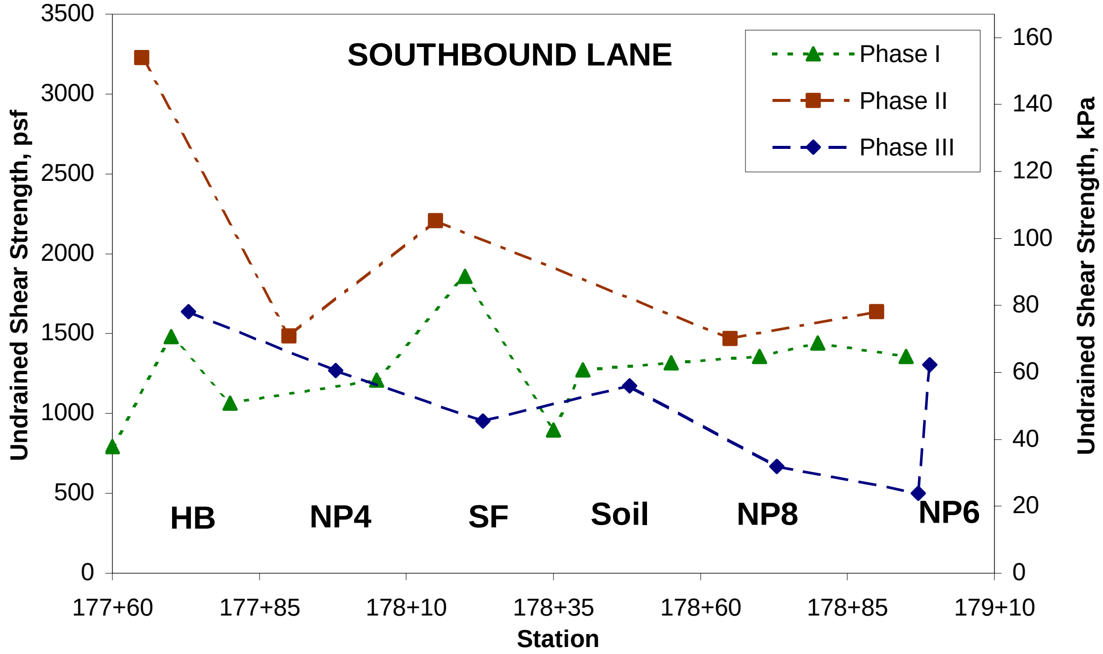

Figure 5.7. Torvane results, subgrade soils, southbound lane.

The pocket penetrometer results show a general trend of decreasing shear strength between the Phase I and the Phase II investigations. There does not appear to be any trend between the Phase II and Phase III investigations; the pocket penetrometer tests determined that both had lower shear strengths than those found during the Phase I investigation.

The torvane results showed a gain in shear strength between the Phase I and Phase II investigations in all of the sections except HB-NB and SF-NB, which were essentially unchanged. These results contradict the pocket penetrometer results described above. The Phase III results were generally the same or lower than the Phase I results, which also contradicts the torvane results between Phase I and Phase II. However, it is unlikely that the subgrade lost shear strength following construction. The most logical reason for this difference is variability in the subgrade conditions, as indicated by, for example, the water content and Atterberg limit test results. The limitations of the pocket penetrometer and torvane tests, the fact that different operators conducted the tests at different times, and the inherent variability of subsurface water conditions due to changes in weather would also contribute to the scatter in the results.

5.1.3 Density and Possible Consolidation

In situ density measurements were taken during the site investigations. All density tests were performed with a Troxler nuclear density gauge. The density test results are summarized in figures 5.8 and 5.9.

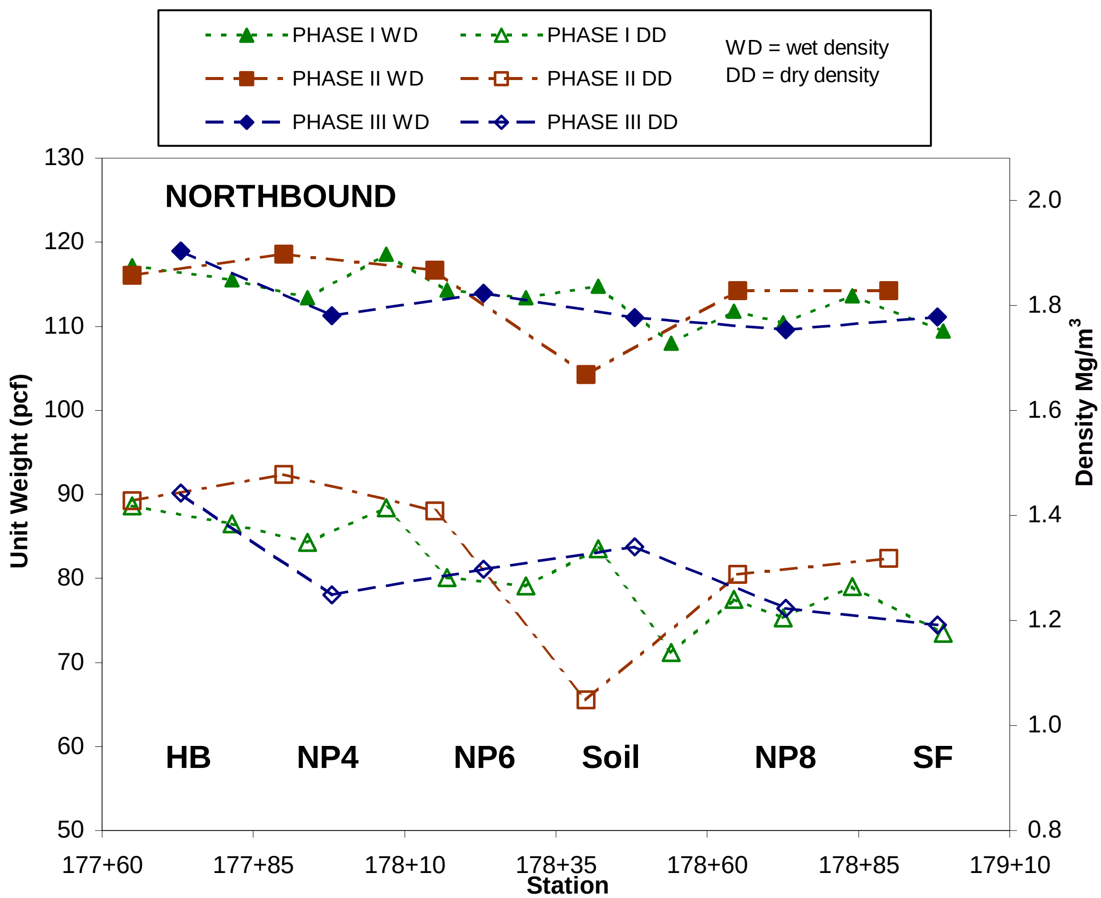

Figure 5.8. Summary of subgrade density tests, northbound lane.

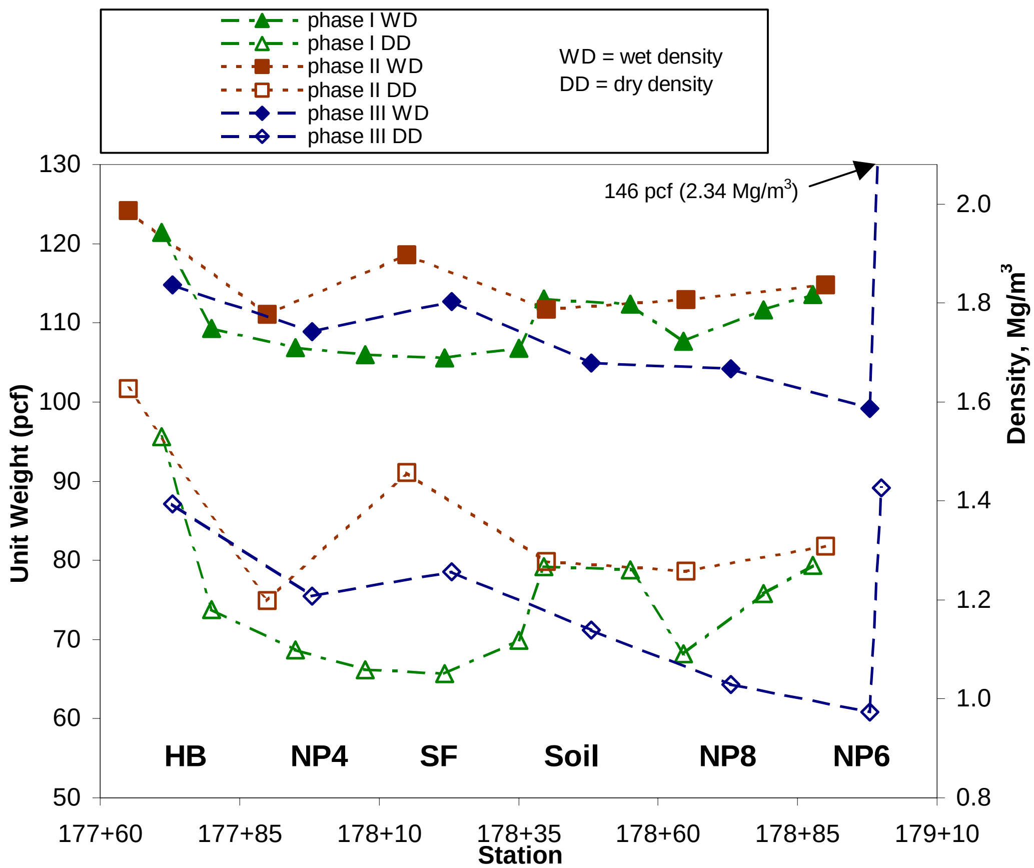

Figure 5.9. Summary of subgrade density tests, southbound lane.

Comparison of the Phase I and Phase II densities shows that a general increase in density occurred in all sections except the soil-only sections. This led Black and Holtz (1997) to the conclusion that the subgrade soils had consolidated. The results of the Phase III investigation, however, are not as conclusive. HB-NB, Soil-NB, and NP6-SB (east) were the only sections that showed an increase in density since the Phase II investigation. The NP6-NB, NP8-NB, SF-NB, HB-SB, NP4-SB, and SF-SB sections had lower densities than those found by the Phase II tests, but the densities were still higher than the Phase I results. The NP4-NB, Soil-NB, NP8-SB, and NP6-SB (west) sections had densities lower than those of both the Phase I and Phase II tests. The Phase III laboratory test results from the NP4-NB, NP8-SB, and NP6-SB (west) sections had much higher water contents, liquid limits, and plasticity indexes than the samples tested from the same sections during the Phase I and Phase II investigations. This could account for the lower densities because water has a much lower density than soil minerals; therefore, as the ratio of water to solids (i.e., water content) increases in a saturated material, the unit weight of the material will decrease.

On the basis of these results, it appears that some consolidation did occur following construction between the Phase I and Phase II investigations, but it is likely that no additional consolidation occurred in the period between the Phase II and Phase III investigations.

5.2 GEOTEXTILE PERFORMANCE

The performance of geotextile separators depends upon the following factors: (1) their ability to survive the installation and construction conditions without excessive damage, (2) the success of separating the base course material from the underlying subgrade, (3) the ability of the geotextile to prevent fines migration while providing subgrade drainage and pore pressure dissipation, and (4) the capability of the geotextile to resist sources of degradation and retain strength over the design life of the roadway. This section evaluates the performance of the geotextiles in terms of these functions. The FHWA design procedure (Holtz et al., 1998) is used to determine the appropriate characteristics for a geotextile separator based on the laboratory test results for the subgrade soils.

Geotextile strength properties are specified on the basis of the anticipated installation conditions. A Class 1 survivability rating refers to high survivability conditions; a Class 2 survivability rating refers to moderate survivability conditions (see

Table 2.2). Survivability becomes more critical with decreasing CBR, increasing equipment ground pressures, and decreasing cover thickness. The geotextile opening properties are specified on the basis of the grain size distribution of the subgrade soil, specifically, the percentage of fines that pass the No. 200 sieve, the D_{85}, and the D_{15} of the soil. A summary of these properties from the sieve-hydrometer test results from all three phases of investigations is presented in Table 5.2. Once again, variability is seen in the test results.

Table 5.1. Summary of subgrade grain size properties: Phase I, Phase II, and Phase III.

	D_{85}[1] (mm)			D_{15}[2] (mm)			Percentage Passing No. 200 Sieve		
Section	**Phase I (1991)**	**Phase II (1996)**	**Phase III (2003)**	**Phase I (1991)**	**Phase II (1996)**	**Phase III (2003)**	**Phase I (1991)**	**Phase II (1996)**	**Phase III (2003)**
HB-NB	0.10	0.12	0.33	0.0005	0.00018	0.0002	84	81	76
NP4-NB	0.075	0.13	0.021	0.00024	0.00013	0.00004	85	80	93
NP6-NB	Na[3]	0.20	0.048	Na	0.00011	0.0002	Na	78	90
Soil-NB	Na	0.0044	0.075	Na	0.00003	0.0002	Na	100	85
NP8-NB	0.31	0.42	0.34	0.0007	0.0002	0.0004	63	61	72
SF-NB	0.020	0.034	0.39	0.00006	0.00007	0.0006	94	96	70
HB-SB	Na	0.10	0.072	Na	0.00027	0.0017	Na	82	86
NP4-SB	Na	0.014	0.019	Na	0.00015	0.00004	Na	100	98
SF-SB	0.12	0.33	0.022	0.0006	0.0008	0.0001	78	58	98
Soil-SB	0.020	0.044	0.012	0.00031	0.00006	0.00007	94	94	99
NP8-SB	0.060	0.017	0.030	0.00025	0.00008	0.000001	88	100	89
NP6-SB (west)	0.022	0.025	0.0012	0.0002	0.00012	*	89	100	98
NP6-SB (east)			0.37			0.0027			50

Notes:
1. D_{85} = particle size that 85% of sample was finer than.
2. D_{15} = particle size that 15% of sample was finer than.
3. Na = test not performed.
* Unable to determine by extrapolation.

5.2.1 Survivability

Before a geotextile can perform under long-term service conditions, it must first survive installation and construction conditions. (Survivability concepts are covered in detail in Chapter 2.)

The average subgrade shear strength determined from pocket penetrometer and torvane tests during the Phase I investigation was about 1970 psf (94 kPa). This shear strength corresponds to a CBR value of about 3 (Holtz et al., 1998) and a survivability rating of 2 (moderate survivability), according to the FHWA requirements shown in Table 2.2. Therefore, the geotextiles should meet the Class 2 property requirements given in Table 2.1. However, only the NP8 and SF geotextiles met all of the specified strength properties shown in Table 2.1. (The Bucoda test site geotextile properties are given in Table 1.1.) The NP6, NP4, and HB geotextiles did not meet the grab strength requirement. In addition, the NP4 geotextile did not meet the tear strength requirement, and the HB geotextile also did not meet the puncture strength requirement.

The survivability of the geotextiles installed for this project was evaluated during the Phase I field investigations in 1991. The NP4-NB and HB-NB geotextiles failed to survive the construction conditions. Up to 136% strain was measured between the rivet markers in the NP4-NB geotextile after the first trafficking test. A tear in the NP4-NB geotextile was encountered during the Phase III investigation. The HB-NB geotextile was considered to have failed because of rut depths that exceeded 3.3 in. (84 mm) during the first trafficking test. Recall that the first traffic test in the northbound lane consisted of ten passes of a fully loaded dump truck after placement of the initial 6-in. (150-mm) lift.

Other than the NP4-NB section, no other tears (indicating a survivability failure during construction) were encountered in the Phase II or Phase III test pits. The relatively lightweight NP4 and HB geotextiles were able to survive the initial installation and construction conditions in the southbound lane with a subrounded to subangular aggregate, where the initial lift was 12 in. (300 mm) and the design base course thickness was 24 in. (610 mm). This supports the conclusion of Metcalfe and Holtz (1994) that lightweight (<6 oz/yd^2 or 200 g/m^2) geotextiles can survive under medium survivability conditions with rounded backfill and initial lift thicknesses of greater than 12 in. (300 mm).

5.2.2 Separation

The primary function of geotextiles used in roadways is to separate the base course from the underlying subgrade. Water content and grain size distribution tests on the base course were used to quantify any fines migration that would determine the success of the separation function of the geotextile. Visual observations were also made during the field investigation. The mudcake observations shown in Table 3.2 did not correlate well to the results of the grain size analyses (Section 4.3.2) of the base course materials immediately above the geotextile. The test results showed much less fines migration than what was observed in the field. The mudcake that was noted in the field was likely discoloration due to water ponding on top of the geotextile.

Although some of the grain size distribution tests indicated that the base course immediately above the geotextile did have up to 5 percent more fines than the base course at higher elevations, it is the author’s opinion that after 12 years in service this small percentage is negligible and would not be detrimental to the performance of the pavement. The additional percentage of fines that results in significant strength loss is reported in the literature at values of between 10 and 20 percent.

The soil-only sections and the failed geotextile section (NP4-NB) did not have more fines in the base course immediately above the geotextile/interface than what was found in the other samples. Given on these observations, the thickness of the base course (12.7 to 26.4 in. [320 to 670 mm] thick) likely resulted in a minimal amount of intermixing at the base course/subgrade interface. From the test results it is possible to conclude that the geotextiles did not allow significant fines migration; however, the results of the soil-only sections indicate that no fines migration occurred even when the geotextiles were not present. When a geotextile separator is incorporated into a design, the conventional “dig-out” or “subexcavation” depth can be reduced if the subgrade soils are moderately stiff (s_u > about 1900 psf [90 kPa], or CBR > about 3). If the base course thickness is reduced, the savings in granular material, hauling, and placement costs will at least partially offset the cost of the geotextile. Alternatively, incorporating the geotextile into the section with no reduction in base course will likely extend the service life of the pavement section, as was discussed in Chapter 2. The soil-only sections at the Bucoda test site performed similarly to the sections with geotextiles, which neither supports nor contradicts the extension of service life hypothesis.

5.2.3 Filtration and Drainage

A secondary function of geotextile separators is to act as a filter between the subgrade and base course and prevent fines migration. At the same time the geotextile must allow drainage and pore water pressure dissipation of the subgrade soil. Grain size analyses discussed in the preceding section and permittivity tests were used to assess the success of the geotextile separators as a filter and to quantify their drainage capabilities.

The FHWA design manual (Holtz et al., 1998) recommends an apparent opening size (AOS) of less than or equal to D_{85} for woven geotextiles and an AOS of less than or equal to 1.8 x D_{85} for nonwoven geotextiles. A comparison of the AOS values in Table 1.1 with the D_{85} values in Table 5.2 shows that most of the geotextiles did not meet the FHWA AOS criteria. If the AOS is smaller than the larger soil particles, the soil will be retained by the geotextile, but if the AOS is larger than the largest soil particles, the filter may not effectively retain the soil. Some of the subgrade soils had a high clay fraction with very low D_{85} values (for example, D_{85} = 0.0012 mm, NP6-SB (west), Phase III). Specifying such a small AOS would not be practical , and it would require a geotextile with a relatively low permeability. The average D_{85} value was about 0.12 mm. Therefore, for nonwoven geotextiles it would be practical to specify a geotextile with an AOS of less than 0.22 mm, and for woven geotextile the AOS should be 0.12 mm or smaller. The values given in Table 1.1 show that all of the nonwoven specimens met this requirement; however, the slit film woven specimen did not. In this situation, the engineer would be best advised to select a geotextile with a relatively small AOS but also a moderate permittivity.

Table 5.2. Summary of permittivity test results on virgin specimens.

Geotextile	Manufacturer's Reported Value, ψ (s^{-1})	Phase II Investigation (1996), ψ (s^{-1})	Phase III Investigation (2003), ψ (s^{-1})
HB	0.1	1.47	0.14
NP4	2.7	3.34	2.71
NP6	2.1	2.65	2.23
NP8	1.6	1.92	1.48
SF	0.1	0.12	0.36

All tests indicated that 50 percent or more passed the No. 200 sieve. Therefore, the most conservative permittivity requirement specified in Table 2.1, or 0.1 sec^{-1}, would be used for design. Because all of the subgrade soils had a high percentage of silt and clay size materials, the permeability would likely be less than 10^{-4} cm/sec (Holtz and Kovacs, 1981, Figure 7.6). Most geotextile separators exceed this permeability, but this supposition should be verified. The permittivity test results show that all of the geotextiles had unwashed permeabilities at least one order of magnitude greater than 10^{-4} cm/sec. The permittivity test results are in Appendix C.

Table 5.3 summarizes average permittivity results for the exhumed specimens from the Phase II and Phase III investigations. Figure 5.10 is a plot of the average permittivity from the unwashed (1st test) results compared to the manufacturers' reported values. Figure 5.11 is a plot of the average washed permittivity compared to the manufacturers' reported values.

Table 5.3. Summary of average permittivity results: Phase II and Phase III.

Section	Manufac-turers' Reported Value ψ (s^{-1})	Phase II Investigation (1996)			Phase III Investigation (2003)		
		Unwashed 1st Test ψ (s^{-1})	Washed Average ψ (s^{-1})	% Increase	Unwashed 1st Test ψ (s^{-1})	Washed Average ψ (s^{-1})	% Increase
HB-NB	0.1	0.38	2.57	951	0.17	0.55	283
NP4-NB	2.7	1.46	2.74	110	0.48	1.96	313
NP6-NB	2.1	1.96	3.05	60	0.35	1.90	449
NP8-NB	1.6	1.46	2.38	70	0.67	1.62	187
SF-NB	0.1	0.13	0.22	62	0.09	0.13	59
HB-SB	0.1	0.30	2.45	1941	0.29	0.66	184
NP4-SB	2.7	1.92	2.67	39	0.86	1.75	107
NP6-SB	2.1	1.92	2.91	67	0.67	2.29	275
NP8-SB	1.6	1.00	2.39	149	0.62	1.60	160
SF-SB	0.1	0.09	0.18	97	0.08	0.14	93

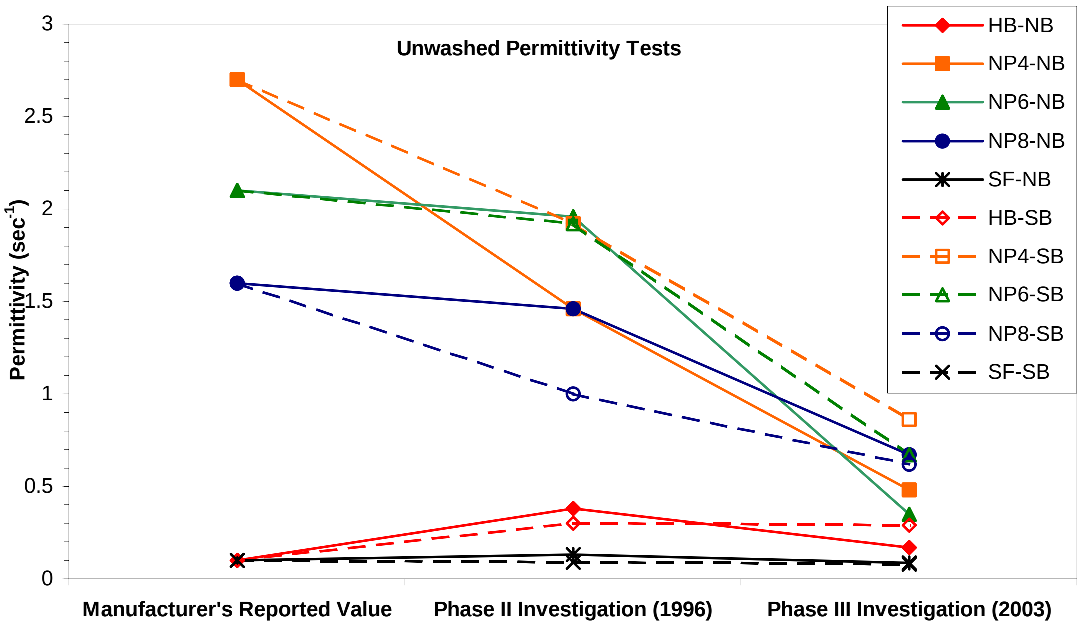

Figure 5.10. Summary of permittivity test results on unwashed exhumed specimens.

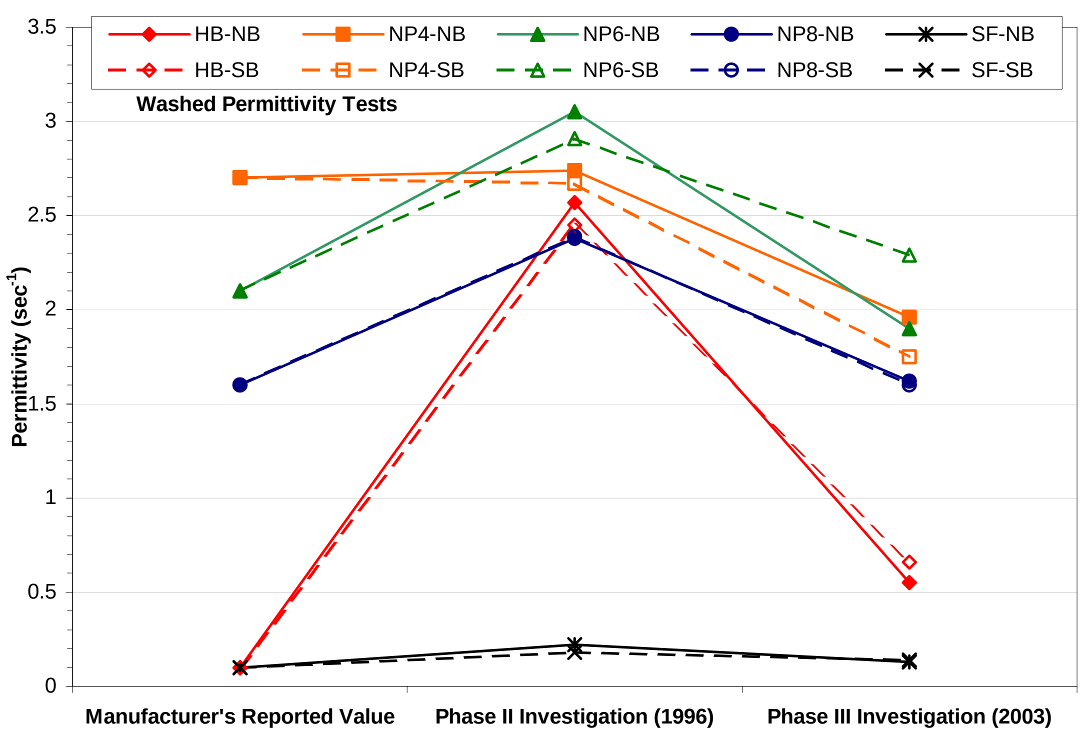

Figure 5.11. Summary of permittivity test results on washed exhumed specimens.

Figures 5.10 and 5.11 seem to be most useful for evaluating the filtration and drainage performance of the geotextile separators. The manufacturers' reported values were used for these figures because of the variability in results between the Phase II and Phase III test results for the virgin specimens. From Figure 5.11 it can be seen that all of the washed specimens met or exceeded the manufacturers' reported permittivity values at the time of the Phase II investigation. Then the permittivity of all of the washed specimens dropped somewhat in the period between the Phase II and Phase III investigation. This is most likely due to more extensive clogging of the geotextiles that would not easily wash away.

As shown in Figure 5.11, the HB specimens had the most significant drop in permittivity from the Phase II to the Phase III investigation. During the Phase III permittivity test, most of the HB specimens were clogged throughout with iron-stained particles that would not rinse out during the washing process. As a result, the percentage increase in permittivity after washing for the HB specimens shown in Table 5.2 dropped significantly from the Phase II to the Phase III investigation. The iron-stained particles that seemed to clog the HB specimens appeared less extensively in the NP and SF specimens. Consequently, these specimens rinsed much cleaner during the washing process.

Table 5.2 shows that the increase in permittivity of the needle-punched nonwoven specimens after washing increased significantly from the Phase II to the Phase III investigation, indicating that more clogged particles were rinsed from the specimens. These observations tend to confirm the conclusion that more clogging of the nonwoven geotextiles occurred in the period between the Phase II and Phase III investigation. This is not a surprise because none of the geotextiles meet the filtration criteria for all of the D_{85} values of the tested soils. No significant change in permittivity was observed in the exhumed slit film specimens between the manufacturer's reported value, the Phase I investigation, and the Phase II investigation for both the unwashed and washed test results.

Although the nonwoven geotextiles seemed to be clogged, almost all permittivity test results exceeded the 0.1 sec^{-1} permittivity value determined by the FHWA design procedure. The only exception was for the unwashed slit film, whose test results fell below 0.1 sec^{-1} in both the Phase II and III investigations.

5.2.4 Durability

The durability of geotextiles can be evaluated by measuring the change over time in ultimate tensile strength and percentage of elongation at failure. Wide width tensile tests conducted during the Phase II and Phase III investigations were compared to the manufacturers' reported strength and elongation values given in Table 1.1. Tensile tests were also conducted on 4-in (100-mm) wide specimens to eliminate some of the sources of testing error and to better compare the tensile strengths of the Phase II and Phase III exhumed specimens. Table 5.4 summarizes the results of the wide width tensile tests for the Phase II and Phase III investigations in comparison with the manufacturers' reported values.

