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Chapter 14
PAVEMENT SUBGRADE

14.1   GENERAL

14.1.1   Overview

The supporting ground beneath a pavement structure is called the subgrade. The subgrade is located below the pavement and base and subbase courses. It extends to such depths as may be important to structural design and pavement life, and it may consist of materials in excavations (cuts) or embankments (fills). This chapter addresses the primary geotechnical attributes of subgrade soils. The intent of this chapter is not to provide pavement analysis or design guidelines.

14.1.2   Division of Responsibilities

The design of pavements requires the combined effort of several sections within the Department. The following describes the responsibilities of these various units:

1.   District. The District laboratory is responsible for performing shallow auger borings along the proposed roadway alignment. Samples of the subgrade soils are collected along the roadway centerline. When collecting the soil samples, the District will determine the soil classification. They will also conduct moisture, density and other index classification tests.

2.   Physical Test Section. The Physical Test Section is responsible for conducting R-value laboratory tests of the materials provided by the District. Based on the R-value test results and the soil classification, the Physical Test Section will determine R-values to be used in the pavement design. *Note that consultants are allowed to use CBR values in lieu of R-values in developing their pavement designs.*

3.   Geotechnical Section. The Geotechnical Section performs a subsurface investigation along the proposed roadway alignment to obtain information on the subgrade soils. This investigation may consist of conventional borings, geophysical testing or in-situ testing (e.g., SPT, CPT, vane shear tests). See Figure 8.2-A for guidance on the number and depth of the borings. Geotechnical laboratory testing will be performed on the soil/rock samples for the purposes of obtaining classification, consolidation and strength data. The Geotechnical Section evaluates the suitability of the subgrade with respect to constructability, overall stability and settlement. Based on this evaluation, design recommendations are provided for subgrades in cut and fill areas. Specific recommendations are provided for the following:

- subgrade undercuts (dig-outs);

- special borrow locations, note that special borrow is typically A-1-A soils and is used to backfill undercuts;

- geotextiles;

- subgrade drainage (e.g., subgrade crown, lateral and edge drains);

- subgrade stabilization (e.g., geogrid, geotextiles);

- subgrade chemical treatment (e.g., lime and cement stabilization); and

- erosion susceptibility location.

4.   Pavement Analysis Section.   The Pavement Analysis Section is responsible for designing the pavement structure from the subgrade up to the driving surface.

14.1.3   References

For further guidance on pavement subgrade, the project geotechnical specialist should review the following documents:

- *Soil Stabilization in Pavement Structures, Volume 1 and 2*, FHWA-IP-80-1 and IP-80-2, FHWA;

- *Highway Subdrainage Design*, FHWA TS-80-224, FHWA;

- *AASHTO Guide for Design of Pavement Structures* and Supplement;

- NHI Course No. 54085675, *Soils and Foundations Workshop Manual*, NHI-00-045, FHWA;

- NHI Course No. 132040, *Geotechnical Aspects of Pavements*, NHI-05-037, FHWA; and

- *Frozen Ground Engineering*, Andersland and Ladanyi, ASCE Press, 2004.

14.2   GEOTECHNICAL CHARACTERIZATION OF SUBGRADE

14.2.1   General

The strength, stiffness, compressibility and moisture characteristics of materials underlying the pavement structure can have a significant influence on pavement performance and long-term maintenance. The subgrade and base layers must be strong enough to resist shear failure and have adequate stiffness to minimize vertical deflection. Stronger and stiffer materials provide a more effective foundation for the riding surface and will be more resistant to stresses from repeated loadings and environmental conditions.

A critical component of the pavement design is, therefore, the characterization of the material upon which the pavement structure will be constructed. In cases where the subgrade is inadequate, methods of improving the existing subgrade conditions must be identified. An important part of this evaluation is the balance between initial construction costs and long-term operations and maintenance costs. These trade-offs are best resolved through direct discussions between the project geotechnical specialist and the Pavement Management Section.

14.2.2   Methods of Characterization

A number of laboratory and in-situ methods are available for characterizing the strength and stiffness of subgrade soils and crushed base course aggregates. This section provides a brief overview of the test methods most commonly used on MDT projects.