The results shown in Table 5.4 do not seem to make much sense. On average, the Phase III tests revealed tensile strengths that were about 23 percent lower than the results of the Phase II tests. This strength loss does not agree with other exhumed strength tests on polypropylene polymer geotextiles (Elias, 2001; Christopher and Valero, 1999;

Salman et al., 1997; and Brorsson and Eriksson, 1986). The literature shows that strength loss primarily occurs during construction, and further strength loss in polypropylene geotextiles due to aging degradation is negligible over the service life of the geotextile.

Table 5.4. Summary of wide width tensile tests: Phase II and Phase III.

	Manufacturers' Reported Values[1]		Exhumed Samples – Phase II (1996)		Exhumed Samples – Phase III (2003)	
Section	**Tensile Strength, lb/in (kN/m)**	**Percent Elongation at Failure**	**Tensile Strength, lb/in (kN/m)**	**Percent Elongation at Failure**	**Tensile Strength, lb/in (kN/m)**	**Percent Elongation at Failure**
HB-NB	34.8 (6.1)	45	32.5 (5.7)	27	22.9 (4.0)	26
NP4-NB	50.2 (8.8)	80	41.7 (7.3)	22	32.9 (5.8)	60
NP6-NB	70.2 (12.3)	95	55.4 (9.7)	24	50.1 (8.8)	49
NP8-NB	90.2 (15.8)	95	74.2 (13.0)	28	57.4 (10.1)	46
SF-NB	174.7 (30.6)	15	165.6 (29.0)	12	145.7 (25.5)	13
HB-SB	34.8 (6.1)	45	42.3 (7.4)	35	26.9 (4.7)	26
NP4-SB	50.2 (8.8)	80	51.4 (9.0)	28	31.5 (5.5)	46
NP6-SB	70.2 (12.3)	95	68.0 (11.9)	35	56.4 (9.9)	59
NP8-SB	90.2 (15.8)	95	82.2 (14.4)	31	68.0 (11.9)	54
SF-SB	174.7 (30.6)	15	181.0 (31.7)	14	137.3 (24.0)	13

[1]See footnotes in Table 1.1.

The percentage of elongation at failure for the Phase II specimens was about 50 percent *lower* on average than the manufacturers' reported values. The Phase III specimens had an average elongation at failure that was about 50 percent *higher* than that of the Phase II specimens, and the Phase III results were about 35 percent lower than the manufacturers' reported values.

The elongation at failure of the HB specimens showed a different pattern than the NP specimens; the elongation at failure decreased between the Phase II and Phase III tests for the HB specimens, whereas the NP specimens showed significant increase in elongation at failure from Phase II to Phase III. The elongation at failure remained essentially unchanged for the SF woven specimens.

The drastic changes in strength and percentage of elongation at failure can probably be explained by examining the testing procedures. The Phase II wide width tensile test specimens were soaked for at least 24 hours before testing, but they were not washed or rinsed. The Phase III test specimens were lightly washed in accordance with Elias's (2001) recommendations and then soaked for 24 hours before testing. Thus the Phase II specimens had a much higher percentage of clogged particles during testing. It is possible that these particles within the matrix of the geotextile restricted the elongation of the fibers during the test. During a wide width tensile test, the fibers elongate in the direction of pull. The soil particles trapped between the fibers would inhibit the elongation, causing failure to occur at lower elongation. That is, the clogged particles could "confine" some of the fibers, causing them to fail at higher strengths and resulting in higher ultimate strengths at a lower percentage of elongation. Another possibility is that the geotextiles were damaged during the washing process, which may explain the lower ultimate strength values for the Phase III tests (washed) in comparison to the Phase II tests (unwashed).

To eliminate some of the above questions, 4-in. (100-mm) strip tensile tests were conducted on archived Phase II specimens and Phase III specimens. The results of the 4-in. strip tests are presented in Chapter 4. In comparison to the manufacturers' reported values, Figure 5.12 shows the Phase II results for ultimate tensile strength, and 5.13 shows the results for percentage of elongation.

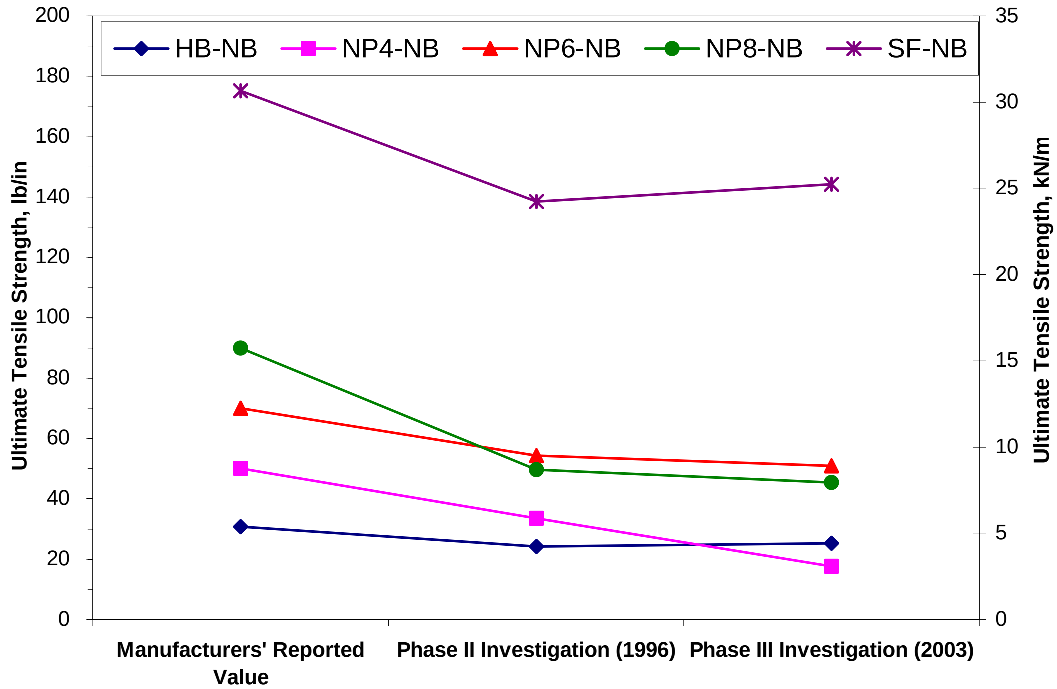

Figure 5.12. Change in ultimate tensile strength over time, 4-in. strip test.

Figure 5.12 shows that the strength loss primarily occurred during construction. Figure 5.13 shows that a general decrease in percentage of elongation at failure occurred during construction and that very little change has occurred in the period since construction. The NP4 test results appear to be an exception to these generalizations. Unfortunately, the sample quantity was very limited, and only broad generalizations can be made from the data. These data are in agreement with the previously mentioned studies by Elias (2001), Christopher and Valero, (1999), Salman et al. (1997), and Brorsson and Eriksson (1986), which showed that the loss of strength in polypropylene polymer geotextiles primarily occurs during construction.

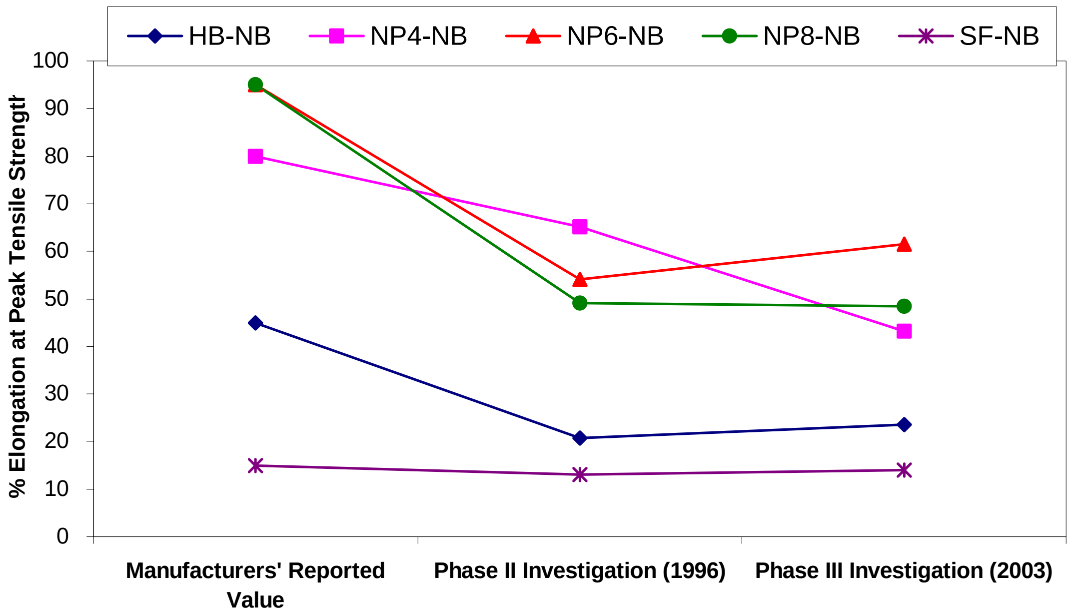

Figure 5.13. Change in percent elongation at failure over time, 4-in. strip test.

Overall, the durability of the geotextiles in the long term appears to be good, but because of scatter in the test results, it is not possible to come to any definite conclusions or to make any correlations.

5.3. SUMMARY

Results of the Phase III field and laboratory tests were analyzed and compared to results from the investigation during construction (Phase I) in 1991 and the first investigation following construction (Phase II) in 1996. Density test results showed that some consolidation might have occurred in the period between construction and the Phase I investigation. Less than 0.5 in. (12 mm) of intermixing was found at the base course/subgrade interface in the soil-only sections, indicating that the pavement sections are too thick to quantify the benefits of the geotextiles at this point in their service life. The permittivity tests indicated that the geotextiles had more clogged particles during the Phase III investigation than they did during the Phase II investigation; however, almost all of the geotextiles exceeded the permittivity value determined by the FHWA design procedure.

6. FALLING WEIGHT DEFLECTOMETER TESTING

As part of the research at the Bucoda test site, WSDOT periodically performed falling weight deflectometer (FWD) testing over the 12-year period. The results were analyzed by using two interpretation methods, the FWD Area Program (version 2.0) and Evercalc Pavement Backcalculation Program (version 5.20). WSDOT uses both of these programs to analyze FWD deflection data to evaluate the structural condition of the pavement section. The results are compared to the findings of the field and laboratory investigations.

6.1. FWD TESTING PROCEDURES

WSDOT has been using the Dynatest FWD since 1983 and about 400 to 600 lane mi (640 to 970 km) are tested each year at 250-ft (76-m) spacings (Pierce, 1999). The Dynatest 8000 FWD is a trailer-mounted, non-destructive testing (NDT) device that delivers a transient impulse load to the pavement surface (WSDOT Pavement Guide, Vol. 2, 1995). The test method is ASTM D 4694, "Standard Test Method for Deflections with a Falling Weight Type Impulse Load Device." The loading plate has a standard diameter of 12 in. (300 mm), and different drop heights and weights are used to vary the impulse load. To complete a test at one location, four drops are made at loads of about 6000 lb (27 kN), 9000 lb (40 kN), 12,000 lb (53 kN), and 16,000 lb (71 kN). Velocity transducers are used to measure the pavement response at distances of 0 in. (0 mm), 8 in. (203 mm), 12 in. (305 mm), 24 in. (610 mm), 36 in. (914 mm), and 48 in. (1219 mm) from the point of load application. Figure 6.1 shows the layout of velocity transducers used by WSDOT. The peak deflections are computed for all four drops.

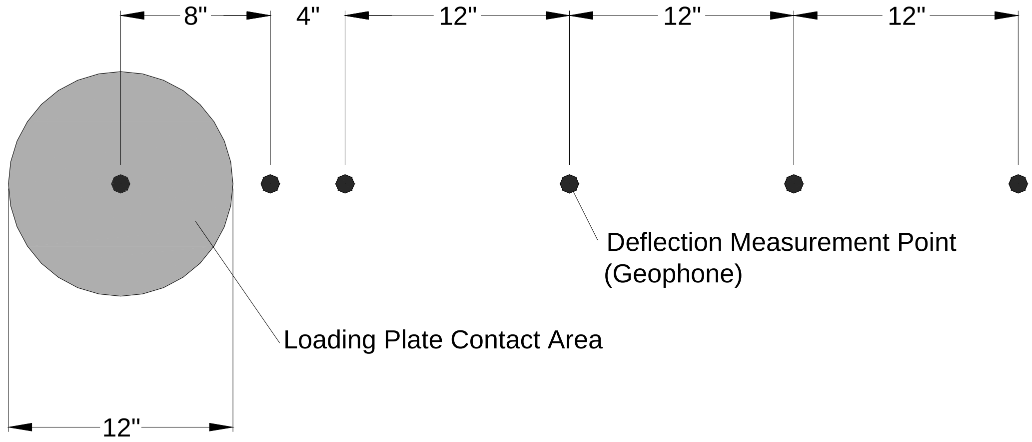

Figure 6.1. Typical location of velocity transducers used by WSDOT for FWD measurements (from WSDOT Pavement Guide, Vol. 2, 1995).

A photograph of WSDOT's Dynatest 8000 FWD trailer and van is shown in Figure 3.4.

WSDOT performed FWD testing with the Dynatest before construction and periodically over the 12 years since reconstruction on the following dates:

- April 2, 1991 – before reconstruction
- July 24, 1991 – 48 days after reconstruction
- November 25, 1991 – 172 days after reconstruction
- March 25, 1996 – 5 years after reconstruction
- October 4, 2000 – 9 years after reconstruction
- August 12, 2003 – 12 years after reconstruction

The results of the FWD tests are tabulated in Appendix E.

6.2. FWD BACK-CALCULATION AND ANALYSIS

The results of the FWD tests were analyzed with two interpretation methods, the FWD Area Program (version 2.0) and Evercalc Pavement Backcalculation Program (version 5.20).

6.2.1. FWD Area Program

The FWD Area Program is a quick method of evaluating the structural condition of the roadway. The FWD area parameter represents the normalized area of a vertical slice taken through a deflection basin between the center and 36 in. (914 mm) away from the center of the test load (Pierce, 1999). Higher area values imply a “stiffer” pavement structure. The Area Program calculates the subgrade modulus by using the deflection at 24 in. (610 mm) from the load in a deflection basin normalized to a 9000-lb (40-kN) load. Newcomb (1986) developed regression equations to calculate the subgrade modulus.

The area value is determined from the following equations (Pierce, 1999):

In English units:

$$Area = \frac{6\left(D_0 + 2D_{12} + 2D_{24} + D_{36}\right)}{D_0}$$

where: D_0 = Deflection at the center of the load,

D_{12} = Deflection 12 in from the load,

D_{24} = Deflection 24 in from the load, and

D_{36} = Deflection 36 in from the load.

In metric units:

$$Area = \frac{150\left(D_0 + 2D_{300} + 2D_{600} + D_{900}\right)}{D_0}$$

where: D_0 = Deflection at the center of the load,

D_{300} = Deflection 300 mm from the load,

D_{600} = Deflection 600 mm from the load, and

D_{900} = Deflection 900 mm from the load.

Table 6.1 summarizes typical area values for various pavements.

Table 6.1. Typical area values (Pierce, 1999)

Pavement	Area in.	Area mm
PCCP[1]	24 – 33	610 – 840
"Sound" PCCP[1]	29 – 32	740 – 810
Thick ACP[2], ≥ 4 in (100 mm)	21 – 30	530 – 760
Thin ACP[2], < 4 in (100 mm)	16 – 21	410 – 530
BST[3], relatively thin structure	15 – 17	380 – 430
Weak BST[3]	12 – 15	300 – 380

Notes:
1. PCCP = Portland Cement Concrete Pavement.
2. ACP = Asphalt Cement Pavement.
3. BST = Bituminous Surface Treatment.

Figures 6.2 and 6.3 show the adjusted deflections for the northbound and southbound lanes, respectively. The adjusted deflections are the deflections at the center of the loading plate normalized to a 9000-lb (40-kN) load and modified to account for pavement thickness and temperature.

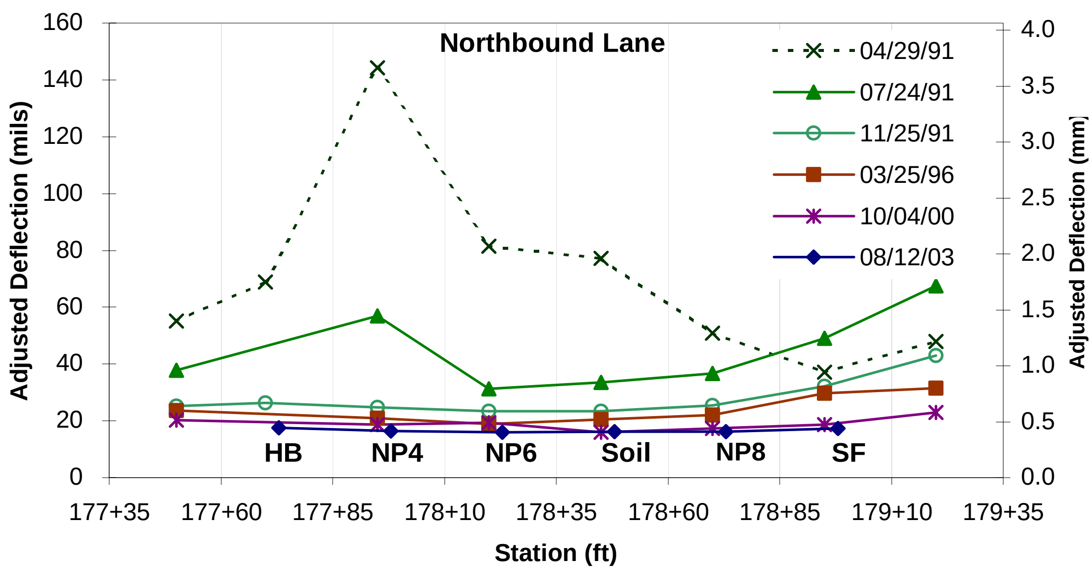

Figure 6.2. Adjusted deflections, northbound lane.

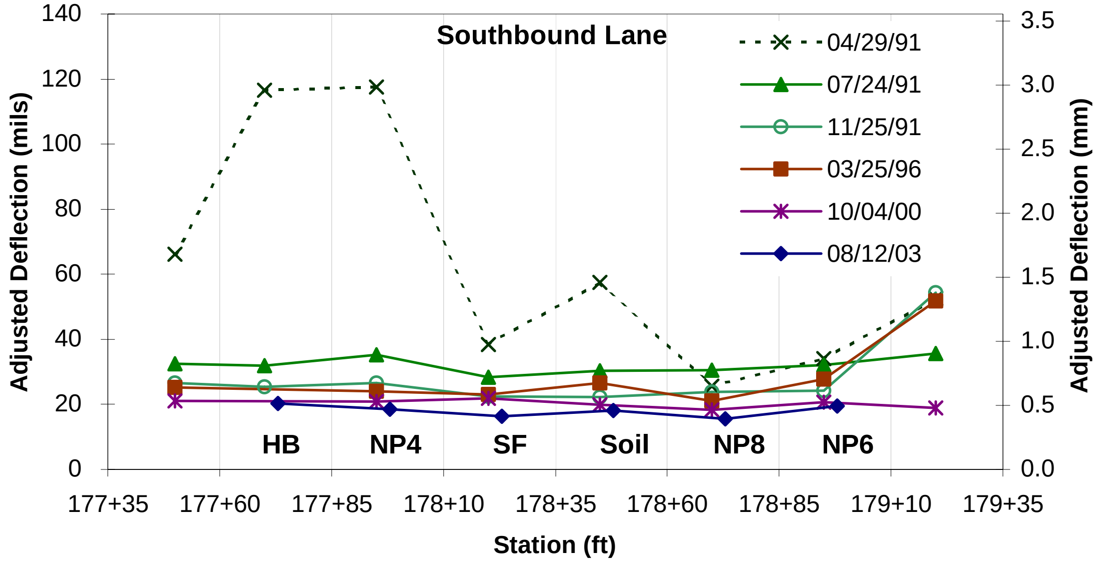

Figure 6.3. Adjusted deflections, southbound lane.

The general trend was a decrease in the adjusted deflection with time. The adjusted deflections decreased most in the first few months following reconstruction, and a relatively small decrease can be seen in the three years between the last two series of FWD tests.

Figures 6.4 and 6.5 show the area parameters for the northbound and southbound lanes, respectively. The area parameter generally increased with time over the 12-year period. As was seen with the adjusted deflection values, the largest change in the area parameter occurred in the first few months after construction. In fact, the area parameter increased from about 15 to 24 in. (400 to 600 mm) over the 12-year period—an indication of substantial stiffening of the pavement structure. The trend of decreasing adjusted deflection values and increasing area parameters is an indication that both the pavement structure and subgrade gained strength over time. This is in agreement with the subgrade moduli calculated by the Area Program for the northbound lanes (Figure 6.6) and southbound lanes (Figure 6.7).

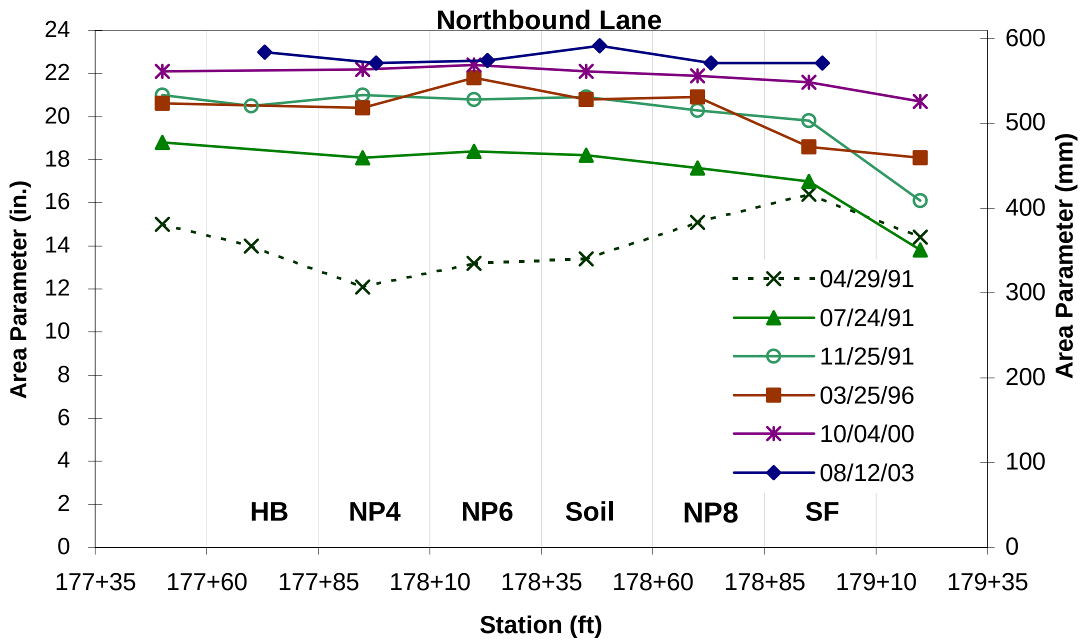

Figure 6.4. FWD area parameter, northbound lane.

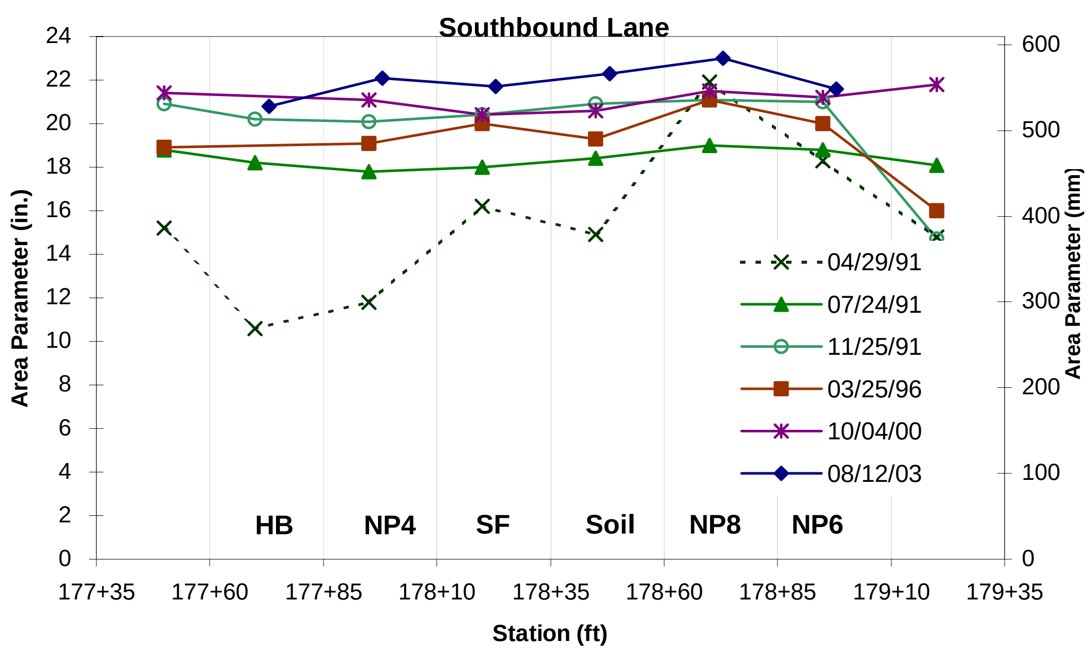

Figure 6.5. FWD area parameter, southbound lane.

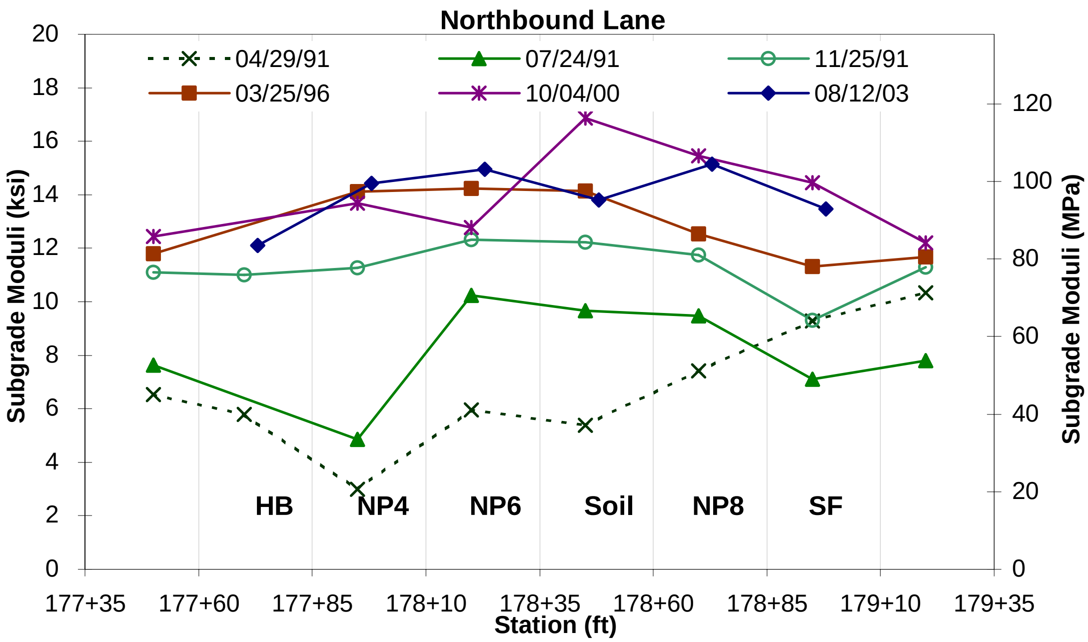

Figure 6.6. Subgrade moduli calculated by Area Program, northbound lane.

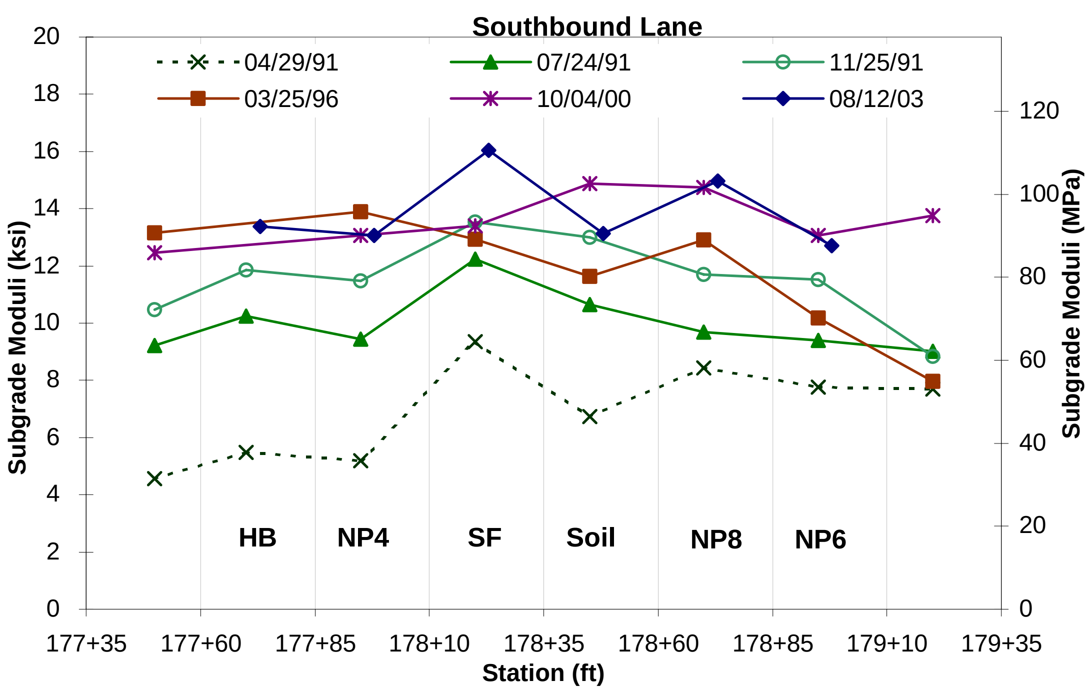

Figure 6.7. Subgrade moduli calculated by Area Program, southbound lane.