14.2.2.1   R-Value

The Resistance R-value and Expansion Pressure of Compacted Soils test (commonly called the R-value test) is used by Department to evaluate the strength and stability of the subgrade and base materials that support the final pavement surface. The test is conducted in accordance with AASHTO T-190 and ASTM D-2844.

R-value tests are used to evaluate the capacity of a soil for supporting overlying layers of soil, crushed aggregate and pavement. The R-value test output is a number ranging from 0 to 100, with 0 representing a viscous liquid slurry with no shear resistance, and 100 representing a rigid solid. Typically, R-values range from less than 5 for the poorest soil to as high as 85 for the best performing soil (e.g., a crushed base course or a well-graded, angular sandy gravel). The accuracy of the test decreases when the R-value is small. Consequently, when an R-value smaller than 5 is measured, the result is typically reported as "< 5" or "–5." High R-values (70 to 80) correspond to high stiffness. Stiffer soils have relatively higher resilient moduli, which indicate less potential for differential settlement and rutting. Consequently, the necessary thickness of the base course and subbase course layers underlying the riding surface can be reduced as their R-values increase.

MDT currently uses results from the R-value test to design the thickness of each layer in a flexible pavement section. The R-value is indirectly related to a soil's resilient modulus, and the value has been correlated with various other strength/stability measures, as shown in Figure

14.2-A, which provides approximate correlations between R-values and AASHTO soil classifications.

All R-value testing conducted by MDT is performed at the MDT Physical Testing Section in Helena. The test requires about 40 lb (18 kg) of soil. Depending on the stage of the project, the soil may be obtained from a borrow source, material stockpile(s) or from the roadway subgrade. A large sack containing about 70 lb (32 kg) of material is generally obtained for R-value testing. This provides the lab technicians with extra soil for performing additional tests (e.g., gradation analyses, Proctor tests).

The *MDT Materials Manual* contains additional information regarding proper methods for obtaining soil samples. The test takes several days to perform, and there is often a 5-day to 7-day turnaround period before results are reported back to field personnel.

14.2.2.2   California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test has been used in pavement design since the mid 1940s. Some state transportation departments continue to use this method; however, MDT uses the R-value test method. The test provides a relative measure of subgrade soil or base course aggregate strength and stability.

During a CBR test, a soil sample is compacted into a cylindrical mold, loaded with surcharge weights and soaked for 4 days. A circular piston having an end area of 3 in^2 (1900 mm^2) is forced into the soil at a standard rate of 0.05 in/min (1.3 mm/min). The unit load at each 0.1 in (2.5 mm) of penetration up to 0.5 in (13 mm) is recorded, and the CBR is computed as a ratio of load required for 0.1 in (2.5 mm) of penetration over a standard load of 3000 lbs (1400 kg). As a general rule, the CBR will decrease as the penetration value increases. Consequently, the ratio at 0.1 in (2.5 mm) of penetration is typically used for design; however, in some cases other values of penetration may be more appropriate. For some soils, the load at 0.2 in (5 mm) of penetration may be higher than the load at 0.1 in (2.5 mm) of penetration. In this case, the ratio computed at 0.2 in (5 mm) of penetration is used for design. Details of the test method are described in AASHTO T-193 and ASTM D-1883-05.

Although the CBR test is one of the most widely used tests for evaluating subgrade support values, some agencies and labs invoke variations to the standard AASHTO procedures. It is important that any variations of the standard procedure are reported with the CBR results.

14.2.2.3   Resilient Modulus (M_R)

The resilient modulus (M_R) is a measure of the elastic property of soil recognizing certain nonlinear characteristics. M_R can be used directly for the design of flexible pavements, but must be converted to a subgrade reaction (K-value) for the design of rigid or composite pavements. M_R is a basic material property that can be used in mechanistic analysis of multilayered systems for predicting roughness, cracking, rutting, faulting, etc. Methods for determining M_R are described in AASHTO T-307 and FHWA LTPP Protocol 46.
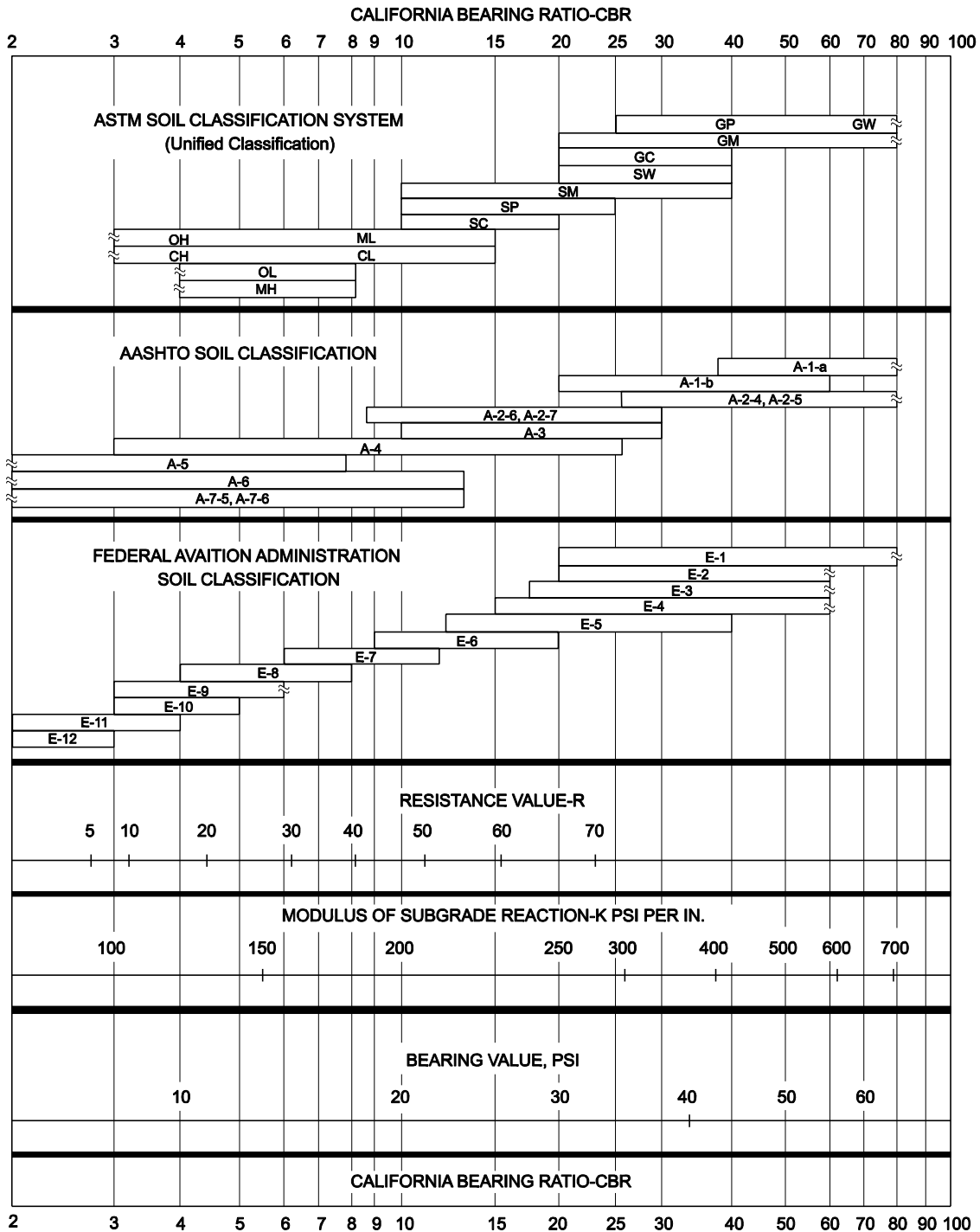
Figure 14.2-A — R-VALUE CORRELATION CHARTS

14.2.2.4   Falling Weight Deflectometer (FWD)

Many nondestructive devices are available to assist the designer in the structural evaluation of in-situ pavements. In addition to pavement structural analysis, these devices are used for back-calculating the elastic moduli of various pavement components, evaluating load transfer efficiency across joints and cracks in concrete pavements and determining the location and extent of voids under concrete slabs. Nondestructive testing devices employ a variety of technologies, including deflection measurements, wave propagation, impact hammer testing, ground-penetrating radar and various impedance devices.