6.2.2. Evercalc Pavement Backcalculation Program

Evercalc is a program that uses FWD deflection data to back-calculate elastic moduli of pavement section layers. The Evercalc program performs iterations of a layered elastic analysis and compares the theoretical deflections to the measured deflections until the specified error is within tolerance, or the number of iterations has reached a limit (WSDOT Pavement Guide, Vol. 3, 1999). The program outputs the moduli for each of the pavement section layers and the underlying subgrade.

Figures 6.8 and 6.9 are the back-calculated asphalt concrete (AC) moduli adjusted to a temperature of 77ºF (25ºC) and a 9000-lb (40-kN) load. The significant increase in the moduli in the period after construction may be attributed, in part, to compaction of the AC by traffic and aging of the asphalt binder in the hot mix.

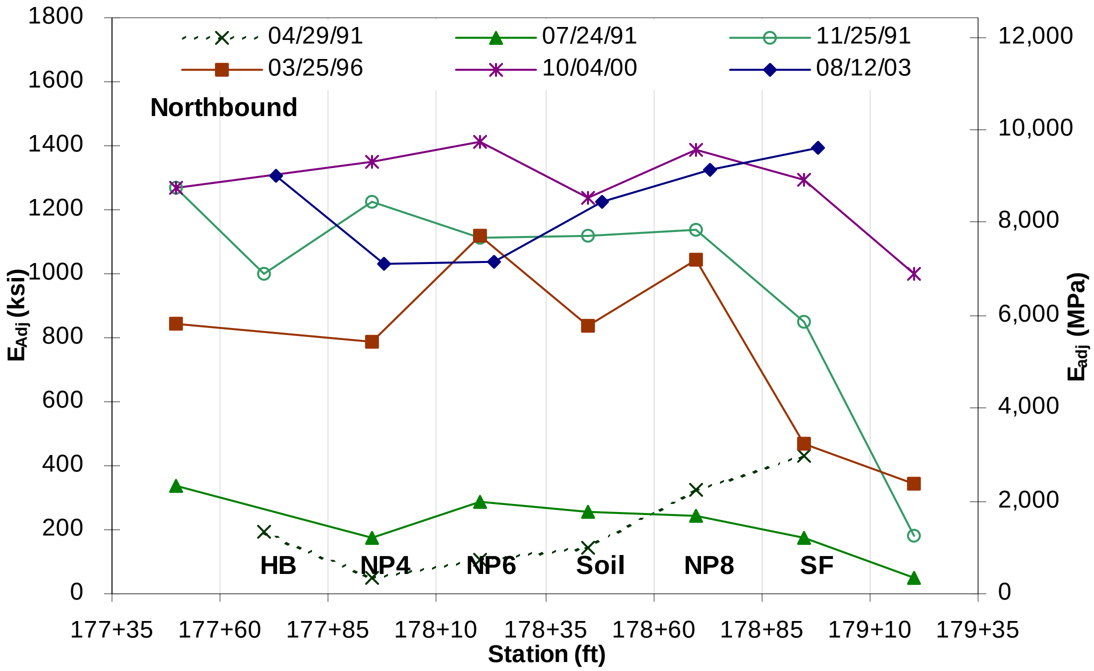

Figure 6.8. Asphalt concrete moduli, northbound lane.

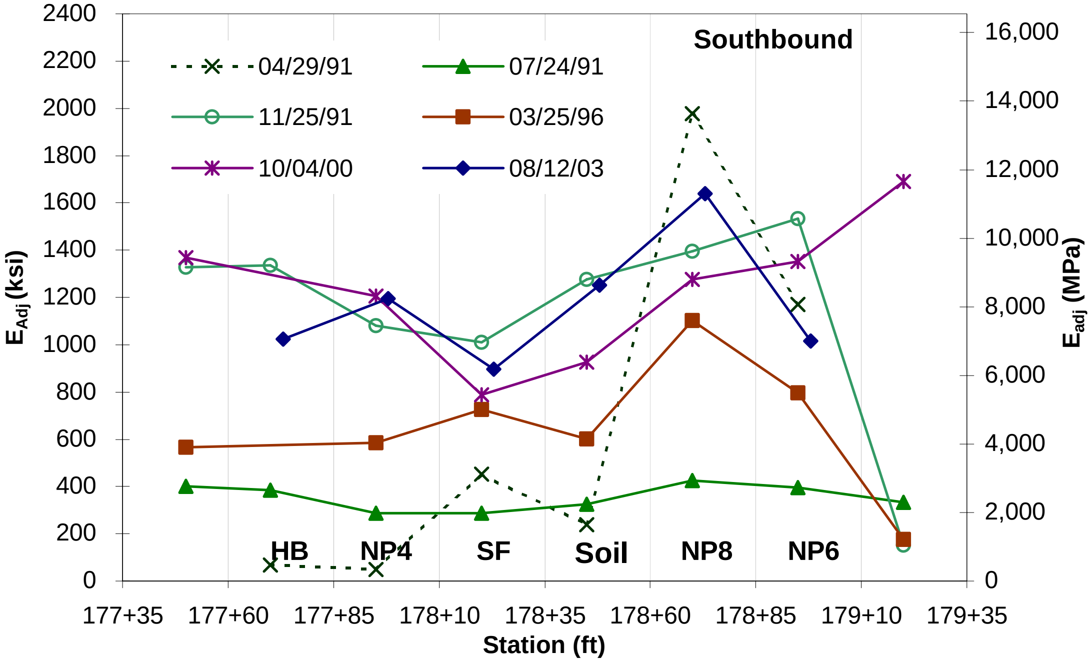

Figure 6.9. Asphalt concrete moduli, southbound lane.

Figures 6.10 and 6.11 show the change in the crushed stone base course moduli over the 12-year period since reconstruction. The figures suggest the possibility that the sections with geotextiles below the base course may have become stiffer than the soil-only sections, but the results are, at best, erratic. This is an indication that the geotextile separators may also have been reinforcing the pavement section.

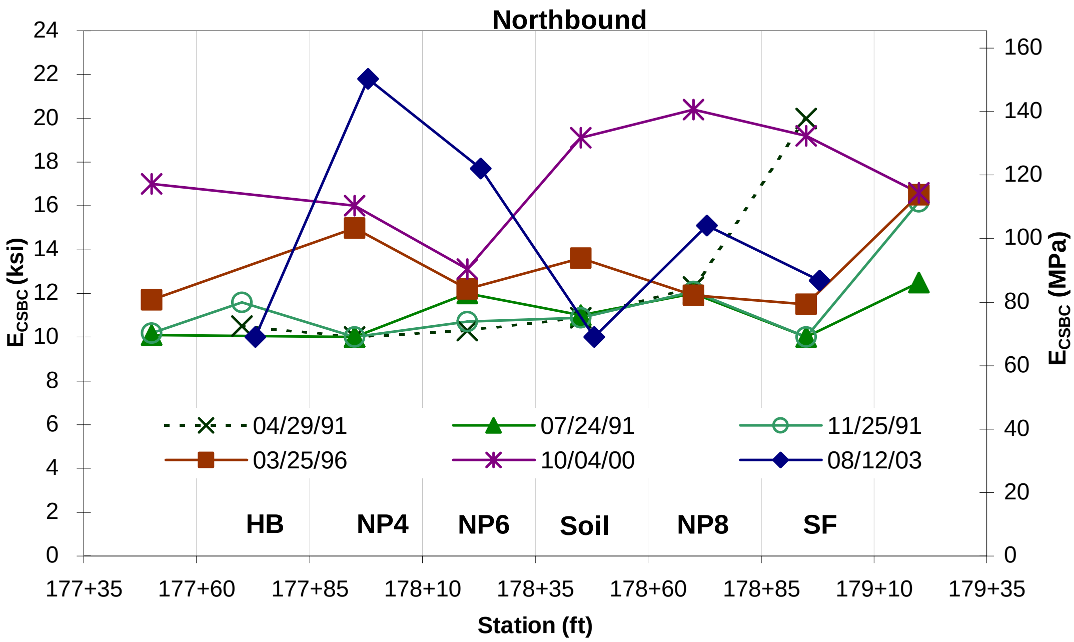

Figure 6.10. Base course moduli, northbound lane.

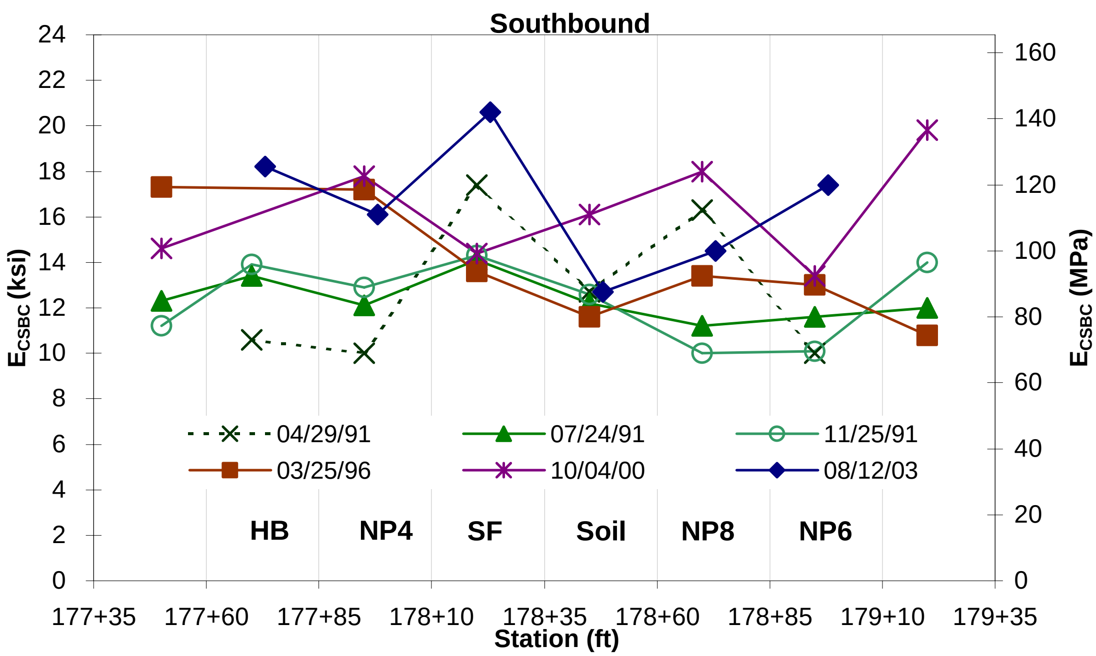

Figure 6.11. Base course moduli, southbound lane.

Figures 6.12 and 6.13 show the change in subgrade moduli over the 12-year period. The results are similar to what was found by the Area Program. The Area Program calculated subgrade moduli about 2 to 3 ksi (15 to 20 MPa) higher than Evercalc (about 15 percent), but both programs showed an increase of 5 to 10 ksi (35 to 70 MPa) (about 40 to 80 percent), in the period since just before reconstruction. The subgrade moduli generally increased with time over the 12-year period, and the largest increase was in the first several months after construction. In the figures, the soil-only sections show a similar increase in moduli when compared with the geotextile sections, suggesting that the geotextiles did not make a difference in the performance of the pavement section.

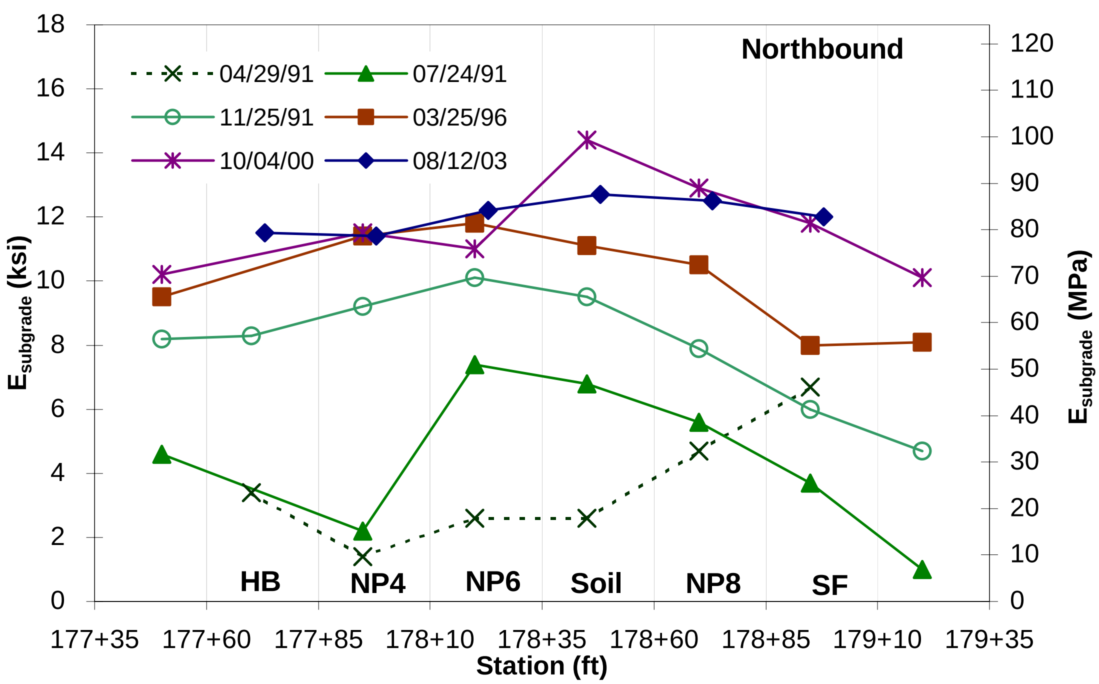

Figure 6.12. Subgrade moduli, northbound lane.

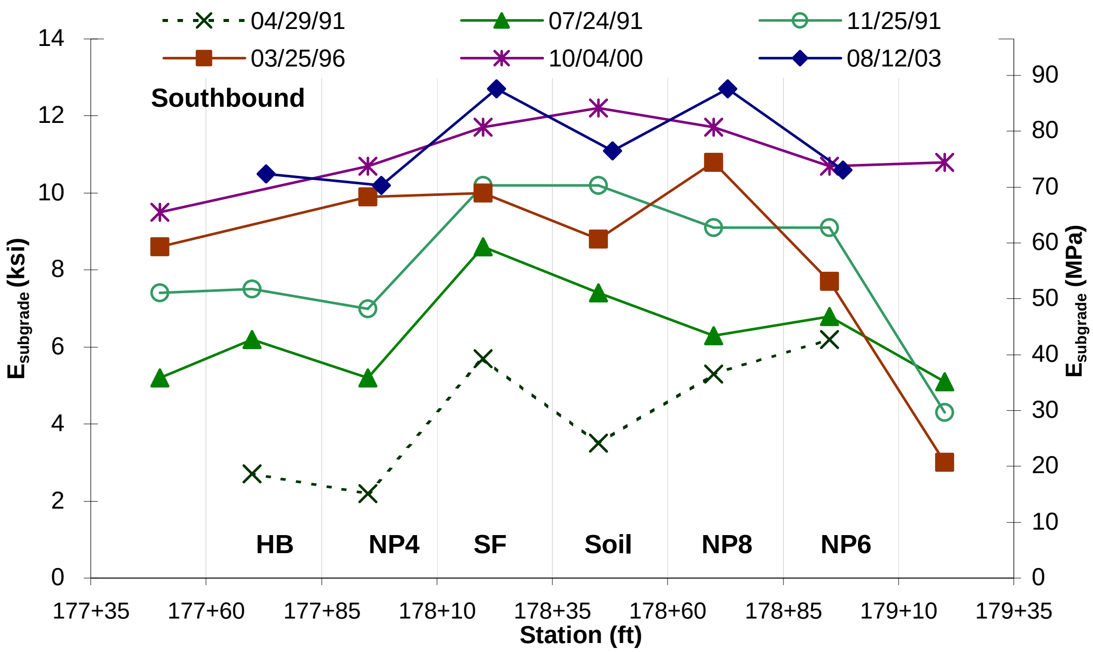

Figure 6.13. Subgrade moduli, southbound lane.

6.2.3. Comparison to Field and Laboratory Results

As discussed in Chapter 3, a pavement condition survey performed during the most recent field investigations found no rutting or cracking within the limits of the test section. A chip seal treatment had been placed in the period since 1996. Some low severity longitudinal cracking was observed just outside the limits of the test section where no geotextiles were present. Overall, both lanes appeared to be in very good condition through the test section.

The field nuclear density tests indicated that the density of some of the subgrades in the geotextile sections increased in the first 5 years after construction, but tests in the soil-only sections had the same or lower results in comparison to the results of tests performed during construction. These results corroborate the results of the FWD back-calculations, which showed that the subgrade moduli increased most dramatically in the first few months after construction and then remained about the same.

The base course was very dense, and steel pry bars were required to loosen it during the 1996 and 2003 test pit excavations. Less than 0.5 in. (13 mm) of base course/subgrade intermixing was observed in the soil-only sections. Grain size distribution tests conducted on the base course material indicated only small or negligible fines migration (<5 percent) between the subgrade and the aggregate located immediately above the zone of intermixing.

These observations indicate that the pavement section may have been too thick to realize the separation benefits of the geotextiles in the 12 years since construction. This was also suggested by the results of some of the other field and laboratory tests. The evidence is that when geotextile separators are used, it is not necessary to “subexcavate” or “dig out” poor soils to the same depth that would be necessary if no geotextile were present, as long as the subgrade has moderate strength. The reinforcement function of the geotextiles may have contributed to an increase in the base course moduli over time. The pavement in the test section was in very good condition, thus the true “long-term” benefits of the geotextiles have yet to be quantified.

6.3. SUMMARY

The results from the FWD Area Program generally showed a decrease in the adjusted deflections and increases in the area parameter and subgrade modulus along the test section over the 12-year period. The greatest increase in subgrade modulus occurred in the 6 months following construction. The control sections exhibited behavior similar to the sections with geotextiles, suggesting that for relatively thick pavement sections, incorporation of geotextiles into the pavement section may not provide a significant contribution to the long-term performance of the overall section.

The Evercalc back-calculations indicated a general increase in the adjusted pavement and subgrade moduli over the 12-year period. The sections that contained geotextiles generally showed an increase in the base course modulus; however, some of the analyses showed little to no increase in the base modulus in the soil-only control sections. The results suggest that geotextiles might contribute to an increase in the base course modulus over time. These findings were also compared with the results of the field and laboratory investigations. The pavement conditions survey indicated that the pavement was in very good condition after being in service for more than 12 years. Density tests suggested that the subgrade might have consolidated in the period between construction in 1991 and the Phase II investigation in 1996. The FWD deflection measurements were more consistent over the 12-year period than the in situ tests performed during the field investigations.

7. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH

7.1. SUMMARY

This research was Phase III of a series of field and laboratory investigations carried out over a 12-year period at a test section on SR 507 near Bucoda, Washington. This project was part of a research effort to quantify the contribution of geotextile separators to the long-term performance of pavement sections. Five different geotextile separators as well as a soil-only control section were installed in the test section in two lanes with a different base course thickness on a low volume but heavily loaded rural highway. Phase I evaluated the performance of the separators during construction. Phase II and III investigations were conducted to evaluate the performance of the separators 5 and 12 years after construction, respectively.

Field and laboratory tests were conducted on the subgrade, granular base, and the geotextiles in an attempt correlate the performance of the pavement section to the presence of the geotextile separators. Falling weight deflectometer (FWD) testing was also performed at the site.

The subgrade along the test section was somewhat variable, but in general it consisted of high plasticity fine-grained soils (USCS soil classification: CH; AASHTO soil classification: A-7-5 and A-7-6). The moisture content was generally about 15 percent higher than the plastic limit of the soil.

Field density tests indicated that the density of the subgrade in the sections with a geotextile generally increased in the period between construction and the first field investigation, whereas the density of the soil-only sections remained about the same.

Similarly, the FWD tests showed that the most significant increase in the subgrade moduli occurred in the first few months following construction.

Mudcaking was observed on top of most of the geotextile samples exhumed during both the Phase II and Phase III field investigations. However, subsequent sampling and grain size analysis tests on the base course material immediately above of the geotextiles indicated only negligible fines migration (<5 percent) through the geotextiles. Moreover, the migrated fines were only present at the very bottom of the base course layer. This is good, considering that none of the geotextiles on the project met the FHWA recommended retention criteria for the worst case of all of the soils found at the site. Permittivity tests indicated that additional clogging occurred in the period since the Phase II investigation.

The geotextiles successfully performed as separators, preventing intermixing of soil at the base course/subgrade interface over the 12-year period. However, the soil-only sections had a minimal amount of intermixing at the base course/subgrade interface, indicating that the separation benefits of geotextiles may not be realized under relatively thick pavement sections. In addition, the soil-only sections exhibited behavior similar to the sections with geotextiles during the FWD testing, which also suggests that for relatively thick pavement sections, incorporation of geotextiles into the pavement section may not significantly contribute to the overall performance of section.

The wide width tests indicated that the geotextiles retained between 60 percent and 80 percent of the manufacturers' reported strength since installation and between about 50 percent and 90 percent of the manufacturers' reported percentage of elongation at failure. In general, the nonwoven specimens in the southbound lane, under a thicker base course than the northbound lane, had higher percentages of the manufacturers' reported strength and elongation at failure. The slit film woven specimens had higher tensile strength in the northbound lane and the same percentage of elongation at failure for both lanes.

Strip tensile tests conducted on archived specimens from the Phase II investigation and those obtained from the Phase III investigation indicated that only small changes in tensile strength and percentage of elongation at failure occurred since the Phase II investigation.

The FWD Area Program was used to calculate adjusted deflection and the area parameter, and to back-calculate subgrade moduli values based on the deflection data. In general, adjusted deflections decreased, while area parameters and subgrade moduli increased over time along the test section. The Evercalc Pavement Backcalculation Program was used to back-calculate moduli for the asphalt concrete, base course, and subgrade. The asphalt concrete and subgrade moduli generally increased in all of the sections with time. The base course moduli seems to have increased more in the sections with geotextiles than in the soil-only sections, suggesting that geotextiles might contribute to an increase in the base course modulus over time.

7.2. CONCLUSIONS

1. The Bucoda test section, which was constructed on a section of road that had a historically poor record of pavement performance, is still performing well after 12 years in service. The successful performance of the section may be at least partly attributed to the presence of geotextile separators.

2. Consolidation and an increase in stiffness (elastic modulus) occurred in the subgrade between construction and the first field investigation 5 years later. However, neither has changed significantly since that time.

3. FWD testing can be successfully used to evaluate the long-term performance of pavement sections with geotextile separators and may be able to detect differences between pavement sections with geotextiles and the same sections without a geotextile.

4. Geotextiles separators can provide reinforcement of the base course that may contribute to an increase in base course moduli over time. However, for relatively thick pavement sections, incorporation of geotextiles into the section may not provide a significant contribution to the overall performance of the pavement section over a 12-year period.

5. Where the subgrade has a moderate stiffness, the thickness of the stabilization aggregate (i.e., subexcavation or dig-out depth) may be reduced to account for the presence of the geotextile separator.

6. Metcalfe and Holtz (1994) concluded that lightweight geotextiles (<6 oz/yd^2 or 200 g/m^2) could survive under medium survivability conditions with rounded backfill and initial lift thicknesses of greater than 12 in. (300 mm). The lightweight NP4 and HB geotextiles installed with an initial lift thickness of 12 in. (300 mm) of subrounded to subangular base course were able to survive construction.

7. The performance of the lightweight geotextiles (HB and NP4) was comparable to the heavier weight geotextiles in the separation, filtration, and drainage functions

after 12 years in service. The lightweight geotextiles had similar losses in strength and percentage of elongation similar to those of the heavier geotextiles over the 12-year period.

8. More than two thirds of the hydrometer tests conducted during the three phases of investigation indicated that the geotextiles did not meet the FHWA retention criteria (AOS < $1.8D_{85}$ for nonwovens; AOS < D_{85} for wovens; Holtz et al., 1998). However, in spite of this, the amount of mudcaking observed in the field was less than ½ in. (13 mm) in all of the test pits, and the grain size distribution tests indicated that a minimal amount of fines (<5 percent) had passed through the geotextiles. This is an indication that the current retention criterion is very conservative, although it cannot be quantified because there was no “failure” of the geotextiles with regards to the retention function.

7.3. RECOMMENDATIONS FOR FUTURE RESEARCH

It is highly recommended that WSDOT continue to evaluate the performance of the geotextiles and the pavement section at the Bucoda test site. The next phase should conduct a third series of field investigations at the site when the section has been in service for about 20 years (around the year 2011). Waiting for the sections to deteriorate and quantifying the geotextile properties at that time will provide the most valuable information. A final field investigation should be conducted before any major reconstruction of the section. Continuation of the FWD testing is highly recommended at least an annually. The FWD tests should be conducted in the inside wheel paths about 5 ft (1.5 m) to the north (towards Bucoda) of the 2003 test pit locations.

Some attempt should be made to determine the long-term water content behavior of the test section, perhaps with periodic test pits, but the installation of tensionmeters in the subgrade also should be seriously considered.

The Bucoda Test Section is one of ten sites that are part of a nationwide study to assess the performance of geotextile separators (Koerner, 2000). The information in the database should be updated periodically, and additional sites should be added to the database. Information in the database should be used to better quantify allowable reductions in stabilizing aggregate (i.e., subexcavation or dig-out depth) when a geotextile separator is incorporated into the pavement section.

To date, a significant amount of research has been conducted to evaluate the performance of geotextile separators. However, long-term data quantifying the contribution of the geotextiles to the pavement section is still lacking. This research has provided some additional insight regarding the long-term performance of geotextile separators. The best way to truly quantify the long-term contribution of geotextiles to the pavement section is to construct test sections similar to the Bucoda test section and evaluate their performance throughout their design life.

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APPENDIX A

Field Investigation Procedure

FIELD INVESTIGATION PROCEDURE

The procedure used during the field investigation was described in Chapter 3. Numerous photographs that were taken during the test pit excavations are also shown in Chapter 3. The test pit excavation procedure is described in detail below. The procedure was adopted from the procedures of Black and Holtz (1997), Metcalfe and Holtz (1994), and Holtz and Page (1991).

The test pit excavation procedure consisted of:

1. Photograph test pit location.

2. Record visual observations of pavement condition in the vicinity of the test pit location. Draw sketch of any cracking. If ruts are present, measure and record rut depth.

3. Perform FWD testing at test pit locations (WSDOT FWD Crew).

4. Remove asphalt concrete material from test pit location using grinding machine (attached to backhoe).

5. Begin removing the base course material at the northwest corner of the test pit from an area about 1 ft^2 (0.1 m^2). This should restrict damage resulting from the digging to one corner of the geotextile. Continue digging in the corner of the excavation until the geotextile is encountered.

6. Obtain about a 3.5 lb (1500 g) sample from upper 12 in. (300 mm) of base course material.

7. Record visual observations of base course condition.

8. Loosen remaining base course material using shovels and steel pry bars to within 6 in. (150 mm) of geotextile. Remove loosened material using vacuum operated pump truck. Obtain a 3.5 lb (1500 g) base course sample from about 6 in. (150 mm) above the geotextile. Remove last 4 to 6 in. (100 to 150 mm) of base course

material by hand (do not use vacuum to prevent it from changing the properties of the geotextile). Obtain a 2 lb (1000 g) base course sample from within about 0.75 in. (20 mm) above the geotextile (mudcake).

9. Photograph test pit prior to removal of geotextile. Document the depth of the geotextile. Do not stand on the geotextile after exposing it in the test pit excavation. If standing in the excavation is required, stand near the perimeter to limit damage to the geotextile.

10. Using utility knife, cut the exposed geotextile about (2 in.) 50 mm from the perimeter of the test pit.

11. Pull back one corner of the geotextile and photograph the bottom of the geotextile and the underlying subgrade.

12. Carefully remove geotextile.

13. Photograph both sides of geotextile. Use note cards to record the geotextile ID, side, orientation etc.

14. Record visual observations of the geotextile, noting any holes, folds, indentations, staining, any evidence of blinding or clogging, etc.

15. Seal geotextile inside a plastic bag and protect the bag from the sunlight. Double bag the geotextile. Fold the geotextile as few times as possible to avoid creases that may change the geotextile properties.

16. Photograph test pit after removal of geotextile.

17. Record observations of subgrade soil conditions, including soil type, color, moisture, rutting, etc.

18. Perform in-situ tests on subgrade: pocket penetrometer, torvane, and nuclear density/moisture (WSDOT field technician).

19. Collect 2 lb (1000 g) soil sample of subgrade.

20. Record the pavement thickness, base course thickness, and depth to geotextile on all four sides of the excavation.

21. Install new geotextile with similar or better properties in the test pit excavation. Overlap the edges at least 1.5 in. (40 mm). Place backfill material along the geotextile edges to hold its position.

22. Backfill test pit and place AC patch (WSDOT maintenance personnel)

APPENDIX B

Grain Size Distribution Curves

GRAIN SIZE DISTRIBUTION CURVES

The results of the sieve-hydrometer analyses are plotted on the following figures. The tests were conducted in accordance with ASTM D 422, “Standard Test Method for Particle-Size Analysis of Soils.” The sampling procedures were described in Chapter 3 and Appendix A. The test results were discussed in Chapters 4 and 5.

The following abbreviations are used in the figures:

upp = upper

BC = base course

GTX = geotextile

imm abv = immediately above

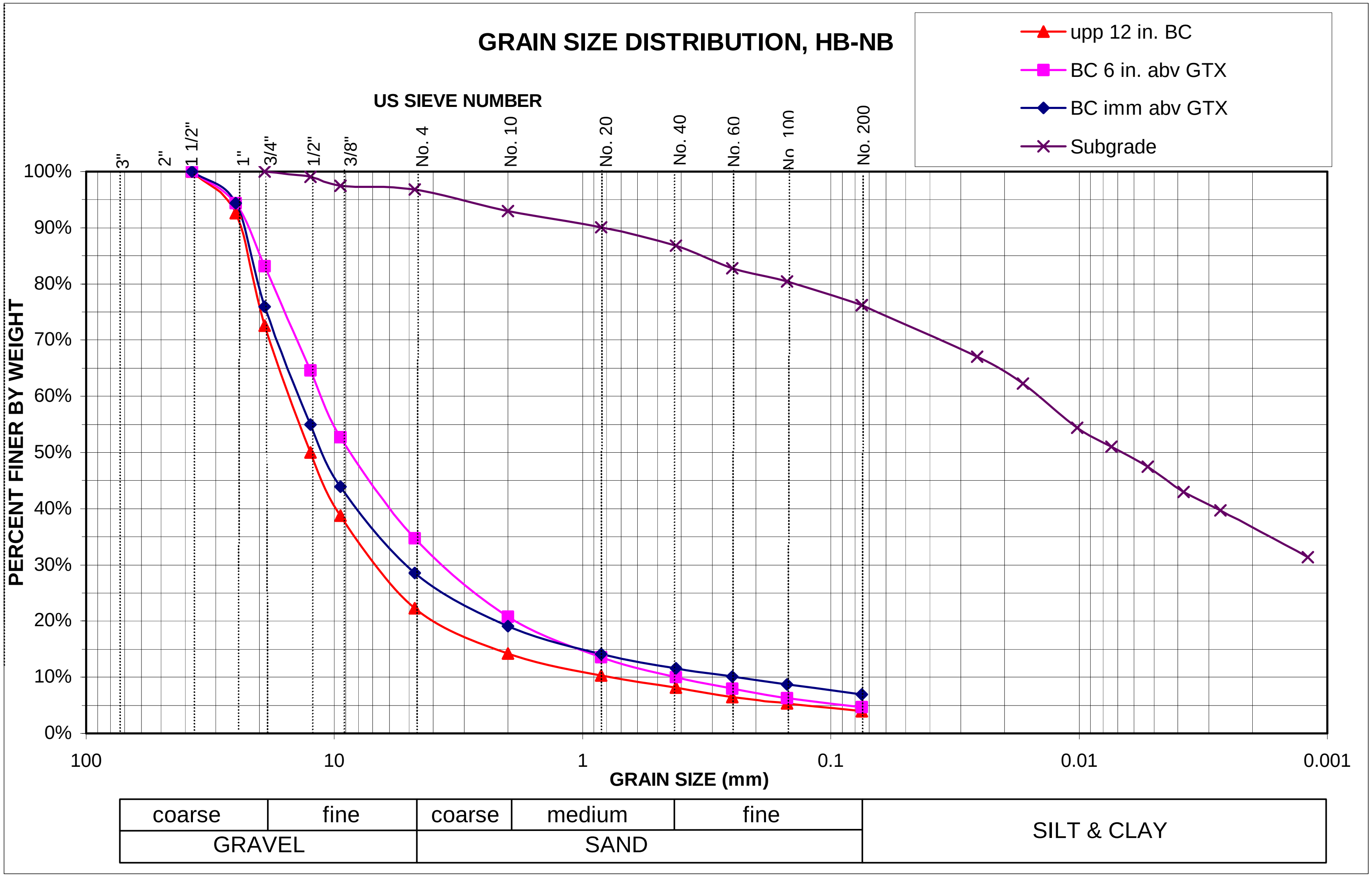

Figure B.1. Grain size distribution curves, HB-NB.

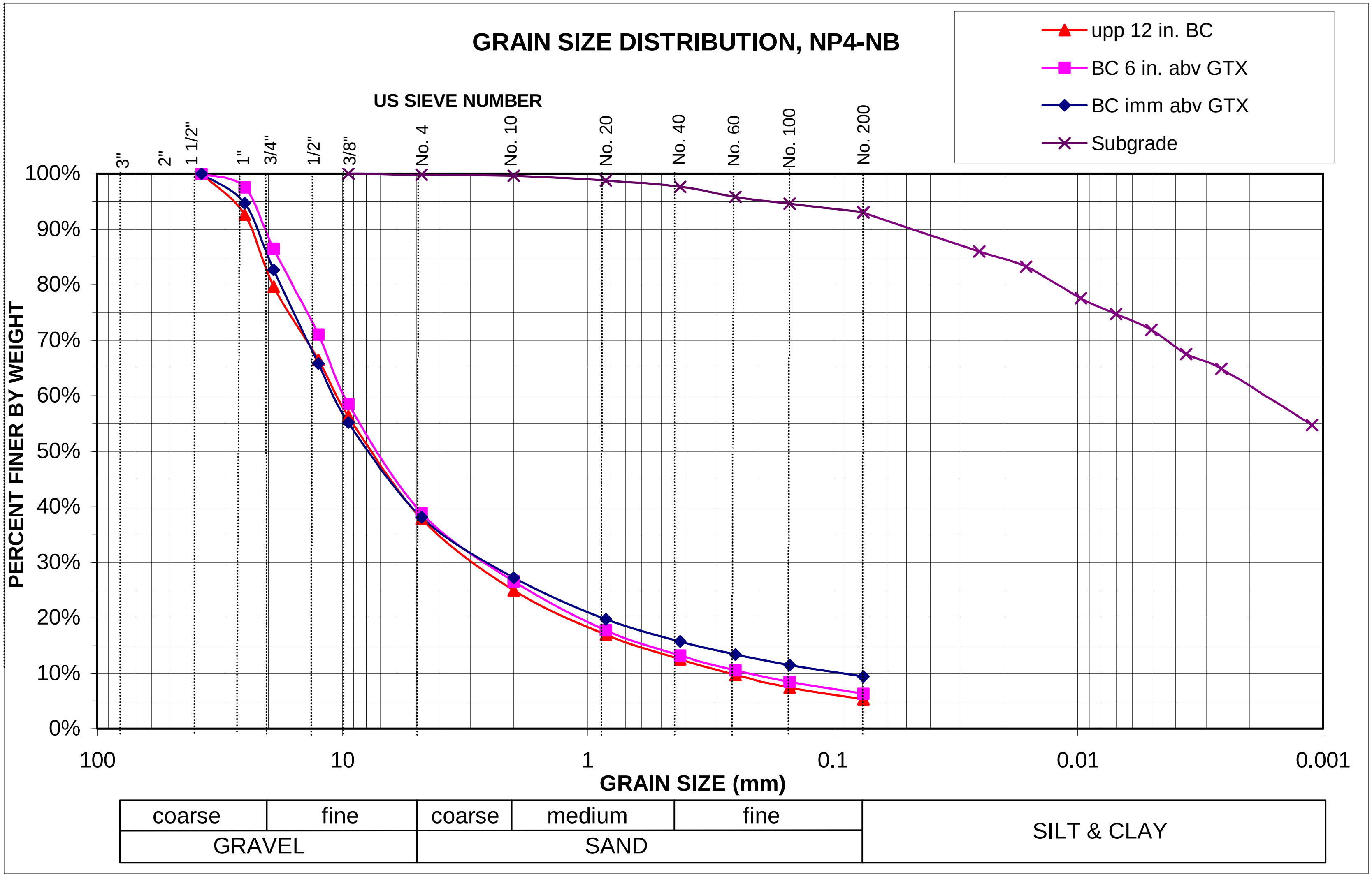

Figure B.2. Grain size distribution curves, NP4-NB.

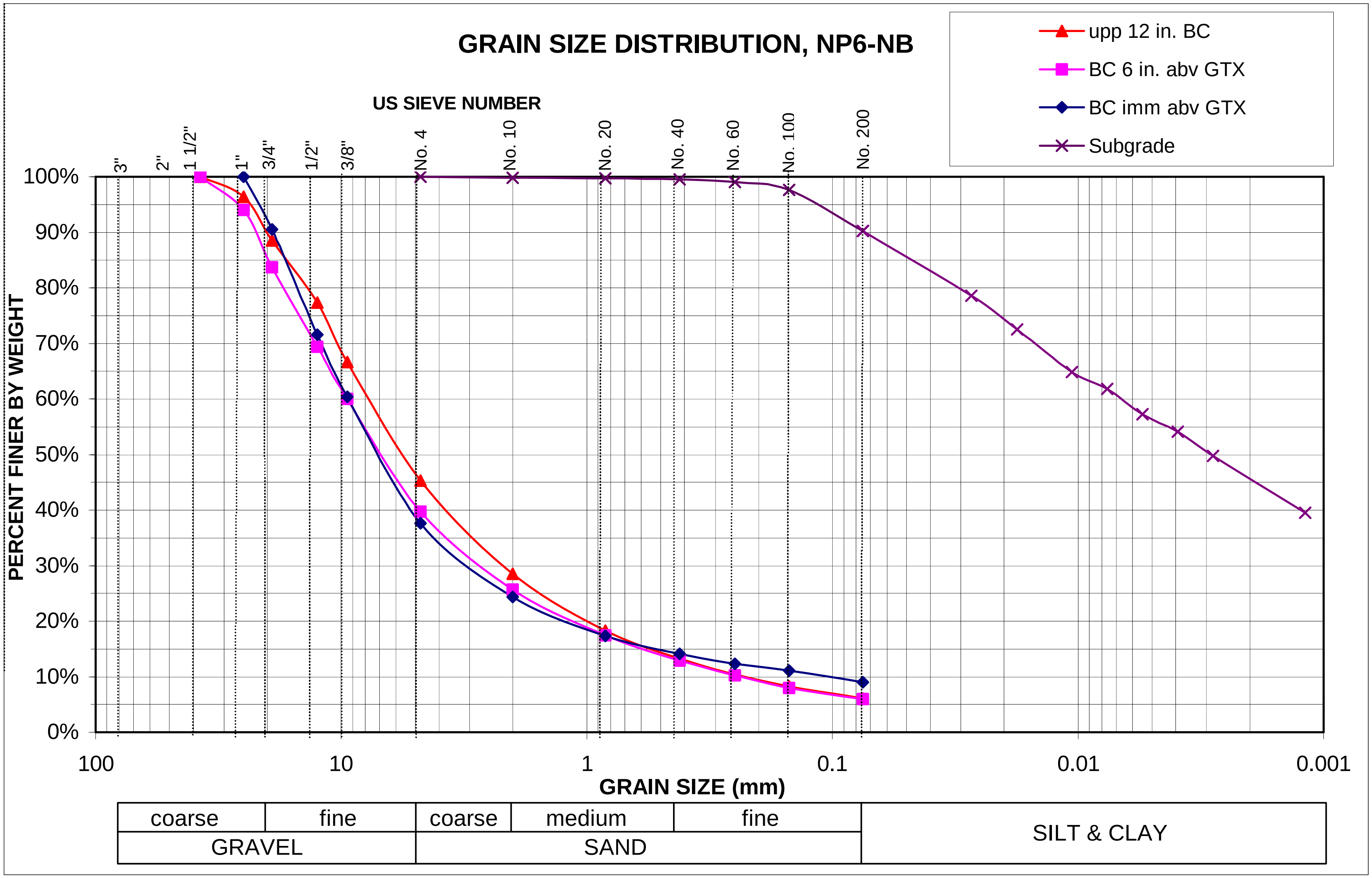

Figure B.3. Grain size distribution curves, NP6-NB.

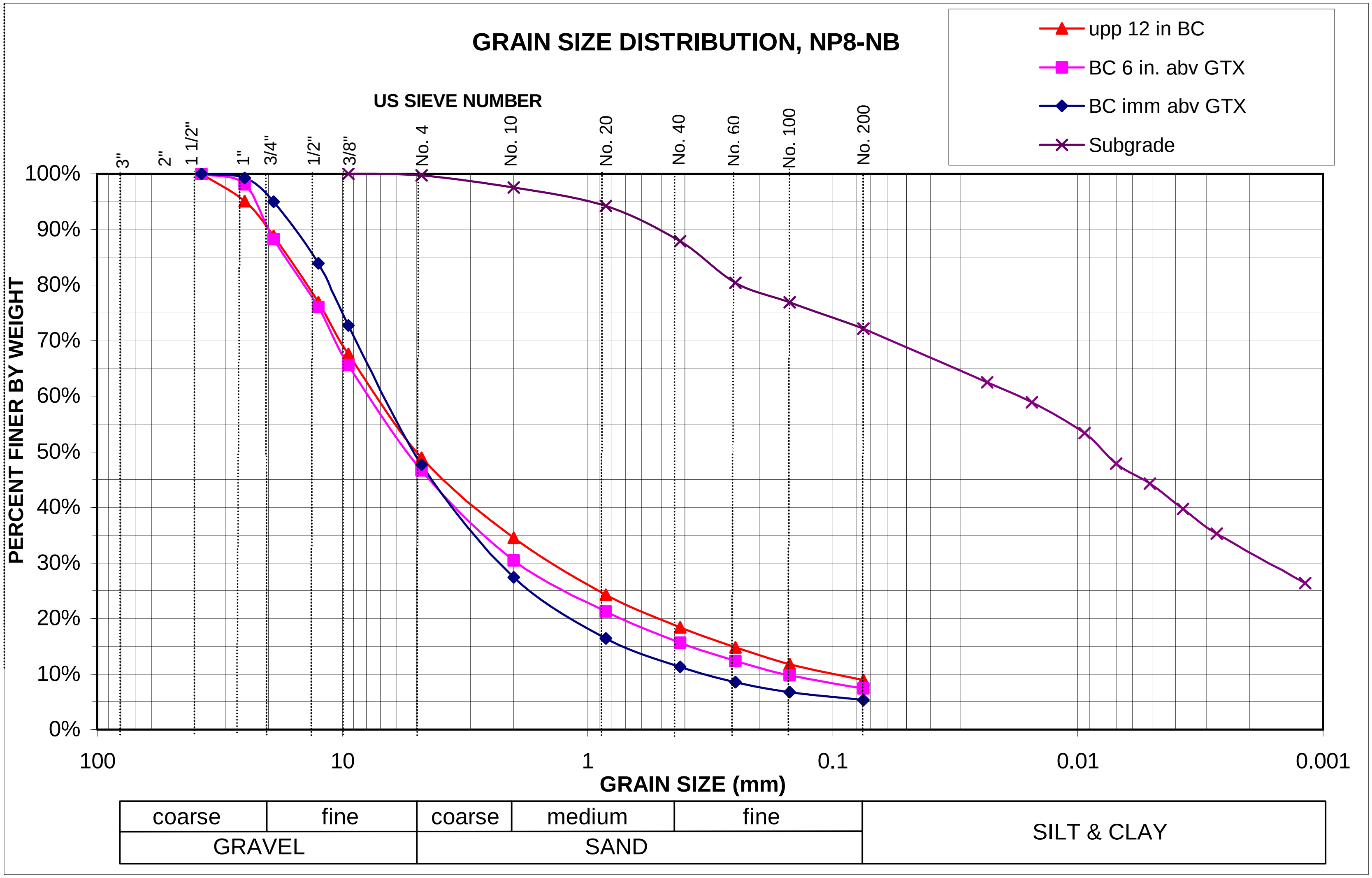

Figure B.4. Grain size distribution curves, NP8-NB.

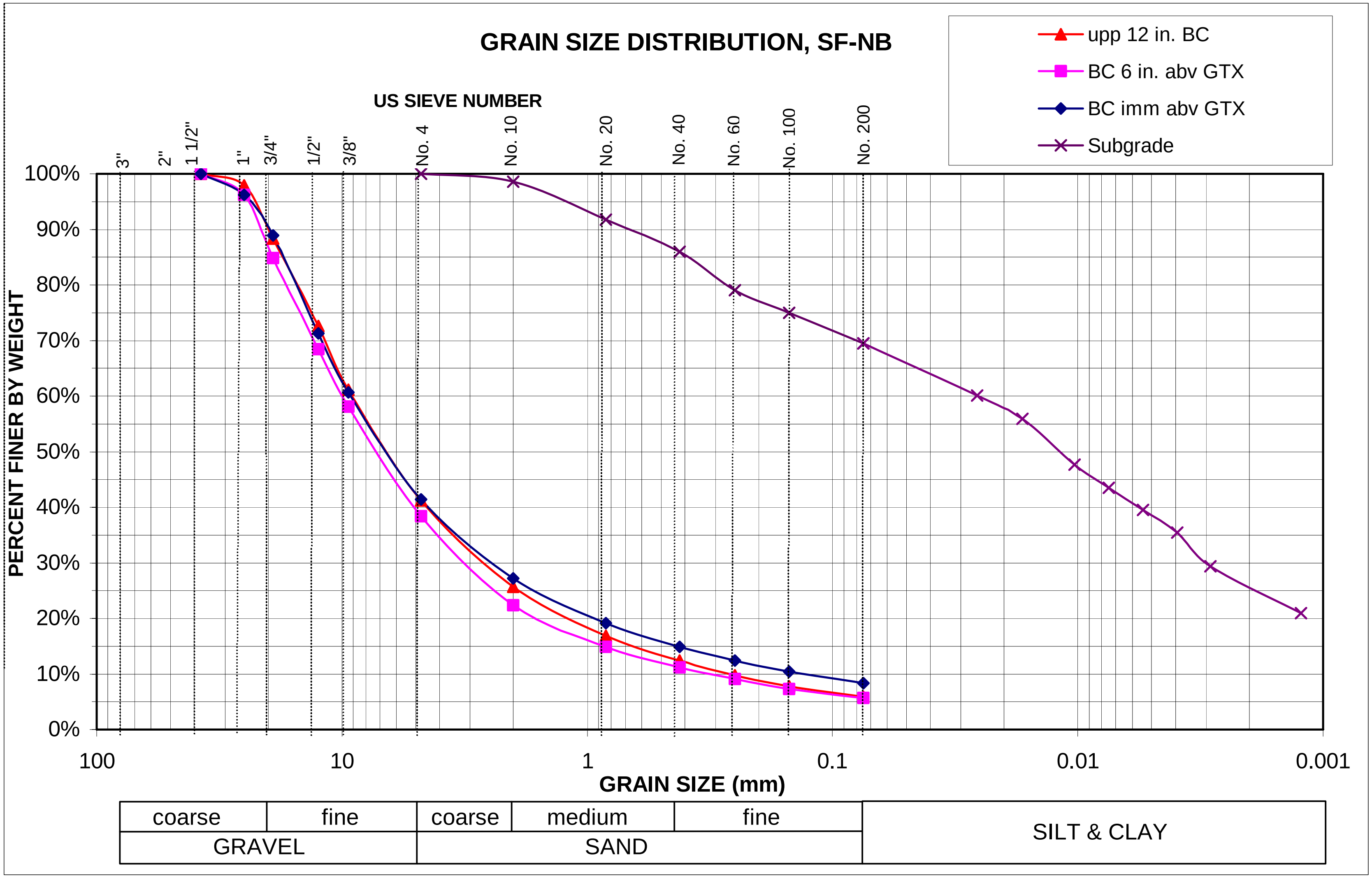

Figure B.5. Grain size distribution curves, SF-NB.

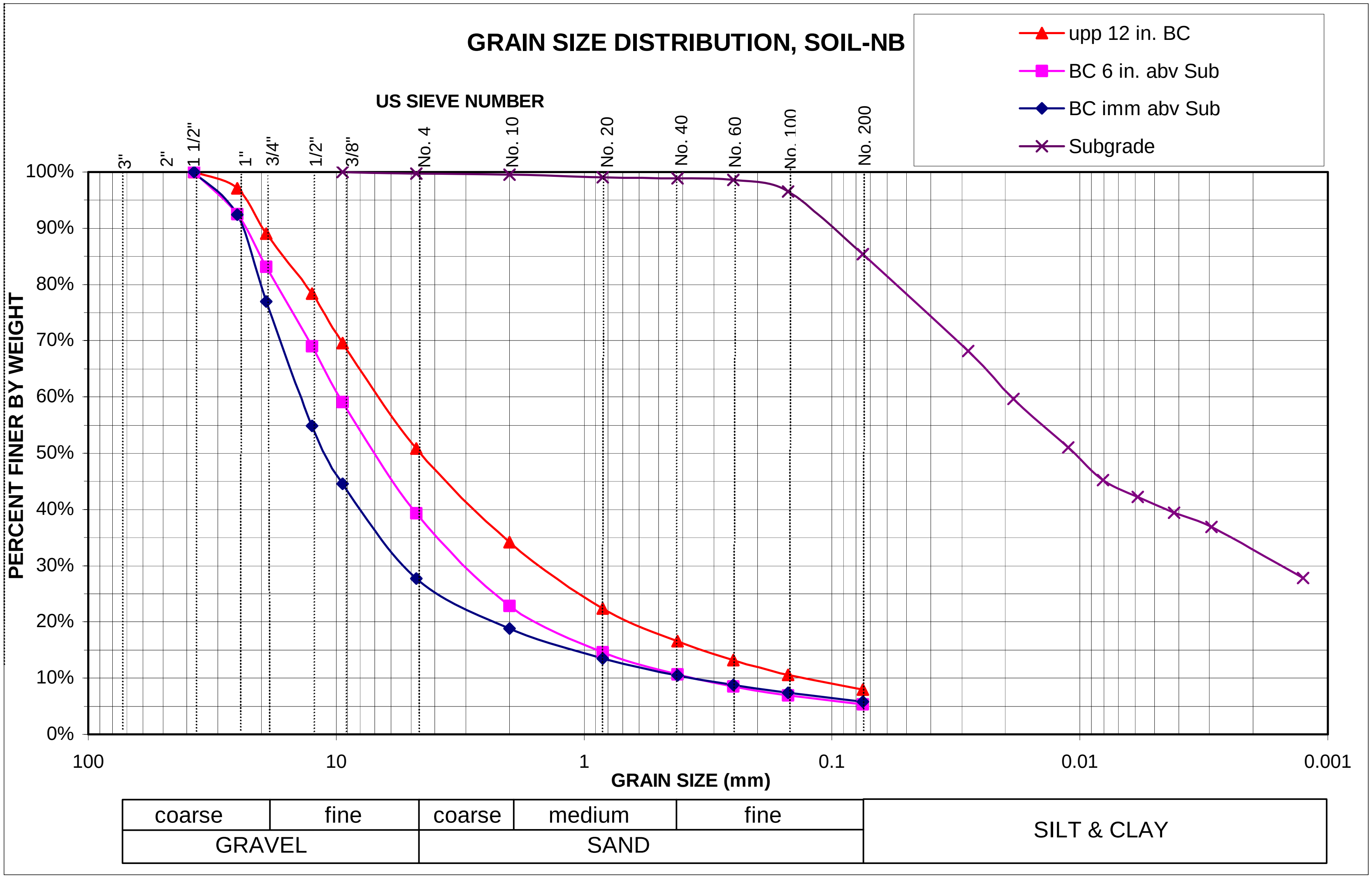

Figure B.6. Grain size distribution curves, Soil-NB.

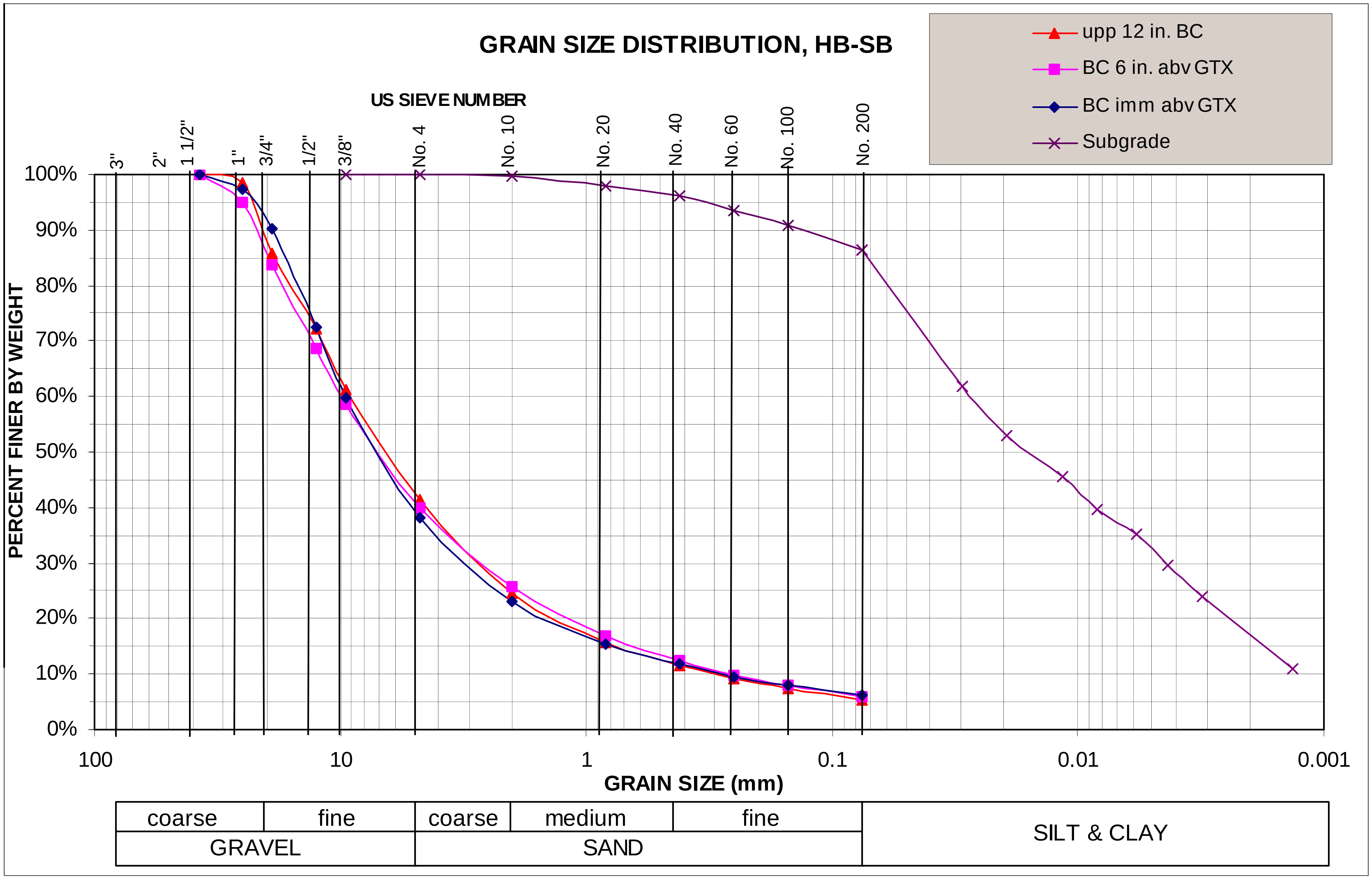

Figure B.7. Grain size distribution curves, HB-SB.

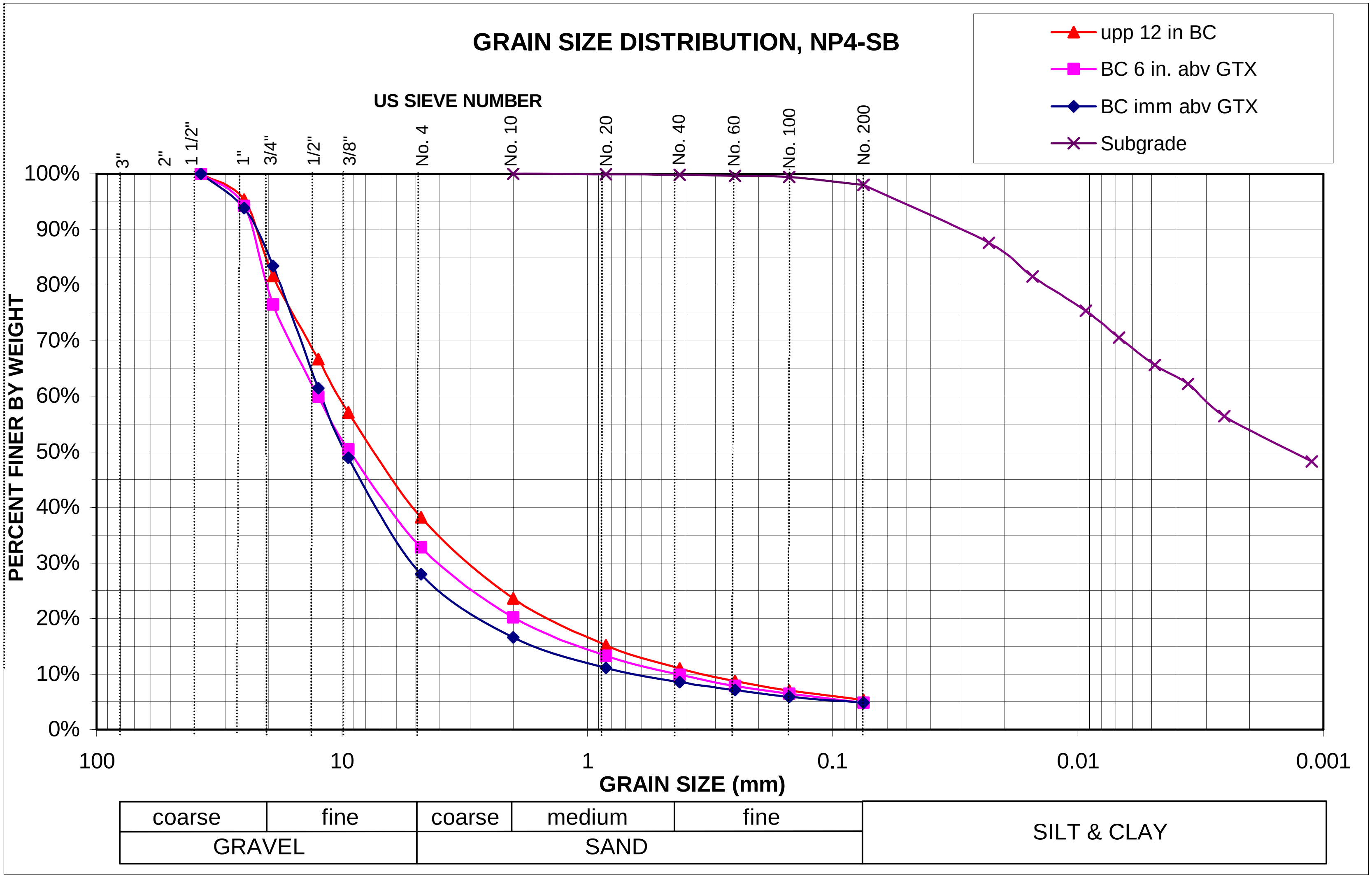

Figure B.8. Grain size distribution curves, NP4-SB.