The Pavement Analysis Section conducts nondestructive in-situ testing using a mobile falling weight deflectometer (FWD). The FWD delivers a transient force impulse to the pavement surface by dropping a weight from a predetermined height to obtain a peak force ranging from 1500 lb to 24,000 lb (7 kN to 109 kN). The load is transmitted to the pavement through a loading plate in the form of a half sine wave with duration of 25 to 30 milliseconds. The magnitude of load is measured by a load cell. Deflections are measured using three to seven (depending on the FWD model) velocity transducers (geophones) mounted on a bar that can be lowered automatically to the pavement surface with the loading plate. The FWD is equipped with a microprocessor-based control console for storing and processing data.

Pavement performance and structural capacity is assessed using the FWD through the use of maximum elastic deflection measurements in combination with an indicator of the radius of curvature of the pavement under load. The average Resilient Modulus (Mr) of the subgrade to a depth of about 48 in (1.2 m) can be back-calculated if the thickness of the pavement and base is known. Most FWD programs should, therefore, include a coring program to provide input information for the deflection evaluation. The Resilient Modulus gives an indication of a soil's potential response to loading by heavy equipment during construction, and loading by traffic after the road is constructed.

"Network-level" FWD information is gathered by MDT at approximately 800 ft (250 m) intervals for all on-system roads in Montana on a 3-year cycle. This information, although widely spaced, provides a preliminary idea of subgrade conditions, especially when viewed in conjunction with the District Soil Survey borings. Because the Network-level data have likely been obtained at the same location at different times of the year, the amount of seasonal moisture change within a soil may be roughly estimated by observing the changes in Resilient Modulus at a given point.

"Project level" FWD information is gathered soon after a project is nominated; consequently, this information likely will be available before geotechnical drilling begins. In order to provide Geotechnical Section advance notice of FWD testing, the Nondestructive Testing (NDT) Unit will forward a list of projects to the District Geotechnical Managers for review before the NDT crews start testing projects. FWD testing takes place before the PFR and soil survey are completed; therefore, an additional field review would be required from the Geotechnical Section in order to determine if additional FWD locations are warranted. On future projects, the spacing of FWD testing likely will be shortened from 330 ft (100 m) to approximately 250 ft (80 m) spacing.

Although the exact nature of the subgrade cannot be determined from a Resilient Modulus graph, breaks or changes in material may be apparent, that will aid in determining the placement of geotechnical borings. In general, the geotechnical boring data will define subsurface conditions, while the FWD information will help determine the variation in subgrade

soil conditions. If changes occur over long stretches, varying surfacing pavement sections might be designed within a single project. On projects in which sporadic areas of very soft subgrade may require special treatment (e.g., a subexcavation replaced by granular backfill and geogrid or separation geotextile), the drilling and FWD data can be used together to better define the areas of concern. Projects with the potential for frequent areas of very soft subgrade may require a large quantity of subgrade treatment. In these cases, it may be warranted to request FWD data at closer intervals to further define the problem areas and reduce the unnecessary treatments that are invariably set up in the unknown material that lies between two known problematic locations. At a gathering rate of 1 point per minute, FWD information is relatively inexpensive.

Additional information, interpretation methods and guidelines for use of the FWD are described in the *AASHTO Guide for Design of Pavement Structures*. Contact the Pavement Analysis Section for specific details about this test.

14.2.3   Geotechnical Design Considerations

The performance of the roadway pavement surface is significantly affected by the characteristics of the subgrade. Desirable properties that the subgrade should possess include strength, drainage, ease of compaction and low compressibility. Adequate subgrade compaction is an essential ingredient for obtaining a high-quality travel surface. Compaction reduces settlement, increases density, increases strength and decreases the sensitivity of the subgrade soil to changes in moisture content.