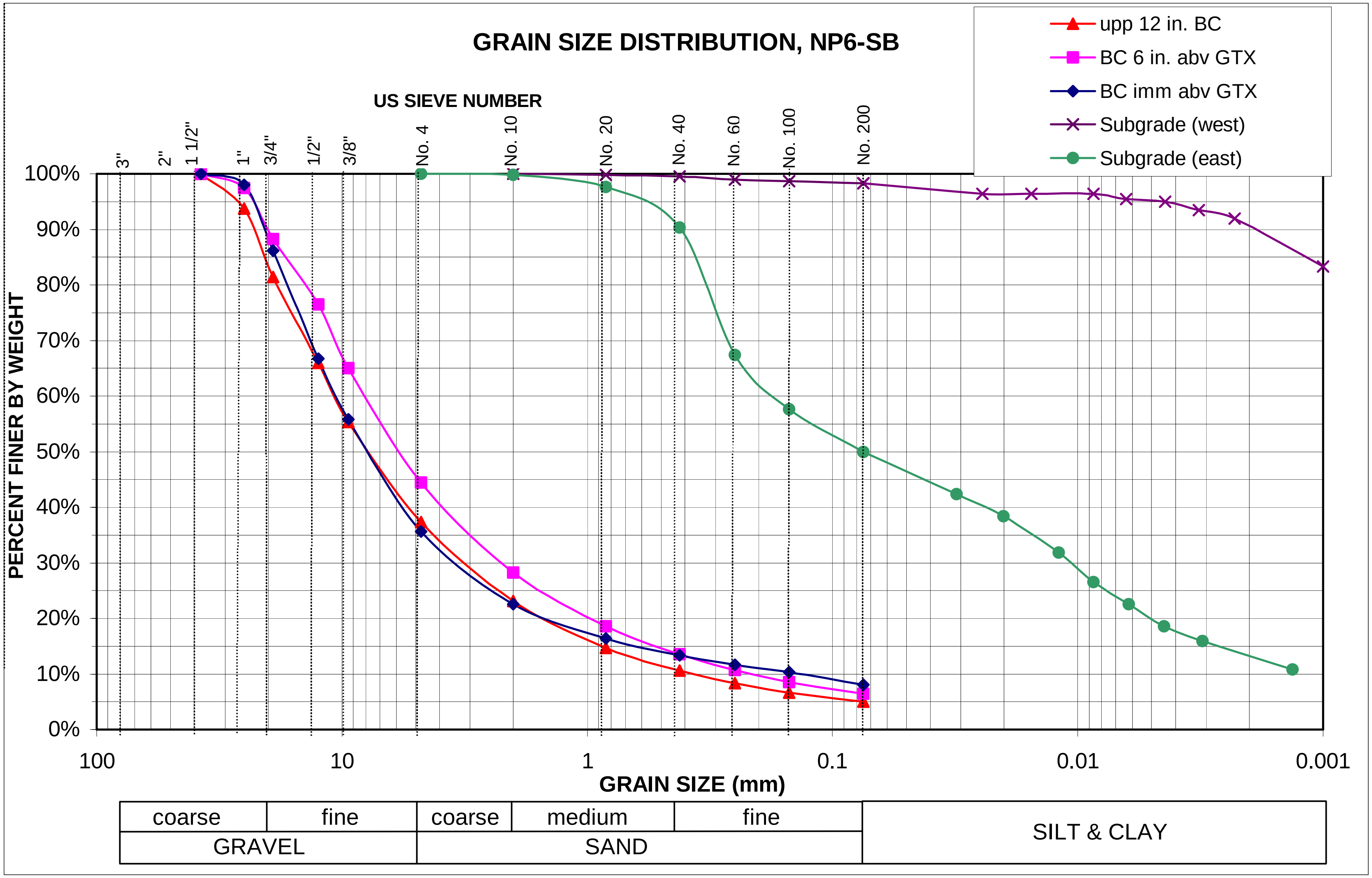

Figure B.9. Grain size distribution curves, NP6-SB.

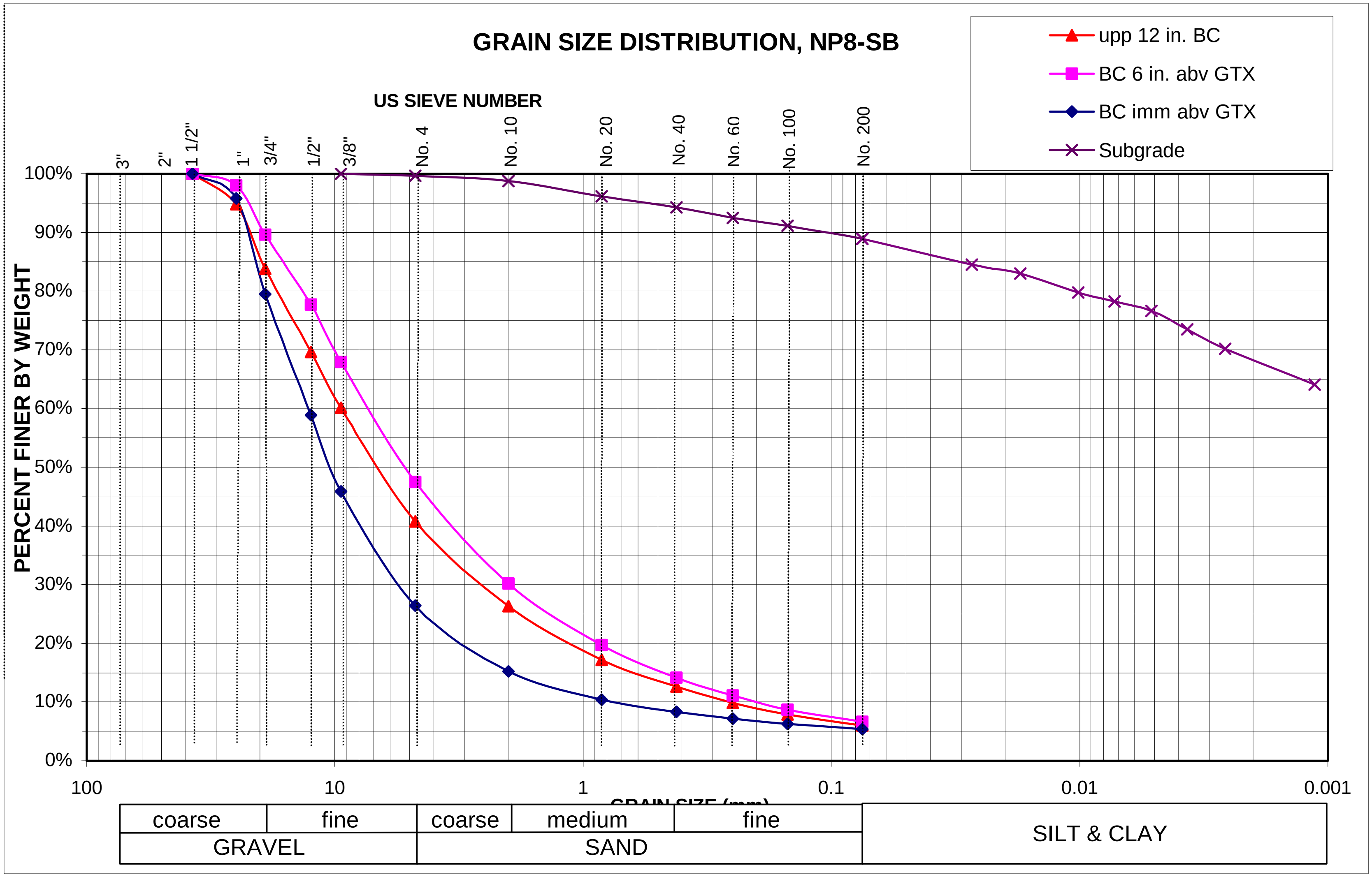

Figure B.10. Grain size distribution curves, NP8-SB.

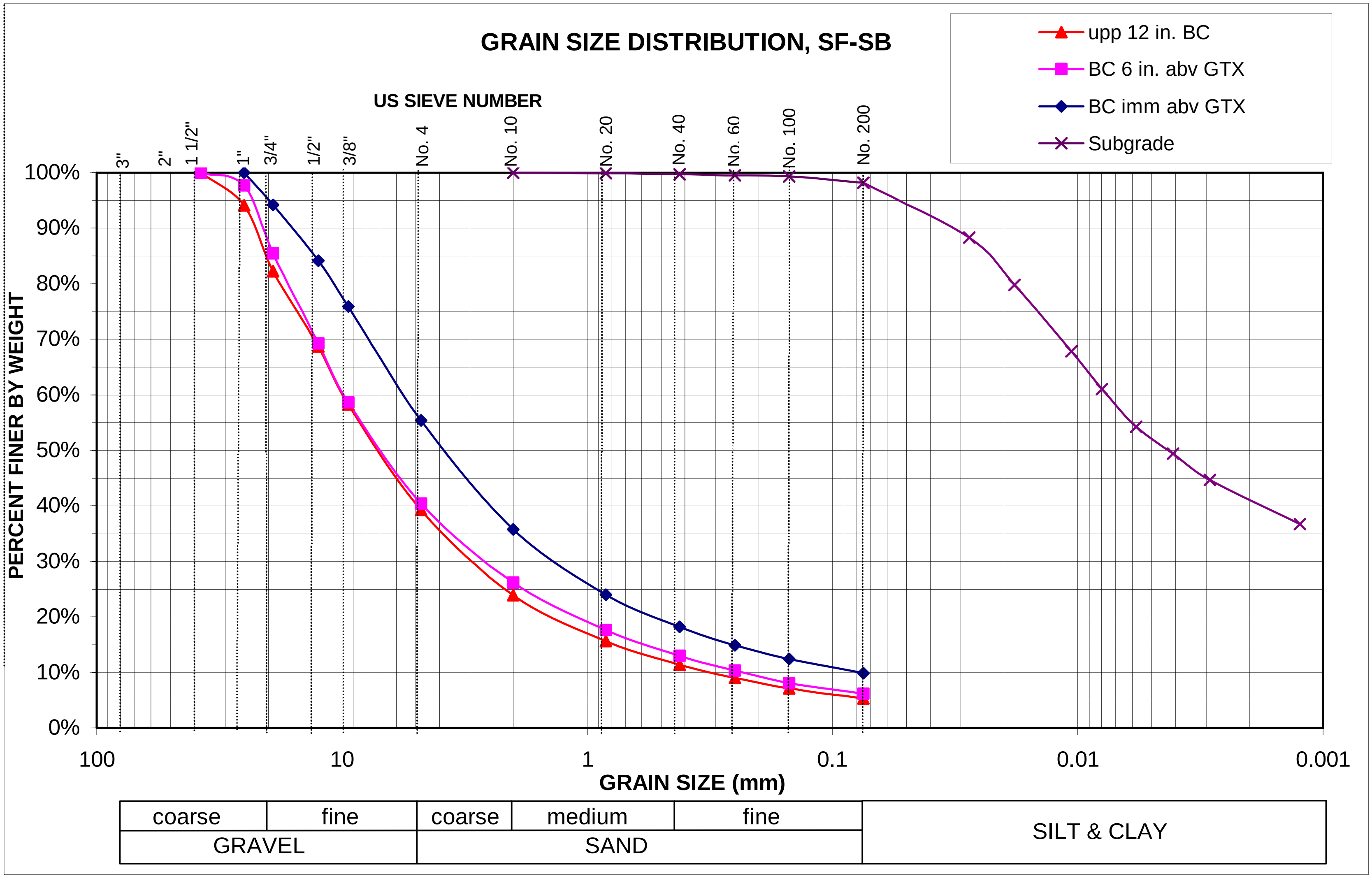

Figure B.11. Grain size distribution curves, SF-SB.

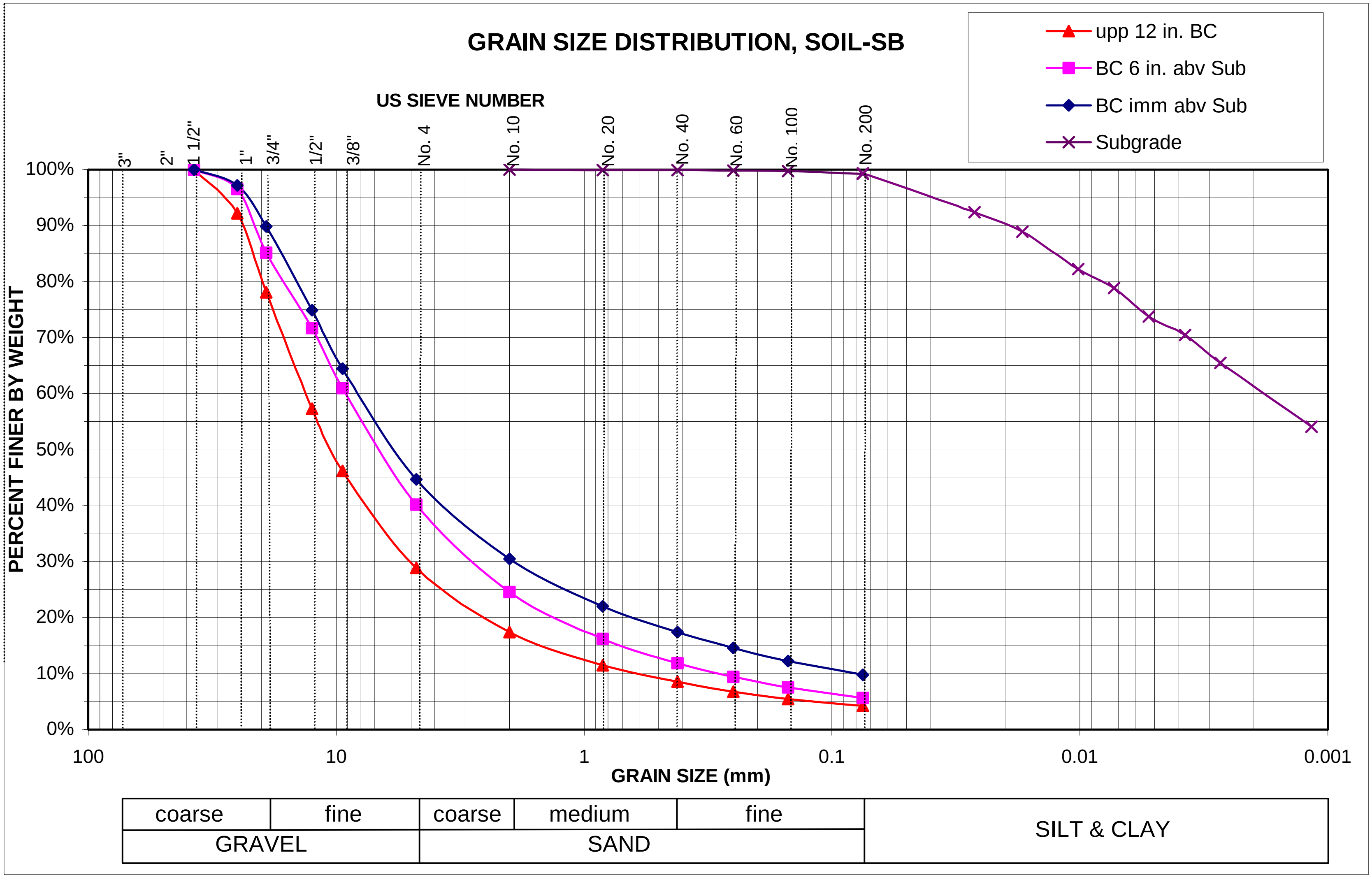

Figure B.12. Grain size distribution curves, Soil-SB.

APPENDIX C

Permittivity Test Procedure and Results

PERMITTIVITY TEST PROCEDURE

The permittivity test procedure was in general conformance with ASTM D 4491, "Standard Test Method for Water permeability of Geotextiles by Permittivity." A "STS geotextile permeameter" described in Christopher (1983), Holtz and Page (1991), and Metcalfe and Holtz (1994) was used to conduct the tests.

The permittivity test procedure consisted of the following:

1. Select 4 representative specimens from each sample. Cut each specimen into a circle about 2.15 in. (55 mm) in diameter.

2. Inundate each specimen in deaired water for a period of 24 hours or more. Use deaired water from a deairing tank that fills by spraying a fine mist of water under a vacuum of 30 in. (75 cm) of Mercury (Hg).

3. Place a prepared specimen on the lower portion of the union joint. The *bottom surface of the specimen should be up* simulating the actual field flow conditions.

4. Carefully lift the lower assembly and attach it to the standpipe.

5. Once the lower assembly is securely attached, very slowly fill the permeameter through the overflow port until water reaches the top of the overflow.

6. Record the water temperature in the overflow pipe.

7. Place a plug over the overflow pipe to keep the specimen from being disturbed.

8. Fill the standpipe by keeping the end of the water supply tube immersed in water to minimize intrusion of air bubbles. Fill the standpipe between the 50 and 60 cm level.

9. Insert the stopper and air supply tube with a crimp placed over the air supply inlet. Adjust the lower end of the air supply tube so it is 2 in. (50 mm) above the elevation of the overflow, thus providing 2 in. (50 mm) of head. Remove the cap on the overflow after the stopper and air supply tube are securely in place.

10. Slowly release the crimp placed over the air supply just enough to allow some air to enter the standpipe so the bottom of the air supply tube will be at atmospheric pressure.

11. Record the water level in the standpipe (H_0).

12. Start the test by simultaneously removing the crimp from the air supply tube and starting the stopwatch. Before stopping the test, allow the water level in the standpipe to drop at least 12 in. (300 mm) or drop for at least 60 seconds. Stop the test by crimping the air supply tube and simultaneously stopping the stopwatch.

13. Record the final water level (H_f).

14. Repeat steps 6 through 13 until 5 runs are complete.

15. Disassemble the apparatus and remove the geotextile specimen.

16. Wash the geotextile by gently massaging it under swiftly moving water until nearly all of the soil particles have been removed. Use care to that the structure of the geotextile is not altered.

17. Follow steps 3 though 5 to place the washed specimen in the permeameter.

18. Repeat steps 7 through 13 until 5 runs have been completed on the washed specimen.

PERMITTIVITY TEST RESULTS

The permittivity test results are presented on the following pages. Testing procedures and results are summarized in Chapter 4, and a detailed description of the permittivity test procedure is contained in Appendix A. Analysis of the permittivity test results and comparisons to the results of the Phase II investigation are made in Chapter 5. The following formulas were used to calculate the permittivity of the specimens:

$$q = \frac{(R_t)(\Delta S)(A_{flow})}{t}$$

where:

q = volumetric flow rate, cm^3/s,

R_t = temperature correction factor, $R_t = u_t/u_{20C}$,

u_t = water viscosity at test temperature, mP,

u_{20C} = water viscosity at 20º C, mP,

ΔS = water level drop, cm,

A_{flow} = standpipe cross-sectional area of flow, cm^2, and

t = time for flow, s.

$$\Psi = \frac{q}{(h)(A_{specimen})}$$

where:

ψ = permittivity, s^{-1},

h = head of water on specimen, cm, and

$A_{specimen}$ = cross-sectional area of specimen, cm^2.

$$k = (\Psi)(L)$$

where:

k = permeability of specimen, cm/s, and

L = thickness of specimen, cm.

Table C.1. Permittivity test results, HB-Control.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: **HB-CON** **Geotextile Thickness:** **0.04** **cm** 23-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	31.4	39.37	22.0	8.1	0.145	0.00579
		2	5.0	30.0	38.03	21.5	8.2	0.145	0.00579
		3	5.0	31.3	39.62	21.2	8.2	0.146	0.00584
		4	5.0	32.4	41.51	21.5	8.1	0.143	0.00573
		5	5.0	34.9	44.54	21.7	8.1	0.143	0.00573
						Averages -	8.1	0.144	0.00578
	washed	1	5.0	32.1	38.00	21.8	8.7	0.154	0.00616
		2	5.0	31.5	38.53	21.5	8.5	0.150	0.00600
		3	5.0	31.6	39.04	21.3	8.4	0.149	0.00597
		4	5.0	30.6	37.67	21.3	8.4	0.150	0.00599
		5	5.0	38.9	48.07	21.4	8.4	0.149	0.00596
						Averages -	8.5	0.150	0.00602
B	unwashed	1	5.0	31.0	46.48	21.8	6.8	0.122	0.00486
		2	5.0	31.3	46.62	21.3	7.0	0.124	0.00495
		3	5.0	30.3	45.53	21.1	6.9	0.123	0.00493
		4	5.0	31.4	47.45	21.3	6.9	0.122	0.00488
		5	5.0	31.9	47.58	21.3	7.0	0.124	0.00495
						Averages -	6.9	0.123	0.00492
	washed	1	5.0	30.8	47.02	21.6	6.8	0.120	0.00480
		2	5.0	30.7	46.19	21.3	6.9	0.123	0.00490
		3	5.0	31.7	49.13	21.4	6.7	0.119	0.00475
		4	5.0	30.4	46.59	21.3	6.8	0.120	0.00482
		5	5.0	32.1	49.22	21.3	6.8	0.120	0.00481
						Averages -	6.8	0.120	0.00482
C	unwashed	1	5.0	31.9	36.03	21.6	9.1	0.162	0.00649
		2	5.0	31.8	37.93	21.6	8.6	0.154	0.00614
		3	5.0	31.0	36.67	21.6	8.7	0.155	0.00619
		4	5.0	36.4	43.36	21.6	8.7	0.154	0.00615
		5	5.0	33.2	39.07	21.6	8.8	0.156	0.00623
						Averages -	8.8	0.156	0.00624
	washed	1	5.0	31.8	37.58	21.3	8.8	0.156	0.00624
		2	5.0	31.0	37.26	21.2	8.7	0.154	0.00615
		3	5.0	31.7	38.09	21.1	8.7	0.154	0.00617
		4	5.0	32.1	39.14	21.1	8.6	0.152	0.00608
		5	5.0	31.5	38.06	21.1	8.6	0.153	0.00614
						Averages -	8.7	0.154	0.00616
D	unwashed	1	5.0	35.6	42.49	21.3	8.7	0.155	0.00618
		2	5.0	31.5	37.21	21.2	8.8	0.157	0.00626
		3	5.0	34.6	41.84	21.1	8.6	0.153	0.00613
		4	5.0	31.4	38.26	21.1	8.6	0.152	0.00609
		5	5.0	48.2	60.29	21.0	8.4	0.149	0.00594
						Averages -	8.6	0.153	0.00612
	washed	1	5.0	32.2	37.82	21.4	8.8	0.157	0.00627
		2	5.0	32.0	38.13	21.2	8.7	0.155	0.00621
		3	5.0	30.8	37.77	21.1	8.5	0.151	0.00605
		4	5.0	31.3	38.24	21.0	8.6	0.152	0.00608
		5	5.0	31.5	38.89	21.0	8.5	0.150	0.00602
						Averages -	8.6	0.153	0.00613

Table C.2. Permittivity test results, NP4-Control.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: **NP4-CON** **Geotextile Thickness:** **0.15** cm 26-Oct-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	38.7	2.57	20.3	160	2.85	0.427
		2	5.0	37.7	2.59	20.0	156	2.77	0.416
		3	5.0	41.4	2.87	19.7	156	2.77	0.415
		4	5.0	38.9	2.67	19.8	157	2.79	0.418
		5	5.0	35.9	2.47	19.8	157	2.78	0.417
							157	2.79	0.418
	washed	1	5.0	38.2	2.55	20.0	161	2.85	0.428
		2	5.0	35.9	2.47	20.0	156	2.77	0.415
		3	5.0	39.4	2.66	19.8	159	2.83	0.425
		4	5.0	39.7	2.67	19.8	160	2.84	0.427
		5	5.0	48.5	3.33	19.9	156	2.78	0.417
							158	2.81	0.422
B	unwashed	1	5.0	40.8	2.80	19.0	160	2.84	0.426
		2	5.0	35.9	2.60	18.7	153	2.72	0.407
		3	5.0	36.1	2.58	18.7	155	2.75	0.413
		4	5.0	39.6	2.83	18.7	155	2.75	0.413
		5	5.0	41.7	3.02	18.7	153	2.72	0.407
							155	2.76	0.413
	washed	1	5.0	37.4	2.61	18.7	159	2.82	0.423
		2	5.0	37.6	2.67	18.7	156	2.77	0.415
		3	5.0	36.7	2.59	18.7	157	2.79	0.418
		4	5.0	36.5	2.59	18.7	156	2.77	0.416
		5	5.0	45.5	3.26	18.7	155	2.74	0.412
							156	2.78	0.417
C	unwashed	1	5.0	28.8	2.23	18.7	143	2.54	0.381
		2	5.0	40.3	3.09	18.3	146	2.59	0.389
		3	5.0	38.0	2.90	18.5	146	2.59	0.388
		4	5.0	33.9	2.56	18.7	147	2.60	0.391
		5	5.0	41.8	3.25	18.5	143	2.54	0.381
							145	2.57	0.386
	washed	1	5.0	38.0	2.76	18.9	152	2.69	0.404
		2	5.0	35.7	2.72	18.5	146	2.59	0.389
		3	5.0	37.8	2.84	18.3	149	2.64	0.396
		4	5.0	37.0	2.81	18.5	147	2.60	0.390
		5	5.0	37.5	2.82	18.7	147	2.62	0.392
							148	2.63	0.394
D	unwashed	1	5.0	40.5	2.97	19.1	149	2.65	0.398
		2	5.0	42.0	3.07	18.4	153	2.71	0.407
		3	5.0	39.3	2.78	18.0	159	2.83	0.424
		4	5.0	39.0	2.73	18.4	159	2.83	0.425
		5	5.0	48.4	3.60	18.3	150	2.67	0.400
							154	2.74	0.411
	washed	1	5.0	40.6	3.07	18.7	146	2.60	0.390
		2	5.0	34.7	2.59	18.0	151	2.68	0.402
		3	5.0	36.8	2.77	18.4	148	2.63	0.395
		4	5.0	34.2	2.71	18.3	141	2.51	0.376
		5	5.0	45.0	3.42	18.6	146	2.59	0.389
							147	2.60	0.390

Table C.3. Permittivity test results, NP6-Control.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: NP6-CON Geotextile Thickness: 0.20 cm 26-Oct-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	41.6	2.96	21.0	147	2.61	0.522
		2	5.0	40.0	2.97	20.7	142	2.52	0.504
		3	5.0	38.6	2.90	20.6	141	2.50	0.499
		4	5.0	42.4	3.14	20.6	143	2.53	0.507
		5	5.0	42.8	3.29	20.5	138	2.45	0.489
						Averages	142	2.52	0.504
	washed	1	5.0	39.8	2.91	20.6	144	2.57	0.513
		2	5.0	37.4	2.84	20.3	140	2.49	0.498
		3	5.0	35.8	2.70	20.1	142	2.52	0.504
		4	5.0	35.7	2.69	20.1	142	2.52	0.504
		5	5.0	39.2	3.01	20.3	139	2.46	0.492
						Averages	141	2.51	0.502
B	unwashed	1	5.0	32.7	2.66	20.4	130	2.32	0.463
		2	5.0	33.0	2.69	20.3	131	2.32	0.464
		3	5.0	37.0	3.02	20.3	130	2.32	0.463
		4	5.0	34.6	2.84	20.2	130	2.31	0.462
		5	5.0	45.5	3.83	20.3	126	2.24	0.449
						Averages	130	2.30	0.460
	washed	1	5.0	38.0	2.82	20.3	143	2.55	0.509
		2	5.0	35.4	2.79	20.1	136	2.41	0.482
		3	5.0	34.3	2.64	20.2	139	2.46	0.492
		4	5.0	39.5	2.96	20.1	143	2.53	0.507
		5	5.0	45.3	3.63	20.3	133	2.36	0.472
						Averages	139	2.46	0.492
C	unwashed	1	5.0	33.2	3.18	20.4	111	1.97	0.394
		2	5.0	33.7	3.33	20.0	108	1.93	0.385
		3	5.0	32.3	3.07	20.0	113	2.00	0.401
		4	5.0	31.6	3.04	20.0	111	1.98	0.396
		5	5.0	34.4	3.29	20.0	112	1.99	0.398
						Averages	111	1.97	0.395
	washed	1	5.0	36.0	3.30	20.5	116	2.05	0.410
		2	5.0	35.3	3.43	20.0	110	1.96	0.392
		3	5.0	34.4	3.29	20.2	112	1.98	0.396
		4	5.0	34.2	3.37	20.2	108	1.92	0.384
		5	5.0	37.2	3.65	20.0	109	1.94	0.388
						Averages	111	1.97	0.394
D	unwashed	1	5.0	34.3	3.05	20.5	119	2.11	0.423
		2	5.0	33.5	2.97	20.0	121	2.15	0.429
		3	5.0	41.8	3.82	19.8	118	2.09	0.419
		4	5.0	38.0	3.43	19.8	119	2.12	0.424
		5	5.0	37.6	3.41	20.2	118	2.09	0.418
						Averages	119	2.11	0.422
	washed	1	5.0	37.3	3.20	20.0	125	2.22	0.444
		2	5.0	33.3	2.91	19.8	123	2.19	0.438
		3	5.0	32.9	2.93	19.8	121	2.15	0.430
		4	5.0	34.6	2.99	19.8	125	2.21	0.443
		5	5.0	33.6	2.91	19.8	124	2.21	0.442
						Averages	124	2.20	0.439

Table C.4. Permittivity test results, NP8-Control.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: NP8-CON Geotextile Thickness: 0.20 cm 6-Oct-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	38.1	3.75	21.4	105.3	1.87	0.486
		2	5.0	40.3	4.05	21.0	104.1	1.85	0.481
		3	5.0	31.0	3.11	21.0	104.3	1.85	0.482
		4	5.0	43.7	4.33	21.2	105.1	1.87	0.485
		5	5.0	43.0	4.29	21.2	104.4	1.85	0.482
						Averages	104.6	1.86	0.483
	washed	1	5.0	41.2	4.00	21.5	106.5	1.89	0.492
		2	5.0	40.3	4.02	21.0	104.9	1.86	0.484
		3	5.0	33.9	3.32	21.2	106.3	1.89	0.491
		4	5.0	37.7	3.79	21.2	103.6	1.84	0.478
		5	5.0	41.2	4.07	21.2	105.4	1.87	0.487
						Averages	105.3	1.87	0.486
B	unwashed	1	5.0	36.0	4.05	20.1	95.0	1.69	0.439
		2	5.0	32.6	3.64	20.1	95.7	1.70	0.442
		3	5.0	34.0	3.85	19.8	95.1	1.69	0.439
		4	5.0	41.1	4.56	19.9	96.8	1.72	0.447
		5	5.0	45.5	5.09	19.8	96.3	1.71	0.445
						Averages	95.8	1.70	0.442
	washed	1	5.0	35.6	4.02	19.5	96.1	1.71	0.444
		2	5.0	36.8	4.05	19.3	99.0	1.76	0.457
		3	5.0	40.5	4.48	19.5	98.1	1.74	0.453
		4	5.0	34.9	4.08	19.7	92.3	1.64	0.426
		5	5.0	40.3	4.39	19.7	99.1	1.76	0.458
						Averages	96.9	1.72	0.448
C	unwashed	1	5.0	41.2	6.52	19.8	68.0	1.21	0.314
		2	5.0	34.5	5.43	19.6	68.8	1.22	0.318
		3	5.0	35.7	5.59	19.6	69.1	1.23	0.319
		4	5.0	36.0	5.75	19.5	67.9	1.21	0.314
		5	5.0	42.1	6.72	19.5	68.0	1.21	0.314
						Averages	68.4	1.21	0.316
	washed	1	5.0	34.9	5.39	19.5	70.2	1.25	0.324
		2	5.0	36.2	5.57	19.5	70.5	1.25	0.326
		3	5.0	35.0	5.38	19.7	70.2	1.25	0.324
		4	5.0	33.7	5.25	19.7	69.3	1.23	0.320
		5	5.0	53.5	8.37	19.7	69.0	1.23	0.319
						Averages	69.8	1.24	0.323
D	unwashed	1	5.0	39.1	6.60	19.8	63.8	1.13	0.295
		2	5.0	35.5	5.93	19.5	64.9	1.15	0.300
		3	5.0	34.1	5.79	19.5	63.9	1.13	0.295
		4	5.0	36.0	6.13	19.7	63.4	1.13	0.293
		5	5.0	40.7	6.83	20.0	63.9	1.13	0.295
						Averages	64.0	1.14	0.295
	washed	1	5.0	31.5	5.41	20.0	62.4	1.11	0.288
		2	5.0	35.3	6.05	19.6	63.1	1.12	0.292
		3	5.0	37.1	6.29	19.8	63.5	1.13	0.293
		4	5.0	43.6	7.39	19.6	63.8	1.13	0.295
		5	5.0	44.3	7.57	19.5	63.5	1.13	0.293
						Averages	63.3	1.12	0.292

Table C.5. Permittivity test results, SF-Control.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity									
TEST PIT ID:	SF-CON		Geotextile Thickness:		0.05	cm			22-Sep-03
Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	36.8	19.29	21.9	19.5	0.347	0.0173
		2	5.0	32.0	16.81	21.5	19.7	0.350	0.0175
		3	5.0	42.0	22.29	21.4	19.5	0.347	0.0173
		4	5.0	39.0	21.04	21.7	19.1	0.339	0.0169
		5	5.0	36.0	19.79	21.6	18.8	0.333	0.0167
						Averages	19.3	0.343	0.0172
	washed	1	5.0	32.0	16.76	21.7	19.6	0.349	0.0174
		2	5.0	32.0	16.80	21.2	19.8	0.352	0.0176
		3	5.0	33.0	17.65	21.5	19.3	0.343	0.0172
		4	5.0	32.7	17.68	21.3	19.2	0.341	0.0171
		5	5.0	33.6	18.01	21.6	19.2	0.342	0.0171
						Averages	19.5	0.345	0.0173
B	unwashed	1	5.0	31.1	35.10	21.7	9.1	0.162	0.0081
		2	5.0	31.5	35.80	21.6	9.1	0.161	0.0081
		3	5.0	32.6	37.26	21.5	9.0	0.161	0.0080
		4	5.0	32.0	36.91	21.4	9.0	0.160	0.0080
		5	5.0	32.0	37.04	21.7	8.9	0.158	0.0079
						Averages	9.0	0.160	0.0080
	washed	1	5.0	32.8	36.77	21.7	9.2	0.163	0.0082
		2	5.0	33.4	37.39	21.7	9.2	0.163	0.0082
		3	5.0	32.1	36.15	21.7	9.1	0.162	0.0081
		4	5.0	35.0	39.51	21.7	9.1	0.162	0.0081
		5	5.0	32.9	37.70	21.5	9.0	0.160	0.0080
						Averages	9.1	0.162	0.0081
C	unwashed	1	5.0	32.0	12.19	21.9	26.9	0.477	0.0239
		2	5.0	33.6	12.63	21.5	27.5	0.488	0.0244
		3	5.0	35.0	13.33	21.0	27.5	0.488	0.0244
		4	5.0	32.4	12.47	21.6	26.8	0.476	0.0238
		5	5.0	37.7	14.63	21.5	26.6	0.473	0.0237
						Averages	27.1	0.481	0.0240
	washed	1	5.0	33.4	13.25	21.7	25.9	0.461	0.0230
		2	5.0	33.0	13.14	21.3	26.1	0.463	0.0232
		3	5.0	31.5	12.60	21.3	26.0	0.461	0.0231
		4	5.0	35.9	14.24	21.3	26.2	0.465	0.0233
		5	5.0	36.3	14.62	21.3	25.8	0.458	0.0229
						Averages	26.0	0.462	0.0231
D	unwashed	1	5.0	33.4	13.09	21.8	26.2	0.465	0.0233
		2	5.0	36.9	14.43	21.2	26.6	0.473	0.0236
		3	5.0	32.5	12.89	21.4	26.1	0.464	0.0232
		4	5.0	32.7	12.86	21.5	26.3	0.467	0.0233
		5	5.0	35.5	14.07	21.4	26.1	0.464	0.0232
						Averages	26.3	0.467	0.0233
	washed	1	5.0	40.0	15.32	21.5	27.0	0.479	0.0240
		2	5.0	34.2	13.00	21.5	27.2	0.483	0.0242
		3	5.0	32.4	12.50	21.4	26.9	0.477	0.0239
		4	5.0	32.0	12.38	21.5	26.7	0.475	0.0237
		5	5.0	33.9	12.98	21.3	27.1	0.482	0.0241
						Averages	27.0	0.479	0.0240

Table C.6. Permittivity test results, HB-NB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: **HB-NB** **Geotextile Thickness:** **0.04** cm 23-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	15.6	60.38	21.7	2.7	0.047	0.00189
		2	5.0	23.0	61.12	21.3	3.9	0.069	0.00278
		3	5.0	26.7	60.55	21.2	4.6	0.082	0.00326
		4	5.0	30.4	60.47	21.3	5.2	0.093	0.00371
		5	5.0	33.2	62.26	21.2	5.6	0.099	0.00394
						Averages	4.4	0.078	0.00312
	washed	1	5.0	32.9	22.63	21.3	15.1	0.27	0.0107
		2	5.0	32.8	22.58	20.8	15.3	0.27	0.0108
		3	5.0	31.9	21.48	21.0	15.5	0.28	0.0110
		4	5.0	35.1	23.82	21.0	15.4	0.27	0.0110
		5	5.0	39.7	27.22	21.0	15.3	0.27	0.0108
						Averages	15.3	0.27	0.0109
B	unwashed	1	5.0	8.7	60.48	21.7	1.5	0.026	0.001051
		2	5.0	10.2	64.63	21.3	1.6	0.029	0.001165
		3	5.0	12.8	73.67	21.0	1.8	0.032	0.001291
		4	5.0	12.6	61.44	21.0	2.1	0.038	0.001524
		5	5.0	15.1	74.66	21.0	2.1	0.038	0.001503
						Averages	1.8	0.033	0.001307
	washed	1	5.0	26.3	60.21	21.3	4.5	0.081	0.00322
		2	5.0	28.6	62.54	21.0	4.8	0.085	0.00340
		3	5.0	28.2	60.31	21.0	4.9	0.087	0.00348
		4	5.0	28.7	60.34	21.0	5.0	0.088	0.00353
		5	5.0	31.8	63.72	21.3	5.2	0.092	0.00368
						Averages	4.9	0.087	0.00346
C	unwashed	1	5.0	33.0	19.32	21.8	17.5	0.31	0.0125
		2	5.0	32.1	16.48	21.1	20.3	0.36	0.0144
		3	5.0	32.0	15.69	21.0	21.3	0.38	0.0152
		4	5.0	33.6	15.29	21.4	22.8	0.40	0.0162
		5	5.0	34.0	15.13	21.5	23.2	0.41	0.0165
						Averages	21.0	0.37	0.0149
	washed	1	5.0	33.9	7.29	21.5	48.1	0.85	0.0342
		2	5.0	35.4	7.34	21.2	50.2	0.89	0.0357
		3	5.0	32.9	6.71	21.2	51.0	0.91	0.0363
		4	5.0	33.3	6.76	21.3	51.2	0.91	0.0364
		5	5.0	38.9	7.83	21.3	51.6	0.92	0.0367
						Averages	50.4	0.90	0.0358
D	unwashed	1	5.0	31.8	20.73	21.7	15.8	0.28	0.0112
		2	5.0	32.1	18.32	21.1	18.3	0.32	0.0130
		3	5.0	31.2	16.30	21.2	19.9	0.35	0.0142
		4	5.0	31.5	15.93	21.4	20.5	0.36	0.0146
		5	5.0	40.2	19.91	21.4	20.9	0.37	0.0149
						Averages	19.1	0.34	0.0136
	washed	1	5.0	33.4	6.67	21.7	51.5	0.92	0.0366
		2	5.0	33.2	6.58	21.1	52.7	0.94	0.0374
		3	5.0	33.1	6.57	21.2	52.5	0.93	0.0373
		4	5.0	39.0	7.70	21.0	53.0	0.94	0.0376
		5	5.0	33.3	6.67	21.0	52.2	0.93	0.0371
						Averages	52.4	0.93	0.0372

Table C.7. Permittivity test results, NP4-NB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: NP4-NB Geotextile Thickness: 0.15 cm 5-Nov-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	31.6	8.90	19.0	39.0	0.69	0.104
		2	5.0	32.1	8.17	17.9	44.4	0.79	0.118
		3	5.0	32.2	7.43	17.9	49.0	0.87	0.130
		4	5.0	33.0	6.98	18.0	53.2	0.95	0.142
		5	5.0	41.6	8.19	17.8	57.5	1.02	0.153
						Averages	48.6	0.86	0.130
	washed	1	5.0	38.2	3.36	17.5	130	2.30	0.346
		2	5.0	34.9	3.13	17.3	128	2.27	0.341
		3	5.0	36.4	3.27	17.0	129	2.28	0.343
		4	5.0	33.7	3.07	17.2	126	2.24	0.336
		5	5.0	37.0	3.42	17.3	124	2.20	0.331
						Averages	127	2.26	0.339
B	unwashed	1	5.0	31.3	21.49	17.9	16.5	0.29	0.044
		2	5.0	32.1	20.76	17.4	17.7	0.31	0.047
		3	5.0	31.2	19.17	17.5	18.6	0.33	0.049
		4	5.0	31.4	17.36	17.3	20.7	0.37	0.055
		5	5.0	32.1	16.99	17.3	21.7	0.38	0.058
						Averages	19.0	0.34	0.051
	washed	1	5.0	33.5	7.37	17.7	51.6	0.92	0.138
		2	5.0	33.5	7.59	17.1	50.9	0.90	0.135
		3	5.0	33.0	7.53	17.4	50.1	0.89	0.134
		4	5.0	32.4	7.41	17.0	50.5	0.90	0.135
		5	5.0	34.8	7.90	17.2	50.6	0.90	0.135
						Averages	50.7	0.90	0.135
C	unwashed	1	5.0	32.0	29.50	25.0	10.3	0.18	0.028
		2	5.0	34.3	20.07	24.1	16.6	0.30	0.044
		3	5.0	32.6	15.83	24.7	19.8	0.35	0.053
		4	5.0	32.0	12.91	24.8	23.7	0.42	0.063
		5	5.0	35.7	12.38	24.9	27.6	0.49	0.073
						Averages	19.6	0.35	0.052
	washed	1	5.0	36.1	2.87	25.0	120	2.13	0.319
		2	5.0	38.0	3.01	24.9	121	2.14	0.321
		3	5.0	34.0	2.63	24.8	124	2.20	0.330
		4	5.0	35.0	2.75	24.4	123	2.18	0.328
		5	5.0	36.3	2.97	24.2	119	2.11	0.316
						Averages	121	2.15	0.323
D	unwashed	1	5.0	31.2	7.33	24.0	41.5	0.74	0.111
		2	5.0	34.9	6.46	23.8	52.9	0.94	0.141
		3	5.0	31.4	5.08	24.4	59.7	1.06	0.159
		4	5.0	32.5	4.76	24.6	65.7	1.17	0.175
		5	5.0	34.4	4.70	24.7	70.2	1.25	0.187
						Averages	58.0	1.03	0.155
	washed	1	5.0	39.0	2.57	25.0	144.7	2.57	0.385
		2	5.0	40.7	2.71	25.0	143.2	2.54	0.381
		3	5.0	42.7	2.86	24.7	143.3	2.55	0.382
		4	5.0	36.5	2.47	24.7	141.8	2.52	0.378
		5	5.0	35.7	2.43	24.1	142.9	2.54	0.381
						Averages	143.2	2.54	0.381

Table C.8. Permittivity test results, NP6-NB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity										
TEST PIT ID:		**NP6-NB**		**Geotextile Thickness:**		**0.15**	**cm**		24-Oct-03	
Sample	**Condition**	**Test No.**	**Applied Head h (cm)**	**Water Level Drop ΔS (cm)**	**Time for Flow t (s)**	**Water Temp. T (C)**	**Volumetric Flow Rate q (cm^3/s)**	**Permittivity ψ s^{-1}**	**Permeability k cm/s**	
A	unwashed	1	5.0	32.9	19.89	20.5	17.5	0.31	0.062	
		2	5.0	32.4	15.87	20.1	21.8	0.39	0.078	
		3	5.0	34.1	14.11	20.0	25.9	0.46	0.092	
		4	5.0	34.2	11.87	20.0	30.9	0.55	0.110	
		5	5.0	34.6	11.19	20.0	33.1	0.59	0.118	
						Averages	25.8	0.46	0.092	
	washed	1	5.0	37.1	4.43	20.0	89.7	1.59	0.319	
		2	5.0	33.7	4.02	19.7	90.5	1.61	0.321	
		3	5.0	34.4	4.09	19.7	90.8	1.61	0.323	
		4	5.0	31.8	3.75	19.7	91.5	1.63	0.325	
		5	5.0	36.2	4.40	19.8	88.6	1.57	0.315	
						Averages	90.2	1.60	0.321	
B	unwashed	1	5.0	32.3	23.14	20.0	15.0	0.27	0.053	
		2	5.0	34.2	19.27	19.7	19.2	0.34	0.068	
		3	5.0	32.0	15.07	19.7	22.9	0.41	0.081	
		4	5.0	32.4	13.60	19.7	25.7	0.46	0.091	
		5	5.0	35.5	13.27	19.7	28.9	0.51	0.103	
						Averages	22.3	0.40	0.079	
	washed	1	5.0	38.5	4.02	19.8	103	1.83	0.366	
		2	5.0	41.6	4.26	19.7	105	1.87	0.374	
		3	5.0	35.9	3.61	19.7	107	1.91	0.381	
		4	5.0	37.0	3.82	19.8	104	1.85	0.370	
		5	5.0	36.4	3.70	19.7	106	1.89	0.377	
						Averages	105	1.87	0.374	
C	unwashed	1	5.0	36.9	12.59	20.3	31.2	0.55	0.111	
		2	5.0	34.4	8.61	19.7	43.1	0.77	0.153	
		3	5.0	33.7	7.00	19.7	52.0	0.92	0.185	
		4	5.0	34.1	6.29	19.8	58.4	1.04	0.207	
		5	5.0	35.1	5.89	19.8	64.2	1.14	0.228	
						Averages	49.8	0.88	0.177	
	washed	1	5.0	37.8	3.15	19.8	129	2.30	0.459	
		2	5.0	39.5	3.18	19.7	134	2.38	0.476	
		3	5.0	40.1	3.41	19.7	127	2.25	0.451	
		4	5.0	40.7	3.44	19.8	127	2.26	0.453	
		5	5.0	42.6	3.56	19.8	129	2.29	0.458	
						Averages	129	2.30	0.459	
D	unwashed	1	5.0	31.8	24.64	20.0	13.8	0.25	0.049	
		2	5.0	32.3	17.41	19.8	20.0	0.35	0.071	
		3	5.0	32.3	13.18	19.8	26.4	0.47	0.094	
		4	5.0	33.2	10.85	19.8	32.9	0.59	0.117	
		5	5.0	34.5	9.60	19.8	38.7	0.69	0.137	
						Averages	26.4	0.47	0.094	
	washed	1	5.0	36.9	3.85	20.0	103	1.82	0.365	
		2	5.0	33.0	3.52	19.7	101	1.80	0.359	
		3	5.0	31.8	3.36	19.7	102	1.81	0.363	
		4	5.0	33.6	3.59	19.7	101	1.79	0.359	
		5	5.0	33.5	3.57	19.8	101	1.79	0.359	
						Averages	102	1.81	0.361	

Table C.9. Permittivity test results, NP8-NB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: NP8-NB   Geotextile Thickness: 0.26 cm   15-Oct-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	33.5	7.38	20.2	48.4	0.86	0.224
		2	5.0	33.5	6.13	19.7	59.0	1.05	0.272
		3	5.0	40.6	6.53	19.8	66.9	1.19	0.309
		4	5.0	35.9	5.69	19.7	68.1	1.21	0.315
		5	5.0	35.8	5.41	19.6	71.6	1.27	0.331
						Averages	62.8	1.12	0.290
	washed	1	5.0	33.5	3.91	19.4	93.2	1.65	0.430
		2	5.0	36.2	4.19	19.5	93.7	1.66	0.433
		3	5.0	39.4	4.59	19.4	93.3	1.66	0.431
		4	5.0	36.5	4.27	19.4	93.0	1.65	0.429
		5	5.0	44.8	5.29	19.4	92.1	1.64	0.425
						Averages	93.1	1.65	0.430
B	unwashed	1	5.0	33.8	25.87	19.7	14.1	0.25	0.065
		2	5.0	33.5	23.10	19.5	15.7	0.28	0.073
		3	5.0	36.2	22.84	19.5	17.2	0.31	0.079
		4	5.0	32.7	19.37	19.8	18.2	0.32	0.084
		5	5.0	42.0	23.19	19.5	19.6	0.35	0.091
						Averages	17.0	0.30	0.078
	washed	1	5.0	39.1	6.22	19.8	67.7	1.20	0.313
		2	5.0	33.6	5.20	19.3	70.4	1.25	0.325
		3	5.0	36.8	5.71	19.3	70.3	1.25	0.324
		4	5.0	32.4	4.99	19.6	70.3	1.25	0.324
		5	5.0	38.5	5.93	19.5	70.4	1.25	0.325
						Averages	69.8	1.24	0.322
C	unwashed	1	5.0	33.4	10.07	20.0	35.5	0.63	0.164
		2	5.0	35.5	8.59	19.5	44.8	0.80	0.207
		3	5.0	35.2	7.41	19.6	51.4	0.91	0.237
		4	5.0	34.9	6.71	19.6	56.3	1.00	0.260
		5	5.0	42.2	7.89	19.5	58.0	1.03	0.268
						Averages	49.2	0.87	0.227
	washed	1	5.0	35.6	4.48	19.6	86.0	1.53	0.397
		2	5.0	37.4	4.77	19.5	85.1	1.51	0.393
		3	5.0	40.2	5.07	19.5	86.0	1.53	0.397
		4	5.0	40.3	5.10	19.5	85.7	1.52	0.396
		5	5.0	55.7	7.10	19.6	84.9	1.51	0.392
						Averages	85.5	1.52	0.395
D	unwashed	1	5.0	35.6	7.32	20.0	52.1	0.93	0.241
		2	5.0	34.7	5.73	19.5	65.7	1.17	0.303
		3	5.0	38.1	5.47	19.5	75.6	1.34	0.349
		4	5.0	44.8	5.92	19.5	82.1	1.46	0.379
		5	5.0	39.8	5.01	19.7	85.8	1.52	0.396
						Averages	72.2	1.28	0.334
	washed	1	5.0	45.4	4.14	20.0	117.5	2.09	0.543
		2	5.0	42.4	3.98	19.5	115.6	2.05	0.534
		3	5.0	48.4	4.59	19.7	113.8	2.02	0.526
		4	5.0	45.0	4.23	19.6	115.1	2.04	0.532
		5	5.0	39.3	3.68	19.7	115.3	2.05	0.532
						Averages	115.5	2.05	0.533

Table C.10. Permittivity test results, SF-NB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: **SF-NB** **Geotextile Thickness:** **0.05** **cm** 21-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	31.7	61.03	21.4	5.38	0.096	0.00478
		2	5.0	31.4	53.67	21.2	6.09	0.108	0.00541
		3	5.0	31.7	52.21	21.2	6.32	0.112	0.00561
		4	5.0	36.4	53.99	21.2	7.02	0.125	0.00623
		5	5.0	36.9	52.29	21.1	7.36	0.131	0.00654
						Averages	6.44	0.114	0.00572
	washed	1	5.0	31.7	39.69	21.0	8.35	0.148	0.00742
		2	5.0	31.1	38.19	20.8	8.56	0.152	0.00760
		3	5.0	32.4	39.22	20.9	8.66	0.154	0.00769
		4	5.0	31.1	38.27	21.0	8.50	0.151	0.00755
		5	5.0	33.7	40.47	20.8	8.75	0.155	0.00777
						Averages	8.57	0.152	0.00761
B	unwashed	1	5.0	19.9	60.50	21.0	3.44	0.0611	0.003056
		2	5.0	20.8	60.43	20.5	3.64	0.0647	0.003236
		3	5.0	21.9	60.40	20.5	3.84	0.0682	0.003409
		4	5.0	25.0	60.47	20.8	4.35	0.0772	0.003859
		5	5.0	25.6	61.87	20.8	4.35	0.0772	0.003862
						Averages	3.92	0.0697	0.003485
	washed	1	5.0	31.3	55.73	20.8	5.90	0.105	0.00524
		2	5.0	31.5	56.03	20.6	5.94	0.105	0.00527
		3	5.0	31.2	54.94	20.7	5.98	0.106	0.00531
		4	5.0	30.7	53.48	20.7	6.05	0.107	0.00537
		5	5.0	30.9	54.87	20.7	5.93	0.105	0.00527
						Averages	5.96	0.106	0.00529
C	unwashed	1	5.0	19.2	60.33	21.0	3.33	0.059	0.00296
		2	5.0	25.1	70.26	20.6	3.77	0.067	0.00335
		3	5.0	24.2	60.17	20.5	4.26	0.076	0.00378
		4	5.0	23.1	60.33	20.5	4.05	0.072	0.00360
		5	5.0	26.9	60.70	20.5	4.69	0.083	0.00417
						Averages	4.02	0.071	0.00357
	washed	1	5.0	35.7	62.25	20.7	6.04	0.107	0.00537
		2	5.0	31.0	54.03	20.6	6.06	0.108	0.00538
		3	5.0	32.8	57.72	20.6	6.00	0.107	0.00533
		4	5.0	31.5	54.69	20.4	6.11	0.109	0.00543
		5	5.0	38.9	68.13	20.3	6.07	0.108	0.00539
						Averages	6.06	0.108	0.00538
D	unwashed	1	5.0	31.4	46.77	21.0	7.02	0.125	0.006237
		2	5.0	31.1	44.56	20.6	7.37	0.131	0.006547
		3	5.0	31.1	43.13	20.6	7.62	0.135	0.006764
		4	5.0	30.6	42.53	20.5	7.62	0.135	0.006765
		5	5.0	37.4	52.67	20.7	7.48	0.133	0.006644
						Averages	7.42	0.132	0.006591
	washed	1	5.0	31.8	39.63	20.8	8.43	0.150	0.007490
		2	5.0	32.6	41.13	20.6	8.37	0.149	0.007435
		3	5.0	33.2	41.63	20.8	8.38	0.149	0.007444
		4	5.0	32.8	41.00	20.6	8.45	0.150	0.007504
		5	5.0	34.9	43.50	20.7	8.45	0.150	0.007507
						Averages	8.42	0.150	0.007476

Table C.11. Permittivity test results, HB-SB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: HB-SB **Geotextile Thickness:** 0.04 cm 18-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	40.5	16.89	20.8	25.2	0.45	0.0179
		2	5.0	36.2	13.08	20.8	29.1	0.52	0.0207
		3	5.0	37.5	12.59	20.8	31.3	0.56	0.0222
		4	5.0	36.2	11.37	20.8	33.5	0.59	0.0238
		5	5.0	42.5	13.18	20.8	33.9	0.60	0.0241
						Averages	30.6	0.54	0.0217
	washed	1	5.0	37.4	6.60	20.8	59.6	1.1	0.0423
		2	5.0	39.7	6.93	20.8	60.2	1.1	0.0428
		3	5.0	41.2	7.08	20.8	61.2	1.1	0.0435
		4	5.0	37.4	6.21	20.8	63.3	1.1	0.0450
		5	5.0	39.0	6.77	20.8	60.5	1.1	0.0430
						Averages	61.0	1.1	0.0433
B	unwashed	1	5.0	34.5	43.06	20.8	8.4	0.15	0.00598
		2	5.0	34.5	37.12	20.8	9.8	0.17	0.00694
		3	5.0	33.0	33.27	20.8	10.4	0.19	0.00741
		4	5.0	32.6	30.49	20.6	11.3	0.20	0.00802
		5	5.0	41.0	37.23	20.6	11.6	0.21	0.00826
						Averages	10.3	0.18	0.00732
	washed	1	5.0	33.4	13.94	20.6	25.3	0.45	0.0180
		2	5.0	35.4	14.44	20.6	25.9	0.46	0.0184
		3	5.0	32.6	12.89	20.8	26.6	0.47	0.0189
		4	5.0	33.9	14.11	20.6	25.4	0.45	0.0180
		5	5.0	40.8	17.39	20.6	24.8	0.44	0.0176
						Averages	25.6	0.45	0.0182
C	unwashed	1	5.0	33.8	12.30	20.9	28.8	0.51	0.0205
		2	5.0	33.3	10.98	20.8	31.9	0.57	0.0226
		3	5.0	33.6	10.53	20.7	33.6	0.60	0.0239
		4	5.0	35.7	10.52	20.7	35.8	0.64	0.0254
		5	5.0	42.9	11.89	20.7	38.0	0.68	0.0270
						Averages	33.6	0.60	0.0239
	washed	1	5.0	35.1	6.99	20.5	53.2	0.94	0.0378
		2	5.0	36.2	7.25	20.4	53.0	0.94	0.0377
		3	5.0	36.2	7.17	20.2	53.8	0.96	0.0383
		4	5.0	35.8	7.17	20.3	53.1	0.94	0.0377
		5	5.0	39.3	7.74	20.3	54.0	0.96	0.0384
						Averages	53.4	0.95	0.0380
D	unwashed	1	5.0	14.2	78.15	20.4	1.9	0.034	0.00137
		2	5.0	16.3	66.13	20.0	2.6	0.047	0.00188
		3	5.0	17.0	66.47	20.0	2.7	0.049	0.00195
		4	5.0	17.9	62.13	20.0	3.1	0.055	0.00219
		5	5.0	22.2	81.01	20.0	2.9	0.052	0.00209
						Averages	2.7	0.047	0.00189
	washed	1	5.0	32.1	45.29	20.4	7.5	0.13	0.00534
		2	5.0	32.1	44.84	20.3	7.6	0.14	0.00541
		3	5.0	33.7	45.73	20.5	7.8	0.14	0.00554
		4	5.0	33.4	45.72	20.3	7.8	0.14	0.00552
		5	5.0	35.2	45.97	20.4	8.1	0.14	0.00577
						Averages	7.8	0.14	0.00552