14.2.3.1   Zone of Influence

As shown in Figure 14.2-B, the vehicle wheel load is distributed through the pavement structural section (hot mix asphalt or Portland cement concrete) and into the underlying base, subbase and subgrade. Distress will be observed in the pavement riding surface if any of these layers are constructed using materials that are not in compliance with the specifications, or if the layers are not well-compacted.

The depth of influence for wheel loading is usually relatively limited – say to a maximum of 5 ft to 10 ft (1.5 m to 3.0 m) below the planned pavement surface elevation. Within this depth, the transient loads from traffic will cause repetitions of load cycle. Both the magnitude and the number of cycles are considered during the design of the pavement section. Field and laboratory testing programs should focus on classifying the consistency of this material. For fill sections, the testing will involve evaluations of imported borrow material, whereas the native material will be evaluated for cut sections. Bulk samples of the top 2 ft (0.6 m) of the subgrade should be obtained for laboratory testing. These tests are typically conducted during the preconstruction soil survey phase of the project.
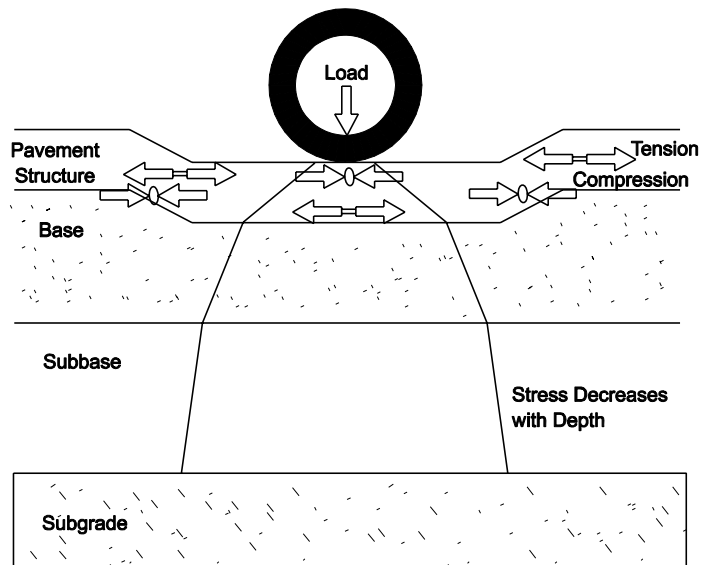
Figure 14.2-B — LOAD DISTRIBUTION IN ROADWAY CROSS SECTION

14.2.3.2   Construction

Prior to placing subbase material (if used) or base material if a subbase is not used, the subgrade surface should be leveled, compacted and inspected by qualified personnel. The inspection process includes visual examinations to detect loose, soft or pumping areas and field compaction tests (nuclear density gage) at selected locations to measure the dry density and moisture content.

If unsuitable subgrade soil is encountered, it is usually excavated, removed and replaced with suitable backfill material. If high groundwater is encountered, drainage measures and stabilization geotextiles are often necessary to provide a firm foundation to support the overlying layers and to increase the trafficability of construction equipment.

It is the Contractor's responsibility to construct the work in accordance with the plans and specifications. The Department's inspectors have a responsibility to thoroughly inspect the subgrade in cooperation with the Contractor's work forces. The following items should be considered for proper subgrade preparation.

1.   If present, frozen earth, snow and ice should be removed from the subgrade area.

2.   The full width of the subgrade should be cleared of sod and vegetative matter.

3.   The top 8 in (200 mm) of subgrade should be scarified, watered and compacted to 95% of the Standard Proctor maximum dry density.

4. The subgrade surface should be proof-rolled with a heavy wheeled vehicle to detect soft, loose or pumping areas. If detected, these areas should be mitigated to improve the conditions.

5. Ensure that the subgrade surface is adequately protected from climatic elements and traffic after the subgrade is approved in accordance with the specifications.

14.2.3.3 Selection of R-Values for Conceptual Design and Estimating

As a general trend, lower R-values are associated with higher AASHTO soil classification categories. For example, it is not uncommon to obtain an R-value < 5 for a highly plastic clay, which may be classified as an A-7-6 soil. This type of soil would be considered undesirable for a roadway subgrade, but is often used in areas of Montana where these soils are predominant or based upon economic considerations. On the other extreme, a well-graded sandy gravel (classification A-1-a) may have an R-value of 80 and would constitute an excellent subgrade material.