Table C.12. Permittivity test results, NP4-SB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: **NP4-SB** **Geotextile Thickness:** **0.15** cm 3-Nov-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	34.7	10.79	18.9	35.4	0.63	0.094
		2	5.0	32.6	8.99	18.3	40.5	0.72	0.108
		3	5.0	33.0	8.13	18.1	45.6	0.81	0.122
		4	5.0	31.9	7.47	17.8	48.4	0.86	0.129
		5	5.0	36.7	8.49	17.9	48.9	0.87	0.130
						Averages	43.8	0.78	0.117
	washed	1	5.0	34.2	5.48	19.0	68.5	1.22	0.183
		2	5.0	34.7	5.59	18.2	69.6	1.24	0.185
		3	5.0	37.1	5.89	17.9	71.2	1.26	0.190
		4	5.0	33.5	5.38	17.8	70.5	1.25	0.188
		5	5.0	39.4	6.17	17.8	72.3	1.28	0.193
						Averages	70.4	1.25	0.188
B	unwashed	1	5.0	34.0	9.53	18.2	40.0	0.71	0.107
		2	5.0	33.2	8.44	17.8	44.6	0.79	0.119
		3	5.0	35.6	8.12	17.5	50.0	0.89	0.133
		4	5.0	33.2	6.90	17.5	54.9	0.98	0.146
		5	5.0	37.3	7.17	17.6	59.2	1.05	0.158
						Averages	49.7	0.88	0.133
	washed	1	5.0	36.2	4.19	18.5	96	1.71	0.256
		2	5.0	36.0	4.14	18.0	98	1.74	0.261
		3	5.0	35.9	3.91	17.9	104	1.84	0.276
		4	5.0	35.8	3.77	17.5	108	1.92	0.289
		5	5.0	38.0	4.17	17.8	103	1.83	0.275
						Averages	102	1.81	0.271
C	unwashed	1	5.0	40.9	8.29	18.7	54.6	0.97	0.146
		2	5.0	34.8	5.14	17.9	76.5	1.36	0.204
		3	5.0	32.4	4.37	17.9	83.8	1.49	0.223
		4	5.0	34.7	4.37	17.6	90.4	1.61	0.241
		5	5.0	34.1	4.03	17.9	95.6	1.70	0.255
						Averages	80.2	1.42	0.214
	washed	1	5.0	32.2	3.35	18.0	108	1.92	0.288
		2	5.0	33.5	3.46	17.7	110	1.95	0.293
		3	5.0	29.4	3.07	17.6	109	1.94	0.290
		4	5.0	31.8	3.33	17.8	108	1.92	0.288
		5	5.0	34.5	3.49	17.5	113	2.00	0.301
						Averages	110	1.95	0.292
D	unwashed	1	5.0	33.9	5.82	18.8	64.3	1.14	0.171
		2	5.0	33.5	4.77	17.8	79.6	1.41	0.212
		3	5.0	32.7	4.28	17.8	86.6	1.54	0.231
		4	5.0	36.1	4.44	17.8	92.1	1.64	0.245
		5	5.0	33.9	4.03	17.5	96.0	1.71	0.256
						Averages	83.7	1.49	0.223
	washed	1	5.0	33.7	3.43	18.4	110	1.95	0.292
		2	5.0	39.4	3.94	17.9	113	2.01	0.301
		3	5.0	35.7	3.54	17.9	114	2.02	0.304
		4	5.0	39.5	3.95	17.8	113	2.01	0.302
		5	5.0	44.5	4.49	17.8	112	1.99	0.299
						Averages	112	2.00	0.300

Table C.13. Permittivity test results, NP6-SB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity										
TEST PIT ID:	NP6-SB		Geotextile Thickness:			0.20	cm		2-Nov-03	
Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s	
A	unwashed	1	5.0	32.7	5.69	18.9	63.3	1.12	0.225	
		2	5.0	31.9	5.11	18.2	70.0	1.24	0.249	
		3	5.0	34.1	5.18	18.7	72.9	1.29	0.259	
		4	5.0	32.5	4.70	18.3	77.3	1.37	0.275	
		5	5.0	33.6	4.73	18.0	80.0	1.42	0.284	
						Averages	72.7	1.29	0.258	
	washed	1	5.0	39.6	2.87	19.0	152	2.69	0.538	
		2	5.0	36.1	2.77	18.0	147	2.61	0.521	
		3	5.0	36.1	2.79	18.5	144	2.56	0.511	
		4	5.0	34.0	2.71	18.0	141	2.51	0.502	
		5	5.0	47.9	3.71	18.0	145	2.58	0.517	
						Averages	146	2.59	0.518	
B	unwashed	1	5.0	32.4	13.27	19.0	26.81	0.48	0.095	
		2	5.0	31.8	11.25	18.0	31.83	0.57	0.113	
		3	5.0	31.6	9.40	18.2	37.68	0.67	0.134	
		4	5.0	34.0	8.82	18.2	43.21	0.77	0.153	
		5	5.0	34.3	7.93	18.0	48.71	0.87	0.173	
						Averages	37.65	0.67	0.134	
	washed	1	5.0	33.8	2.86	19.0	130	2.30	0.461	
		2	5.0	33.3	2.76	18.9	133	2.36	0.472	
		3	5.0	35.8	3.01	18.8	131	2.33	0.467	
		4	5.0	32.4	2.71	18.5	133	2.36	0.473	
		5	5.0	35.3	2.96	19.5	129	2.30	0.460	
						Averages	131	2.33	0.466	
C	unwashed	1	5.0	32.8	12.15	19.2	29.5	0.52	0.105	
		2	5.0	33.2	10.40	17.9	36.0	0.64	0.128	
		3	5.0	30.8	8.25	18.0	42.0	0.75	0.149	
		4	5.0	32.9	7.84	18.0	47.3	0.84	0.168	
		5	5.0	33.8	7.37	18.0	51.7	0.92	0.183	
						Averages	41.3	0.73	0.147	
	washed	1	5.0	33.6	3.53	19.3	104	1.84	0.369	
		2	5.0	29.7	3.13	18.7	105	1.87	0.373	
		3	5.0	38.3	3.97	18.8	107	1.89	0.379	
		4	5.0	33.1	3.49	18.4	106	1.88	0.376	
		5	5.0	35.5	3.78	18.0	106	1.88	0.376	
						Averages	105	1.87	0.374	
D	unwashed	1	5.0	31.9	11.38	18.7	31.0	0.55	0.110	
		2	5.0	32.1	10.99	18.3	32.7	0.58	0.116	
		3	5.0	31.8	10.13	17.9	35.4	0.63	0.126	
		4	5.0	32.2	9.69	18.3	37.2	0.66	0.132	
		5	5.0	35.5	9.93	17.9	40.4	0.72	0.143	
						Averages	35.3	0.63	0.126	
	washed	1	5.0	34.9	2.89	19.0	133	2.36	0.471	
		2	5.0	36.5	3.00	18.7	135	2.39	0.479	
		3	5.0	32.1	2.69	18.6	132	2.35	0.471	
		4	5.0	36.2	3.07	18.5	131	2.33	0.466	
		5	5.0	39.8	3.37	18.6	131	2.33	0.466	
						Averages	132	2.35	0.470	

Table C.14. Permittivity test results, NP8-SB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: NP8-SB — Geotextile Thickness: 0.26 cm — 19-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	33.9	13.62	20.5	26.35	0.47	0.122
		2	5.0	35.0	11.27	20.4	32.96	0.59	0.152
		3	5.0	35.6	9.87	20.1	38.56	0.68	0.178
		4	5.0	35.2	9.13	20.4	40.92	0.73	0.189
		5	5.0	34.7	8.59	20.2	43.08	0.77	0.199
						Averages	36.37	0.65	0.168
	washed	1	5.0	37.5	5.28	20.3	75.56	1.3	0.349
		2	5.0	36.1	4.75	20.0	81.44	1.4	0.376
		3	5.0	34.2	4.59	20.0	79.84	1.4	0.369
		4	5.0	33.5	4.62	20.0	77.70	1.4	0.359
		5	5.0	44.4	5.99	20.0	79.43	1.4	0.367
						Averages	78.79	1.4	0.364
B	unwashed	1	5.0	35.8	9.88	20.1	38.74	0.69	0.179
		2	5.0	35.2	8.15	20.0	46.28	0.82	0.214
		3	5.0	33.0	6.88	20.2	51.15	0.91	0.236
		4	5.0	36.4	6.80	20.0	57.36	1.02	0.265
		5	5.0	36.8	6.73	20.0	58.59	1.04	0.271
						Averages	50.42	0.90	0.233
	washed	1	5.0	35.6	4.43	20.2	85.70	1.52	0.396
		2	5.0	36.2	4.33	20.0	89.59	1.59	0.414
		3	5.0	35.3	4.01	20.0	94.33	1.68	0.436
		4	5.0	34.6	4.13	20.0	89.77	1.59	0.415
		5	5.0	45.7	5.52	20.0	88.72	1.58	0.410
						Averages	89.62	1.59	0.414
C	unwashed	1	5.0	34.0	10.47	20.0	34.80	0.62	0.161
		2	5.0	32.1	8.29	20.0	41.49	0.74	0.192
		3	5.0	34.0	7.48	20.0	48.71	0.87	0.225
		4	5.0	35.0	7.16	20.0	52.38	0.93	0.242
		5	5.0	38.7	7.19	20.0	57.68	1.0	0.266
						Averages	47.01	0.84	0.217
	washed	1	5.0	31.5	3.68	20.3	91.07	1.618	0.421
		2	5.0	31.6	3.59	20.0	94.32	1.675	0.436
		3	5.0	34.3	3.93	20.0	93.52	1.661	0.432
		4	5.0	35.9	4.11	20.2	93.15	1.655	0.430
		5	5.0	41.7	4.63	20.2	96.05	1.706	0.444
						Averages	93.62	1.663	0.432
D	unwashed	1	5.0	32.9	8.57	20.3	40.84	0.725	0.189
		2	5.0	34.6	7.08	20.0	52.37	0.930	0.242
		3	5.0	34.5	5.66	20.0	65.32	1.160	0.302
		4	5.0	38.0	5.75	20.0	70.82	1.258	0.327
		5	5.0	37.1	5.41	20.2	73.13	1.299	0.338
						Averages	60.50	1.075	0.279
	washed	1	5.0	34.7	3.78	20.5	97.19	1.726	0.449
		2	5.0	35.1	3.81	20.0	98.72	1.753	0.456
		3	5.0	38.7	4.15	20.0	99.93	1.775	0.461
		4	5.0	36.7	3.90	20.2	100.36	1.783	0.463
		5	5.0	39.6	4.32	20.4	97.29	1.728	0.449
						Averages	98.70	1.753	0.456

Table C.15. Permittivity test results, SF-SB.

ASTM D 4491 / Water Permeability of Geotextiles by Permittivity

TEST PIT ID: SF-SB Geotextile Thickness: 0.05 cm 22-Sep-03

Sample	Condition	Test No.	Applied Head h (cm)	Water Level Drop ΔS (cm)	Time for Flow t (s)	Water Temp. T (C)	Volumetric Flow Rate q (cm^3/s)	Permittivity ψ s^{-1}	Permeability k cm/s
A	unwashed	1	5.0	29.7	60.62	21.3	5.09	0.0904	0.00452
		2	5.0	31.0	57.67	21.0	5.62	0.100	0.00499
		3	5.0	31.6	57.96	20.9	5.72	0.102	0.00508
		4	5.0	31.7	53.47	20.8	6.23	0.111	0.00553
		5	5.0	33.2	56.57	20.8	6.17	0.110	0.00548
						Averages	5.77	0.102	0.00512
	washed	1	5.0	32.7	42.72	20.7	8.07	0.143	0.00716
		2	5.0	32.5	41.35	20.7	8.28	0.147	0.00735
		3	5.0	31.5	40.70	20.6	8.17	0.145	0.00726
		4	5.0	32.6	41.38	20.4	8.36	0.149	0.00743
		5	5.0	32.0	40.33	20.8	8.34	0.148	0.00741
						Averages	8.24	0.146	0.00732
B	unwashed	1	5.0	26.0	60.43	21.8	4.42	0.078	0.003922
		2	5.0	29.4	60.67	20.8	5.09	0.090	0.004523
		3	5.0	30.9	60.37	21.0	5.35	0.095	0.004755
		4	5.0	28.1	60.33	21.0	4.87	0.087	0.004327
		5	5.0	35.4	66.96	21.2	5.50	0.098	0.004888
						Averages	5.05	0.090	0.004483
	washed	1	5.0	32.0	38.83	21.3	8.56	0.152	0.00760
		2	5.0	32.0	38.51	20.8	8.73	0.155	0.00776
		3	5.0	31.2	37.20	20.8	8.82	0.157	0.00783
		4	5.0	31.8	38.03	20.8	8.79	0.156	0.00781
		5	5.0	35.6	42.52	20.9	8.78	0.156	0.00780
						Averages	8.74	0.155	0.00776
C	unwashed	1	5.0	12.1	60.47	21.8	2.05	0.036	0.00182
		2	5.0	17.3	60.51	21.3	2.97	0.053	0.00264
		3	5.0	20.3	60.21	21.3	3.50	0.062	0.00311
		4	5.0	21.5	60.48	21.4	3.68	0.065	0.00327
		5	5.0	25.3	67.57	21.5	3.87	0.069	0.00344
						Averages	3.22	0.057	0.00286
	washed	1	5.0	31.4	59.08	21.3	5.52	0.098	0.00490
		2	5.0	31.3	59.80	21.4	5.42	0.096	0.00482
		3	5.0	40.9	78.54	21.4	5.40	0.096	0.00479
		4	5.0	31.5	59.07	21.3	5.54	0.098	0.00492
		5	5.0	31.9	60.20	21.7	5.45	0.097	0.00484
						Averages	5.47	0.097	0.00485
D	unwashed	1	5.0	40.3	64.27	21.7	6.45	0.115	0.005729
		2	5.0	31.9	46.49	21.3	7.13	0.127	0.006329
		3	5.0	32.0	45.82	21.3	7.25	0.129	0.006442
		4	5.0	31.8	42.89	21.5	7.66	0.136	0.006807
		5	5.0	32.0	43.18	21.7	7.62	0.135	0.006771
						Averages	7.22	0.128	0.006416
	washed	1	5.0	32.5	37.35	21.6	8.97	0.159	0.007969
		2	5.0	30.9	35.18	21.5	9.08	0.161	0.008064
		3	5.0	31.7	36.27	21.3	9.08	0.161	0.008062
		4	5.0	32.1	36.20	21.3	9.21	0.164	0.008180
		5	5.0	36.6	40.88	21.3	9.30	0.165	0.008259
						Averages	9.13	0.162	0.008107

APPENDIX D

Geotextile Tensile Strength Test Results

WIDE WIDTH TENSILE STRENGTH.

Wide width tensile strength tests were conducted on exhumed and virgin geotextile specimens in accordance with ASTM D 4595, “Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method.” All tests were conducted in the machine direction of the geotextile, which was parallel to centerline for the exhumed specimens. The specimens were 8 in. (200 mm) long and 8 in. (200 mm) wide. The distance between the clamps (gauge distance) was 4 in. (100 mm). ASTM does not specify procedures for preparing exhumed test specimens; therefore, the procedures recommended in Elias (2001) for preparing geosynthetics exhumed from construction projects were used. The specimens were hand washed gently under tap water prior to testing. The wide width specimens were soaked in water for a period of at least 24 hr prior to testing. The specimens were tested using a constant rate of extension of 10%/min. Ten specimens were prepared from each of the virgin (control) samples. Six test specimens were cut from each exhumed geotextile sample.

Tests were performed on virgin samples in order to determine an appropriate method of clamping the specimens. Two types of clamps were used, knurled and roughened. The knurled clamps have diamond shaped teeth and the roughened clamps have a 60-grit sandpaper type texture created by coarse sandblasting. Two methods of protecting the geotextile from the clamps were also evaluated in addition to using no protection. Tests were conducted with duct tape placed on the geotextile between the clamps; alternatively, thin particleboard strips were placed between the geotextile specimens and the clamps. The pressure applied to the clamps was also varied on the virgin specimens.

The NP4, NP6 and NP8 virgin specimens were taken from samples of the same manufacturer and model number. Whether the samples are from the same lot is unknown because the lot number for the samples placed at the test site is unknown. No identification was available on the HB and SF virgin samples to identify the manufacturer and model number. The samples were selected from University of Washington Geosynthetics Laboratory samples with the same element, polymer, manufactured structure, and color with similar physical properties to those installed below the test section. Based on the results of the tensile strength tests, the virgin HB samples were not the same type as the HB geotextile installed at the test section. The results of the Wide Width Tensile tests are presented in Tables D.1 to D.3.

The knurled clamps were used for testing all of the exhumed geotextile specimens. No additional protection was provided between the clamps for the nonwoven specimens. Duct tape was applied to the slit film woven specimens between the knurled clamps for additional protection. The nonwoven specimens were tested with a clamping pressure of 2000 psi (13.8 MPa) and the slit film woven specimens were tested at a clamping pressure of 3000 psi (20.7 MPa) due to their higher tensile strength. The number and size of holes between the clamps was measured for each of the exhumed specimens. This is generally the same procedure that was used by Black (1996) during the Phase II investigation.

Table D.1. Wide width tensile test results on virgin (control) specimens.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Clamping Method[1,2,3,4]	Clamping Pressure, psi[5]
HB	A	92	16.1	79	Roughened	3000
	B	110	19.2	86	Roughened	3000
	C	100	17.4	76	Roughened	3000
	D	96	16.8	69	Knurled	3000
	E	97	17.0	68	Knurled	3000
	F	102	17.8	58	Knurled	3000
	G	100	17.4	63	Knurled	3000
	H	106	18.5	76	Knurled	2000
	I	114	20.0	70	Knurled	2000
	J	109	19.1	84	Knurled	2000
	Average	102	17.9	72.9		
	Standard Deviation	7.0	1.2	8.9		
	Coefficient of Variation	0.07	0.07	0.12		
NP4	A	56	9.8	77	Roughened	2000
	B	50	8.8	85	Roughened	2000
	C	58	10.2	86	Roughened	2000
	D	54	9.5	79	Roughened	3000
	E	60	10.5	80	Roughened	3000
	F	66	11.5	85	Roughened	3000
	G	50	8.7	74	Knurled	3000
	H	57	10.0	68	Knurled	3000
	I	60	10.4	75	Knurled w/PBS	3000
	J	63	10.9	73	Knurled w/PBS	3000
	Average	57	10.0	78		
	Standard Deviation	5.1	0.9	5.9		
	Coefficient of Variation	0.09	0.09	0.08		

Table D.1. Continued.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Clamping Method[1,2,3,4]	Clamping Pressure, psi[5]
NP6	A	78	13.7	86	Roughened w/PBS	3000
	B	88	15.4	85	Roughened w/PBS	3000
	C	73	12.7	72	Roughened w/PBS	3000
	D	72	12.5	69	Roughened w/PBS	3000
	E	74	12.9	86	Roughened	3000
	F	79	13.8	83	Roughened	3000
	G	65	11.4	88	Roughened	3000
	H	74	12.9	75	Knurled w/DT	3000
	I	78	13.7	97	Knurled w/DT	3000
	J	80	13.9	98	Knurled w/DT	3000
	Average	76	13.3	84		
	Standard Deviation	6.1	1.1	9.7		
	Coefficient of Variation	0.08	0.08	0.12		
NP8	A	92	16.1	82	Knurled w/DT	3000
	B	95	16.6	91	Knurled w/DT	3000
	C	83	14.5	84	Knurled w/DT	3000
	D	92	16.0	84	Knurled w/DT	3000
	E	82	14.3	81	Knurled	3000
	F	73	12.7	83	Knurled	3000
	G	83	14.5	89	Knurled	3000
	H	101	17.6	91	Roughened w/PBS	3000
	I	77	13.5	92	Roughened w/PBS	3000
	J	85	14.8	100	Roughened w/PBS	3000
	Average	86	15.1	88		
	Standard Deviation	8.5	1.5	6.0		
	Coefficient of Variation	0.10	0.10	0.07		
SF	A	181	31.7	15	Knurled	3000
	B	189	33.0	17	Knurled	3000
	C	186	32.6	17	Knurled	3000
	D	192	33.6	17	Knurled	3000
	E	189	33.0	16	Knurled	3000
	F	198	34.7	18	Knurled w/DT	3000
	G	210	36.7	21	Knurled w/DT	3000
	H	203	35.5	19	Knurled w/DT	3000
	I	201	35.2	18	Knurled w/DT	3000
	J	211	36.8	19	Knurled w/DT	3000
	Average	196	34.3	18		
	Standard Deviation	10.2	1.8	1.7		
	Coefficient of Variation	0.05	0.05	0.10		

Notes:

1. The "Roughened" clamps were made by coarse sandblasting and have a texture similar to 60-grit sand paper.
2. The "Knurled" clamps have diamond shaped teeth.
3. PBS = Particle Board Strips placed between clamps and geotextile.
4. DT = Duct Tape placed on geotextile between clamps.
5. 2000 psi = 13.8 MPa, 3000 psi = 20.7 MPa.

Table D.2. Wide width tensile tests, northbound lane.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Geotextile Damage[1]
HB-NB	A	26	4.6	22	63 mil hole, 250 mil tear.
	B	20	3.6	22	86, 53 mil holes.
	C	27	4.7	23	61 mil tear.
	D	23	4.0	38	66, 116, 61, 77 mil holes.
	E	21	3.6	14	114, 96, 54 mil holes.
	F	21	3.6	35	33, 65, 39 mil holes.
	Average	23	4	25	
	Standard Deviation	2.9	0.5	9.0	
	Coefficient of Variation	0.13	0.13	0.36	
NP4-NB	A	34	5.9	48	61 mil hole.
	B	39	6.8	68	-
	C	31	5.4	59	69, 56 mil holes.
	D	32	5.6	62	87, 36 mil holes.
	E	35	6.1	64	89 mil hole.
	F	27	4.8	61	63, 60, 56, 36, 46, 54, 42 mil holes.
	Average	33	6	60	
	Standard Deviation	3.8	0.7	6.8	
	Coefficient of Variation	0.11	0.11	0.11	
NP6-NB	A	51	8.9	44	34 mil hole.
	B	53	9.3	46	-
	C	57	10.0	56	-
	D	48	8.4	59	67 mil hole.
	E	55	9.6	53	75, 96 mil holes.
	F	36	6.4	38	30 mil hole.
	Average	50	9	49	
	Standard Deviation	7.4	1.3	8.0	
	Coefficient of Variation	0.15	0.15	0.16	
NP8-NB	A	69	12.1	48	-
	B	69	12.1	48	-
	C	50	8.7	38	-
	D	44	7.6	39	-
	E	60	10.5	51	-
	F	53	9.3	52	-
	Average	57	10	46	
	Standard Deviation	10.5	1.8	6.2	
	Coefficient of Variation	0.18	0.18	0.14	
SF-NB	A	145	25.4	13	-
	B	149	26.1	13	-
	C	142	24.8	12	-
	D	137	23.9	12	-
	E	152	26.6	13	-
	F	150	26.2	13	-
	Average	146	26	13	
	Standard Deviation	5.8	1.0	0.5	
	Coefficient of Variation	0.04	0.04	0.04	

Notes: 1. 1 mil = 0.001 in = 0.0254 mm

Table D.3. Wide width tensile tests, southbound lane.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Geotextile Damage[1]
HB-SB	A	22	3.8	18	41 mil hole.
	B	26	4.5	25	74 mil hole.
	C	28	4.9	31	-
	D	27	4.8	23	58 mil hole.
	E	29	5.0	27	-
	F	30	5.3	32	100 mil tear.
	Average	27	4.7	26	
	Standard Deviation	2.9	0.5	5.0	
	Coefficient of Variation	0.11	0.11	0.19	
NP4-SB	A	41	7.2	55	-
	B	32	5.5	48	-
	C	34	6.0	44	-
	D	31	5.5	49	-
	E	22	3.8	35	-
	F	29	5.1	47	43, 87 mil holes.
	Average	31	5.5	46	
	Standard Deviation	6.3	1.1	6.6	
	Coefficient of Variation	0.20	0.20	0.14	
NP6-SB	A	61	10.6	59	-
	B	51	9.0	66	62 mil hole.
	C	57	10.0	62	193 mil tear.
	D	58	10.2	58	183 mil hole.
	E	62	10.9	62	-
	F	49	8.6	48	-
	Average	56	9.9	59	
	Standard Deviation	5.2	0.9	6.0	
	Coefficient of Variation	0.09	0.09	0.10	
NP8-SB	A	52	9.1	43	-
	B	82	14.3	64	-
	C	79	13.9	59	-
	D	57	10.0	47	-
	E	72	12.5	56	-
	F	67	11.7	53	-
	Average	68	11.9	54	
	Standard Deviation	11.8	2.1	7.7	
	Coefficient of Variation	0.17	0.17	0.14	
SF-SB	A	142	24.8	14	-
	B	137	24.0	13	-
	C	134	23.4	12	-
	D	138	24.2	13	-
	E	135	23.6	13	-
	F	138	24.2	13	-
	Average	137	24.0	13	
	Standard Deviation	2.9	0.5	0.5	
	Coefficient of Variation	0.02	0.02	0.04	

Notes: 1. 1 mil = 0.001 in = 0.0254 mm

FOUR-INCH TENSILE STRIP TESTS

Archived geotextile samples exhumed during the Phase II investigation were kept in the University of Washington Geosynthetic Laboratory. For comparison purposes, and to eliminate operator and procedural differences in testing methods, four-inch strip tensile tests were performed on samples exhumed from the northbound lane during the Phase II investigation in 1996 and during the Phase III investigation in 2003. Four-inch strip tests were also performed on some virgin samples.

The exact same procedures that were used for the wide width tensile tests were used for the four-inch tensile tests, the only exception being that duct tape was applied to the NP4 specimens as additional protection from the knurled clamps. As discussed in the preceding section, the virgin HB geotextile had different strength properties than the HB geotextile installed at the Bucoda test site. Summaries of the four-inch strip tensile tests are presented in Tables D.4 to D.6.

Table D.4. Four-inch strip tensile tests, virgin (control) specimens.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Clamping Method[1,2]	Clamping Pressure[3], psi
HB	A	62	10.9	41	Knurled	2000
	B	61	10.6	55	Knurled	2000
	C	66	11.5	58	Knurled	2000
	D	59	10.4	44	Knurled	2000
	Average	62	11	50		
	Standard Deviation	2.7	0.5	8.3		
	Coefficient of Variation	0.04	0.04	0.17		
NP4	A	42	7.3	72	Knurled	2000
	B	38	6.7	81	Knurled	2000
	C	46	8.0	87	Knurled w/DT	2000
	D	54	9.4	90	Knurled w/DT	2000
	E	46	8.1	74	Knurled	2000
	F	46	8.1	76	Knurled w/DT	2000
	Average	45	8	80		
	Standard Deviation	5.3	0.9	7.2		
	Coefficient of Variation	0.12	0.12	0.09		
NP6	A	57	9.9	88	Knurled	2000
	B	57	10.0	94	Knurled w/DT	2000
	C	59	10.3	87	Knurled w/DT	2000
	D	56	9.9	81	Knurled	2000
	E	54	9.4	88	Knurled	2000
	F	55	9.6	80	Knurled w/DT	2000
	Average	56	10	86		
	Standard Deviation	1.8	0.3	5.1		
	Coefficient of Variation	0.03	0.03	0.06		
NP8	A	82	14.3	93	Knurled	2000
	B	64	11.1	82	Knurled	2000
	C	70	12.2	100	Knurled w/DT	2000
	D	60	10.5	61	Knurled w/DT	2000
	E	64	11.2	114	Knurled	2000
	F	45	7.9	79	Knurled	2000
	G	76	13.4	114	Knurled w/DT	2000
	H	62	10.9	83	Knurled w/DT	2000
	Average	65	11	91		
	Standard Deviation	11.1	1.9	18.1		
	Coefficient of Variation	0.17	0.17	0.20		
SF	A	186	32.6	17	Knurled w/DT	3000
	B	213	37.3	20	Knurled w/DT	3000
	C	219	38.3	20	Knurled w/DT	3000
	D	207	36.2	20	Knurled w/DT	3000
	Average	206	36	19		
	Standard Deviation	14.3	2.5	1.6		
	Coefficient of Variation	0.07	0.07	0.08		

Notes:
1. The “Knurled” clamps have diamond shaped teeth.
2. DT = Duct Tape placed on geotextile between clamps.
3. 2000 psi = 13.8 MPa, 3000 psi = 20.7 MPa.