An inverse relationship exists between the subgrade R-value and the thickness of the pavement section. In other words, a site that has a low R-value subgrade soil will require a thicker pavement section (pavement, base course, subbase) than a site that has a high R-value subgrade soil. The trend described above can be readily observed in Figure 14.2-C, which was developed from tests performed by the MDT Materials Bureau on soil samples obtained from various locations around the State. The higher AASHTO soil classification numbers are indicative of soils that contain a larger percentage of fines (soil particles smaller than the #200 sieve). Strength and stability of fine-grained soils are generally low, especially when exposed to water. The fines reduce the overall capacity of the soil because they reduce the particle-to-particle contact that provides strength to a soil matrix.

The typical ranges of R-values shown in Figure 14.2-C are useful for conceptual design and estimating purposes, and for checking the reasonableness of laboratory test results. However, because the final pavement section thickness is based on the R-value (and anticipated traffic volumes), it is imperative that representative soil samples be obtained and tested to obtain values for design. The ranges of values shown in Figure 14.2-C are too approximate for use in design of the roadway section.

Pavement sections are sometimes designed with a subbase (commonly referred to as special borrow by MDT) that is intended to provide a higher R-value material below the base course that will ultimately allow for a reduced base course and/or asphalt thickness. The soil type that is used for special borrow is sometimes specified in contract documents only by a minimum R-value. The project geotechnical specialist should be cognizant of this situation because, as shown in Figure 14.2-C, many soil types can potentially satisfy the same minimum R-value. A particular soil type may satisfy the required minimum R-value, but may not satisfy other geotechnical design requirements. An example would be where a silt could potentially satisfy the required minimum R-value criteria, but this soil type is highly frost susceptible and the particular project may have experienced frost heaving problems in the past. In order to minimize these types of situations the project geotechnical specialist should recommend that the special borrow be specified in the contract documents by both a minimum R-value and a required soil classification.

Soil Classification	Type of Soil	R-Value Range
A-1-a	sandy gravel	50 – 85
A-1-b	gravely sand	50 – 75
A-3	fine sand	50 – 75
A-2-4	silty or clayey sand, PI < 10	10 – 45
A-2-6	silty or clayey sand, PI > 11	< 5 – 30
A-4	silty soil with LL < 40	< 5 – 50
A-5	silty soil with LL > 41	< 5 – 40
A-6	clayey soil with LL < 40	< 5 – 30
A-7-5	clayey soil with LL > 41 and PI < (LL-30)	< 5 – 30
A-7-6	clayey soil with LL > 41 and PI > (LL-30)	< 5 – 20

Figure 14.2-C — TYPICAL RANGES OF R-VALUES FOR MONTANA SOILS

14.2.3.4   In-Slope Drainage

It is common practice in Montana and other States to place 3 in to 4 in (75 mm to 100 mm) of topsoil on the in-slopes of roadways to facilitate the establishment of vegetation. The Geotechnical Section should review these situations on a case-by-case basis to evaluate the potential for drainage issues associated with placement of low permeability cover soil. In some situations, the topsoil cover can clog or block the drainage path of the base and subbase courses. This could lead to excessive water accumulation in the base and possibly lead to premature pavement distress and failure.

Where the potential for clogging is determined to be high, the Geotechnical Section should recommend an alternative design scheme. These alternative schemes can involve specific measures to drain water from the side of the roadways or simply to recommend against topsoiling the inslopes.

14.3   GEOSYNTHETICS

Geosynthetic products are frequently used to help stabilize poor soil conditions at embankment foundation areas and other locations during construction. Geosynthetics used as an aid during the construction of the roadway may serve the functions of reinforcement, separation, filtration and drainage. Chapter 20 provides more detailed coverage of geosynthetic applications.

Geosynthetics can also be used as a tensile element at the bottom of a base (or subbase) or within a base course to:

* improve the service life of the pavement, and/or
* provide equivalent performance with a reduced structural section.