Table D.5. Four-inch strip tensile tests, Phase II (1996) specimens, northbound lane.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Geotextile Damage*
HB-NB	A	22	3.9	17	80 mil hole.
	B	18	3.1	18	-
	C	22	3.8	27	-
	Average	21	3.6	21	
	Standard Deviation	2.5	0.4	5.3	
	Coefficient of Variation	0.12	0.12	0.26	
NP4-NB	A	31	5.4	57	100, 140 mil holes.
	B	37	6.5	79	-
	C	33	5.8	60	40, 80 mil holes.
	Average	34	5.9	65	
	Standard Deviation	3.2	0.6	11.7	
	Coefficient of Variation	0.10	0.10	0.18	
NP6-NB	A	51	8.9	53	-
	B	58	10.1	55	-
	Average	54	9.5	54	
	Standard Deviation	5.0	0.9	1.7	
	Coefficient of Variation	0.09	0.09	0.03	
NP8-NB	A	59	10.4	53	-
	B	45	7.9	51	-
	C	44	7.7	43	-
	Average	50	8.7	49	
	Standard Deviation	8.6	1.5	5.4	
	Coefficient of Variation	0.17	0.17	0.11	
SF-NB	A	139	24.3	13	-
	B	134	23.4	13	-
	C	143	25.1	13	-
	Average	138	24.2	13	
	Standard Deviation	4.9	0.8	0.4	
	Coefficient of Variation	0.04	0.04	0.03	

Notes:

1. 1 mil = 0.001 in = 0.0254 mm

Table D.6. Four-inch strip tensile tests, Phase III (2003) specimens, northbound lane.

Geotextile	Specimen	Ultimate Wide Width Tensile Strength, lb/in	Ultimate Wide Width Tensile Strength, kN/m	Elongation at Ultimate Strength, %	Geotextile Damage*
HB-NB	A	22	3.9	32	900 mil tear, 370 mil hole.
	B	19	3.3	23	90 mil hole.
	C	20	3.6	24	160 mil hole.
	D	20	3.5	22	380, 130, 100 mil holes.
	E	25	4.3	23	70, 70, 80, 110 mil holes.
	F	18	3.1	17	630, 130, 70, 370 mil holes, 620 mil tear.
	Average	21	4	24	
	Standard Deviation	2.6	0.4	4.8	
	Coefficient of Variation	0.12	0.12	0.20	
NP4-NB	A	19	3.4	65	240, 220, 120, 100, 90, 90 mil holes.
	B	16	2.8	22	120 mil hole.
	Average	18	3	43	
	Standard Deviation	2.4	0.4	30.2	
	Coefficient of Variation	0.14	0.14	0.70	
NP6-NB	A	61	10.7	66	80 mil hole.
	B	50	8.8	58	-
	C	42	7.3	60	100 mil hole.
	Average	51	9	62	
	Standard Deviation	9.5	1.7	4.2	
	Coefficient of Variation	0.19	0.19	0.07	
NP8-NB	A	37	6.5	42	-
	B	47	8.2	53	-
	C	52	9.2	50	-
	Average	45	8	49	
	Standard Deviation	7.7	1.4	5.8	
	Coefficient of Variation	0.17	0.17	0.12	
SF-NB	A	146	25.5	15	-
	B	155	27.1	14	-
	C	132	23.1	13	-
	Average	144	25	14	
	Standard Deviation	11.4	2.0	1.4	
	Coefficient of Variation	0.08	0.08	0.10	

Notes:

1. 1 mil = 0.001 in = 0.0254 mm

APPENDIX E

FWD Deflection Data

FWD DEFLECTION DATA

This appendix presents the measured deflections for each drop of the FWD. Each applied load represents one drop of the FWD. Four drops were performed at each point to complete one set of data. The FWD Area Program and Evercalc backcalculation software was used to interpret the data. The data sets were used to determine the deflections for a normalized load of 9000 lb (40 kN). The data for the normalized load was adjusted for pavement thickness and temperature. A description of the software used to interpret the data analyses of the FWD results are presented in Chapter 6.

Table E.1. FWD deflection data, April 1991, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+50	13090	70.61	58.46	48.62	25.69	13.06	7.76
	9963	54.36	44.63	36.77	18.81	9.45	5.89
	8585	46.23	37.80	30.96	15.61	7.82	4.98
	4632	23.68	19.07	15.24	7.31	3.80	2.50
177+70	12883	86.41	66.86	57.69	27.41	18.77	13.34
	9733	66.16	51.83	43.04	20.62	9.63	5.94
	8454	56.98	44.30	36.55	17.26	7.94	4.54
	4608	29.67	22.69	18.04	8.04	3.98	2.54
177+95	11922	124.86	116.80	98.61	48.25	17.19	9.58
	9081	128.85	102.31	84.97	36.68	12.43	7.24
	7893	108.35	89.28	73.11	31.17	10.44	6.12
	4342	60.36	48.71	38.31	14.71	5.08	2.99
178+20	12736	97.39	78.93	63.61	27.10	8.75	6.74
	9625	77.06	61.78	49.13	19.91	6.79	4.87
	8326	66.36	53.00	41.80	16.43	5.43	4.04
	4568	35.24	27.52	21.03	7.31	2.62	2.09
178+45	12796	93.47	73.20	60.79	29.27	12.32	6.10
	9709	73.53	56.87	46.98	22.15	8.51	4.89
	8410	63.76	49.09	40.17	18.46	7.15	3.87
	4572	34.68	26.05	20.86	9.09	3.65	1.93
178+70	13197	65.47	53.02	44.10	22.55	11.20	6.59
	9987	50.43	40.57	33.43	16.67	8.23	4.78
	8620	42.90	34.33	28.16	13.84	6.85	4.02
	4664	22.20	17.44	13.91	6.51	3.34	2.03
178+95	13344	48.99	39.47	32.97	18.14	9.49	6.63
	10110	37.01	29.61	24.66	13.20	7.22	4.96
	8660	31.43	25.11	20.87	11.20	6.00	4.20
	4696	16.48	13.11	10.69	5.66	3.13	2.16
179+20	13236	63.46	48.17	38.30	17.00	7.79	5.35
	10031	47.98	36.06	28.30	12.14	5.61	3.83
	8652	40.48	30.28	23.59	9.93	4.63	3.16
	4688	20.04	14.72	11.14	4.52	2.22	1.54

Table E.2. FWD deflection data, April 1991, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
179+20	13332	71.00	56.20	46.81	22.95	11.13	7.17
	10396	54.90	43.44	35.92	17.02	8.00	5.18
	9061	46.81	36.98	30.47	14.22	6.67	4.37
	5117	24.05	18.79	15.20	6.71	3.34	2.18
178+95	13475	46.66	41.35	36.66	21.85	12.28	7.23
	10253	35.23	31.19	27.61	16.43	9.15	5.36
	8970	29.91	26.48	23.44	13.91	7.73	4.51
	5025	15.84	13.89	12.19	7.15	3.93	2.29
178+70	13622	36.38	37.90	36.86	21.42	9.96	6.35
	10662	27.49	28.66	27.75	15.81	7.32	4.57
	9240	23.32	24.34	23.48	13.26	6.26	3.96
	5160	12.32	12.82	12.22	6.66	3.07	2.02
178+45	13300	76.99	65.92	53.49	25.69	10.52	6.39
	10285	59.29	50.32	41.00	19.24	8.19	4.63
	8946	50.36	42.89	34.70	15.97	6.42	3.99
	5013	26.99	22.76	18.21	8.28	4.24	2.18
178+20	13483	54.45	44.22	37.13	19.38	9.22	5.45
	10436	40.62	32.84	27.45	14.10	6.57	3.98
	9069	34.15	27.59	23.00	11.70	5.41	3.31
	5013	17.34	13.86	11.38	5.60	2.64	1.69
177+95	12088	119.48	103.63	80.27	29.60	10.20	7.17
	9363	108.12	81.25	62.69	21.96	7.43	5.41
	8156	94.06	70.07	53.69	18.35	6.21	4.55
	4596	51.61	37.13	27.64	8.75	3.15	2.41
177+70	12668	101.72	79.65	64.77	29.32	12.28	7.38
	9824	89.95	62.04	50.16	22.04	9.11	5.46
	8509	110.60	52.93	42.66	18.47	7.68	4.61
	4767	71.33	27.25	21.55	8.84	3.81	2.40
177+50	12911	86.67	73.76	64.04	36.18	18.37	10.52
	9939	65.77	55.84	48.20	26.92	13.57	7.91
	8612	55.35	46.94	40.44	22.46	11.48	6.73
	4807	27.93	23.56	20.04	10.91	5.61	3.46

Table E.3. FWD deflection data, July 1991, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+50	13848	43.80	37.68	33.78	22.21	13.91	8.78
	10396	33.03	28.65	25.50	16.54	10.30	6.53
	8803	28.04	24.30	21.59	13.89	8.63	5.50
	4866	14.80	12.72	11.20	7.03	4.30	2.78
177+95	13590	63.57	55.17	47.81	33.35	20.18	11.76
	10233	48.89	42.74	36.76	25.41	15.39	8.85
	8744	41.82	36.72	31.85	21.67	13.07	7.48
	4803	22.55	19.76	17.35	11.35	6.80	4.06
178+20	13936	35.71	29.94	26.30	16.20	10.06	6.79
	10480	27.31	23.04	20.13	12.30	7.60	5.13
	8887	23.44	19.76	17.22	10.46	6.43	4.34
	4870	12.86	10.78	9.29	5.53	3.37	2.30
178+45	13916	37.87	31.78	27.52	17.08	10.40	6.93
	10428	29.08	24.46	21.18	12.96	7.87	5.22
	8863	24.98	21.00	18.17	11.04	6.67	4.44
	4835	13.74	11.48	9.89	5.84	3.47	2.33
178+70	13864	42.06	34.78	30.53	17.76	10.10	6.30
	10388	31.94	26.73	23.13	13.30	7.52	4.69
	8847	27.26	22.80	19.65	11.24	6.33	3.94
	4839	14.56	12.06	10.28	5.72	3.19	2.04
178+95	13673	55.16	45.11	40.59	23.28	13.03	7.95
	10372	42.34	34.96	30.89	17.58	9.76	5.93
	8795	36.28	29.99	26.32	14.91	8.24	5.00
	4807	19.54	16.13	13.85	7.65	4.18	2.56
179+20	12510	70.50	59.90	42.16	14.93	17.83	5.48
	9407	53.56	46.29	31.26	11.38	5.65	3.85
	8068	45.35	39.32	26.15	19.74	4.65	3.26
	4425	23.78	20.38	12.52	4.54	2.35	1.66

Table E.4. FWD deflection data, July 1991, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
179+20	13622	40.41	33.92	29.48	17.94	10.78	7.18
	10293	30.72	25.91	22.55	13.72	8.24	5.45
	8803	26.47	22.31	19.41	11.77	6.97	4.61
	4831	14.47	12.20	10.54	6.29	3.69	2.41
178+95	13673	35.64	30.42	26.54	16.87	10.76	7.42
	10225	27.34	23.34	20.41	13.04	8.28	5.66
	8751	23.68	20.27	17.65	11.25	7.13	4.87
	4791	13.35	11.32	9.78	6.17	3.83	2.63
178+70	13646	33.71	28.76	25.41	16.41	10.39	6.86
	10170	25.92	22.13	19.53	12.57	7.95	5.22
	8664	22.33	19.04	16.81	10.81	6.78	4.44
	4775	12.51	10.56	9.28	5.89	3.67	2.39
178+45	13626	33.50	28.07	24.36	15.06	9.48	6.43
	10166	25.81	21.59	18.71	11.50	7.19	4.87
	8672	22.19	18.55	16.06	9.82	6.13	4.13
	4731	12.33	10.20	8.76	5.26	3.22	2.15
178+20	13598	31.05	25.54	21.87	13.10	7.98	5.48
	10110	23.93	19.62	16.76	9.99	6.02	4.11
	8652	20.59	16.86	14.42	8.51	5.11	3.47
	4715	11.57	9.37	7.95	4.58	2.71	1.84
177+95	13507	38.55	31.76	27.37	16.75	10.55	7.20
	10094	29.68	24.47	21.09	12.87	8.07	5.49
	8640	25.66	21.13	18.17	11.02	6.90	4.68
	4680	14.46	11.72	10.00	5.94	3.64	2.48
177+70	13467	34.71	28.91	25.04	15.38	9.83	6.94
	10062	26.82	22.37	19.35	11.83	7.54	5.29
	8597	23.12	19.26	16.64	10.12	6.44	4.50
	4648	12.98	10.70	9.13	5.41	3.38	2.35
177+50	13276	35.48	30.03	26.47	17.21	11.47	7.99
	10186	27.37	23.04	20.30	13.07	8.60	5.86
	8632	23.64	19.93	17.54	11.35	7.52	5.28
	4672	13.00	10.83	9.49	6.06	3.99	2.79

Table E.5. FWD deflection data, November 1991, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+50	14162	23.91	21.81	20.20	15.12	10.76	7.47
	10599	18.33	16.67	15.43	11.54	8.22	5.71
	9034	15.69	14.23	13.19	9.81	7.00	4.87
	4898	8.63	7.85	7.19	5.34	3.76	2.61
177+70	14170	25.13	22.71	20.85	15.23	10.60	7.41
	10551	19.13	17.27	15.83	11.55	8.04	5.66
	9034	16.39	14.74	13.52	9.89	6.93	4.84
	4890	8.96	8.08	7.35	5.35	3.74	2.61
177+95	14162	23.45	21.23	19.67	14.72	10.48	7.47
	10623	17.94	16.08	14.94	11.24	7.98	5.73
	9045	15.42	13.91	12.83	9.68	6.88	4.94
	4934	8.47	7.57	7.01	5.31	3.75	2.69
178+20	14198	21.84	19.27	17.87	13.24	9.18	6.51
	10619	16.90	15.06	13.88	10.20	7.08	4.99
	9061	14.63	13.12	12.01	8.85	6.13	4.32
	4874	8.25	7.37	6.76	5.00	3.41	2.41
178+45	14150	21.72	19.58	18.02	13.31	9.47	6.68
	10591	16.84	15.16	13.93	10.30	7.29	5.15
	8998	14.53	13.09	12.02	8.87	6.28	4.42
	4858	8.17	7.41	6.75	4.96	3.49	2.46
178+70	14126	23.89	21.35	19.51	13.96	9.46	6.39
	10531	18.30	16.05	14.83	10.69	7.41	4.90
	9041	15.87	14.01	12.89	9.28	6.36	4.24
	4846	8.69	7.63	7.00	5.05	3.49	2.31
178+95	14071	30.01	26.85	24.75	17.52	11.77	7.69
	10531	23.07	20.65	19.00	13.47	9.04	5.91
	8962	19.85	17.80	16.37	11.61	7.81	5.09
	4843	10.88	9.63	8.87	6.27	4.21	2.67
179+20	41.51	36.95	30.42	15.64	8.15	5.23	0.00
	10396	31.19	27.61	22.56	11.42	6.01	3.89
	8922	26.42	23.32	18.97	9.51	5.02	3.29
	4910	13.68	11.93	9.52	4.63	2.55	1.72

Table E.6. FWD deflection data, November 1991, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
179+20	14468	54.21	39.42	33.02	19.82	11.57	7.16
	11064	42.22	30.61	25.61	15.41	8.99	5.57
	9657	36.30	26.30	21.98	13.25	7.74	4.79
	5788	20.73	14.93	12.43	7.55	4.43	2.74
178+95	14750	23.70	21.46	19.79	14.75	10.54	7.49
	11330	18.67	16.92	15.56	11.62	8.31	5.88
	9772	16.31	14.81	13.64	10.19	7.26	5.13
	5713	9.81	8.93	8.18	6.09	4.32	3.04
178+70	14746	23.18	21.04	19.50	14.44	10.34	7.26
	11254	18.30	16.67	15.41	11.44	8.07	5.67
	9753	16.01	14.61	13.50	10.02	7.07	4.95
	5625	9.51	8.70	8.00	5.93	4.14	2.89
178+45	14623	21.63	19.39	17.71	12.98	9.15	6.50
	11179	16.99	15.28	13.95	10.23	7.20	5.11
	9661	14.81	13.32	12.17	8.93	6.26	4.43
	5514	8.78	7.94	7.23	5.29	3.67	2.57
178+20	14102	21.67	19.07	17.39	12.36	8.48	5.96
	11016	17.11	15.15	13.74	9.81	6.72	4.71
	9578	14.81	13.13	11.90	8.52	5.82	4.05
	5506	8.93	7.92	7.37	4.99	3.37	2.33
177+95	14671	25.30	22.42	20.55	14.58	10.04	7.02
	11119	19.99	17.73	16.27	11.50	7.94	5.53
	9542	17.41	15.40	14.15	10.01	6.90	4.78
	5415	10.28	8.66	8.30	5.81	3.99	2.73
177+70	14257	23.87	21.40	19.36	13.92	9.71	6.85
	10988	18.96	16.99	15.37	11.01	7.66	5.39
	9447	16.53	14.83	13.42	9.59	6.65	4.67
	5391	9.82	8.81	7.92	5.61	3.81	2.63
177+50	13507	24.41	22.31	20.44	15.18	10.95	7.96
	10487	19.36	17.66	16.16	12.04	8.67	6.29
	9097	16.69	15.36	14.07	10.47	7.53	5.45
	5176	9.80	8.91	8.11	5.96	4.24	3.03

Table E.7. FWD deflection data, March 1996, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+50	15390	27.47	24.72	22.57	15.63	10.39	7.20
	11930	21.82	19.61	17.89	12.36	8.26	5.67
	9184	16.56	14.91	13.59	9.38	6.17	4.32
	5883	10.61	9.59	8.69	5.94	3.83	2.67
177+95	15545	24.91	21.91	20.17	13.46	8.86	6.17
	11993	19.44	17.05	15.78	10.41	6.83	4.75
	9228	14.73	13.03	11.74	7.88	5.11	3.53
	5879	9.28	8.28	7.41	4.91	3.13	2.14
178+20	15580	21.70	19.63	18.06	12.76	8.84	6.31
	11930	17.11	15.45	14.37	10.02	6.89	4.87
	9069	13.06	11.86	11.11	7.68	5.24	3.71
	5875	8.38	7.67	7.05	4.90	3.29	2.30
178+45	15497	23.65	20.98	19.16	13.09	8.71	6.07
	11854	19.44	16.41	14.98	10.31	6.89	4.81
	9045	14.04	12.41	11.35	7.71	5.14	3.52
	5788	8.99	7.98	7.27	4.88	3.23	2.19
178+70	15409	25.17	22.64	20.61	14.33	9.69	6.96
	11933	20.07	18.07	16.39	11.39	7.65	5.56
	9173	15.42	13.70	12.63	8.81	5.91	4.12
	5863	9.84	9.04	8.21	5.68	3.78	2.62
178+95	15342	34.51	30.35	26.87	16.34	9.59	6.20
	11755	27.07	23.82	20.99	12.77	7.42	4.82
	8990	20.35	17.85	15.79	9.58	5.57	3.56
	5768	12.84	11.31	10.00	6.04	3.44	2.13
179+20	15187	37.11	31.37	28.03	16.61	10.30	6.85
	11763	28.95	24.34	21.92	12.75	7.96	5.37
	8954	21.51	18.00	16.27	9.23	5.80	3.96
	5685	13.20	11.30	9.63	5.77	3.48	2.33

Table E.8. FWD deflection data, March 1996, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
179+20	15127	58.10	48.93	41.39	23.71	13.09	8.11
	11485	45.70	38.22	32.24	18.06	9.97	6.16
	8688	34.17	28.31	23.72	13.04	7.01	4.50
	5498	20.87	17.24	14.28	7.65	4.15	2.69
178+95	15322	31.15	27.02	24.61	17.32	12.02	8.53
	11695	24.57	21.46	19.59	13.77	9.48	6.72
	8807	18.67	16.35	14.93	10.43	7.14	5.07
	5661	11.98	10.48	9.52	6.62	4.44	3.13
178+70	15215	24.23	20.75	19.04	13.64	9.28	6.83
	11802	19.15	16.50	15.20	10.86	7.28	5.51
	8902	14.26	12.67	11.69	8.32	5.66	4.07
	5645	9.24	8.20	7.53	5.32	3.58	2.58
178+45	15374	29.17	24.94	22.30	14.77	9.40	6.47
	11636	23.16	20.11	17.93	11.87	7.50	5.08
	8759	17.82	15.45	13.79	9.09	5.70	3.84
	5502	11.39	9.98	8.89	5.82	3.57	2.35
178+20	15346	25.30	21.87	19.71	13.41	8.92	6.27
	11743	20.18	17.50	15.76	10.67	7.06	4.91
	8807	15.52	13.56	12.19	8.22	5.37	3.69
	5585	10.09	8.84	7.93	5.28	3.39	2.28
177+95	15445	26.31	22.15	19.59	12.51	8.18	5.94
	11584	20.87	17.70	15.63	9.91	6.43	4.62
	8831	16.17	13.74	12.15	7.67	4.94	3.52
	5522	10.56	9.00	7.93	4.91	3.08	2.16
177+50	15147	27.31	22.69	20.20	13.38	8.85	6.39
	11703	21.83	18.30	16.26	10.69	7.03	4.99
	8958	17.13	14.24	12.68	8.20	5.39	3.85
	5661	11.43	9.49	8.34	5.38	3.39	2.32

Table E.9. FWD deflection data, October 2000, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+50	16347	23.46	21.27	19.51	14.79	10.87	8.12
	12311	18.87	17.06	15.66	11.86	8.66	6.46
	9129	13.95	12.78	11.74	8.85	6.55	4.80
	6189	9.41	8.65	7.89	5.96	4.29	3.22
177+95	16152	21.51	19.58	18.17	13.60	9.82	7.35
	12144	17.13	15.54	14.39	10.79	7.78	5.78
	8978	12.62	11.47	10.61	7.91	5.68	4.24
	5987	8.48	7.67	7.07	5.25	3.74	2.76
178+20	16041	22.00	20.32	18.78	14.33	10.54	7.95
	12073	17.52	16.20	14.94	11.37	8.36	6.32
	8910	12.93	11.91	11.02	8.40	6.15	4.66
	5939	8.69	7.98	7.33	5.63	3.96	3.17
178+45	16128	18.54	16.72	15.26	11.12	7.88	5.84
	12092	14.57	13.19	12.01	8.73	6.15	4.56
	8910	10.66	9.60	8.70	6.36	4.43	3.28
	6086	7.13	6.46	5.81	4.25	2.93	2.17
178+70	16069	20.04	18.02	16.48	12.17	8.57	6.37
	12164	16.00	14.36	13.12	9.65	6.79	5.04
	8986	11.76	10.52	9.61	7.01	5.01	3.66
	6074	7.91	7.05	6.41	4.65	3.24	2.41
178+95	15993	21.21	18.95	17.29	12.69	9.04	6.60
	12013	16.81	15.12	13.74	10.09	7.10	5.22
	8799	12.32	11.12	10.04	7.33	5.16	3.80
	5923	8.29	7.47	6.77	4.95	3.44	2.53
179+20	15970	25.26	22.53	20.55	14.62	10.18	7.48
	11965	20.24	18.05	16.46	11.68	8.08	5.89
	8732	15.04	13.38	12.18	8.63	5.92	4.31
	5852	10.07	9.02	8.20	5.77	3.93	2.82

Table E.10. FWD deflection data, October 2000, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
179+20	16073	21.14	18.80	17.28	13.03	9.30	6.99
	12017	16.98	15.13	13.94	10.48	7.54	5.66
	8898	12.72	11.35	10.42	7.80	5.62	4.22
	5951	8.53	7.65	7.00	5.24	3.74	2.79
178+95	15898	22.73	20.22	18.59	13.63	9.54	6.83
	11941	18.35	16.28	14.89	10.93	7.54	5.43
	8791	13.67	12.17	11.16	8.12	5.63	4.00
	5891	9.20	8.12	7.39	5.37	3.61	2.60
178+70	15882	20.14	17.84	16.28	12.00	8.53	6.25
	11902	16.25	14.41	13.14	9.64	6.82	4.96
	8755	12.12	10.76	9.82	7.16	5.02	3.67
	5919	8.13	7.24	6.58	4.77	3.27	2.36
178+45	16013	21.64	18.94	17.03	11.84	8.02	5.86
	11953	17.49	15.31	13.75	9.54	6.42	4.63
	8819	13.16	11.52	10.35	7.15	4.76	3.37
	5911	8.87	7.77	6.91	4.74	3.13	2.20
178+20	15906	23.77	21.03	19.02	13.22	8.89	6.49
	11965	19.29	17.04	15.40	10.68	7.09	5.16
	8728	14.42	12.71	11.45	7.85	5.17	3.74
	5923	9.75	8.68	7.72	5.28	3.41	2.47
177+95	15835	22.62	20.05	18.27	13.44	9.73	7.37
	11906	18.49	16.34	14.94	10.92	7.83	5.93
	8783	13.83	12.20	11.12	8.11	5.78	4.35
	5911	9.35	8.20	7.44	5.38	3.80	2.85
177+50	15838	23.51	21.17	19.51	14.46	10.42	7.97
	11878	18.88	16.97	15.64	11.59	8.31	6.23
	8775	13.95	12.52	11.49	8.47	6.04	4.47
	5820	9.22	8.26	7.52	5.52	3.87	2.82

Table E.11. FWD deflection data, August 2003, northbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
177+73	16192	23.27	21.58	19.81	14.97	10.44	7.56
	11890	18.34	17.31	15.10	12.28	8.17	5.93
	8942	13.33	12.56	10.93	8.89	5.89	4.13
	5995	8.78	8.12	7.35	5.63	3.84	2.82
177+98	16180	22.01	19.89	18.04	13.27	9.33	6.78
	12204	17.59	15.87	14.42	10.59	7.41	5.39
	9109	12.89	11.62	10.43	7.61	5.38	3.93
	6074	8.52	7.74	7.00	5.15	3.56	2.56
178+23	16117	21.39	19.01	17.25	12.54	8.83	6.47
	12128	16.93	15.10	13.68	9.96	7.00	5.09
	9002	12.35	11.05	10.00	7.25	5.07	3.70
	5963	8.22	7.35	6.63	4.80	3.26	2.43
178+48	16168	21.41	19.51	17.99	13.34	9.34	6.82
	11779	17.07	15.51	14.27	10.60	7.38	5.36
	8902	12.42	11.42	10.50	7.76	5.44	3.85
	5927	8.27	7.62	6.96	5.11	3.50	2.55
178+73	16049	21.36	19.50	17.72	12.57	8.42	6.04
	11783	16.81	15.31	13.89	9.82	6.65	4.65
	8831	12.19	11.04	10.00	7.00	4.78	3.33
	5796	7.93	7.25	6.53	4.50	3.07	2.23
178+98	15989	22.55	20.57	18.64	13.25	8.96	6.48
	11675	17.70	16.29	14.57	10.51	7.05	5.07
	8759	12.94	11.87	10.59	7.83	5.13	3.73
	5971	8.46	7.86	6.96	5.01	3.31	2.38

Table E.12. FWD deflection data, August 2003, southbound lane.

Station	Applied Load (lb)	Measured Deflection at Distance from Load (mils)					
		0.00 in.	8.00 in.	12.00 in.	24.00 in.	36.00 in.	48.00 in.
178+98	16085	25.94	23.24	20.95	14.85	9.79	7.41
	12208	20.76	18.62	16.72	11.88	7.78	5.91
	9073	15.16	13.65	12.21	8.61	5.64	4.32
	6026	9.85	8.91	7.94	5.55	3.56	2.72
178+73	15982	19.90	18.08	16.53	12.12	8.48	6.21
	11842	15.95	14.50	13.27	9.70	6.80	4.93
	8871	11.76	10.70	9.78	7.13	4.93	3.56
	5788	7.71	7.07	6.44	4.66	3.16	2.27
178+48	15874	23.40	20.92	19.13	13.96	9.63	6.98
	11914	18.64	16.74	15.27	11.12	7.68	5.50
	8922	13.72	12.33	11.24	8.18	5.62	4.02
	5832	9.00	8.20	7.40	5.30	3.71	2.55
178+23	15942	21.30	18.40	16.52	11.63	7.97	5.79
	12061	16.94	14.67	13.11	9.20	6.34	4.53
	8962	12.44	10.73	9.67	6.73	4.56	3.29
	5804	8.13	7.10	6.33	4.39	2.96	2.06
177+98	15870	23.54	21.08	19.16	13.95	9.87	7.32
	11957	18.94	17.02	15.50	11.24	7.87	5.86
	8914	14.02	12.57	11.40	8.21	5.74	4.25
	5792	9.31	8.38	7.57	5.41	3.74	2.76
177+73	15886	25.80	22.47	20.02	13.72	9.40	6.96
	12033	20.80	18.17	16.15	10.99	7.46	5.51
	8962	15.50	13.53	12.00	8.07	5.40	4.00
	5816	10.42	9.07	8.00	5.29	3.51	2.55