Do not use geosynthetics without a detailed geotechnical evaluation. The mechanisms of geosynthetic base reinforcement are complex and not always easily represented by simple design methodologies. There has been a tendency for distributors of geosynthetic products to "oversell" their products, resulting in unnecessary cost to the project with little benefit and poor performance on some projects. However, with appropriate care and use of current design guidance summarized in textbooks and FHWA design guidance documents, improved performance of pavements and subgrades can result. Where soils and load conditions are complex, performance of geosynthetics in base reinforcement applications is best determined by product-specific testing. Laboratory and/or field tests with specific products, similar pavement materials and cross-sections, and similar subgrade conditions should be used to quantify the contribution of the geosynthetic reinforcement to the pavement performance. Design procedures incorporate the results of product-specific testing and use a traffic benefit ratio (TBR), base course reduction (BCR) percentage, or layer coefficient ratio (LCR) value.

Base reinforcement design includes the following steps:

1.   assess applicability through careful review of soil conditions and proposed loads;

2.   perform an unreinforced design;

3.   select the target benefit in terms of service life improvement or reduced structural section;

4.   evaluate the benefit offered by various geosynthetics in terms of TBR, BCR or LCR;

5.   perform base course reinforcement design; and

6.   perform life-cycle cost analysis.

For some projects, particularly those with a base and subbase, two layers of geosynthetic reinforcement may be used to provide both subgrade restraint and base reinforcement. Each layer of reinforcement should be independently designed.

14.4   SUBGRADE CHEMICAL TREATMENT

14.4.1   General

Subgrade soils that are unsatisfactory in their natural state can be altered by admixtures, by the addition of aggregate or by proper compaction, and thus made suitable for subgrade construction. This section addresses admixture stabilization methods, which may be warranted for a number of reasons, including:

- soft or weak soils (R-value < 5),
- high plasticity soils,
- excessively wet soils,
- expansive soils,
- stabilization or salvaging of unpaved roads,
- dust control, and
- shrink/swell control.

14.4.2   Alternatives

Chemical admixtures are generally categorized according to the properties imparted to the soil. Types of admixtures include cementing agents, modifiers, waterproofing agents, water-retaining agents, water-retarding agents and miscellaneous chemicals. The behavior of each of these admixtures is vastly different from the others. Each has its particular use; and conversely, each has its own set of limitations. The most common chemical admixtures include:

- Portland cement,
- lime, and
- lime-flyash mixtures.

Portland cement can be used to stabilize in-situ subgrade soils, reduce the plasticity index of fine-grained soils, strengthen crushed base courses and improve the surface of existing gravel roads. Portland cement stabilization can be used with granular soils, silty soils and lean clays, but it cannot be used in organic materials. Because soil cement has the capacity for relatively rapid and substantial strength gains, it is the most commonly used additive for base course construction. The percentage of Portland cement required for low plasticity clayey soils generally ranges from 9% to 15% by weight, while sandy soils generally require about 5% to 9% by weight.

Lime, or hydrated lime, increases soil strength primarily by pozzolanic action, which involves the formation of cementatious silicates and aluminicates. Lime is most efficient when used in granular materials and lean clays. The percentage of lime necessary for adequate hydration is generally low (2% to 5% by weight). Lime can also increase the potential for swell in soils containing high sulfate contents, which are common in the eastern half of Montana.

Flyash is generally rich in silica and alumina; consequently, the addition of flyash to lime stabilized soil speeds the pozzolanic action. However, the quantity of flyash required for adequate stabilization is relatively high (10% to 20% by weight), restricting its use to areas that have available large quantities of flyash at relatively low cost.

14.4.3   Geotechnical Design Considerations

Chemical modification of subgrade soils consists of uniformly mixing dry Portland cement, lime, flyash or a combination of the materials with the soil to improve the strength and workability of soils that have excessively high-moisture content. The choice of the proper admixture depends upon the use for which it is intended. The quantity of admixture is generally determined by means of laboratory tests, which should evaluate the blended material's strength. The response of the blended material to weather and material handling (abrasion and degradation) and chemical interactions with sulfate bearing soils should also be evaluated. Various types of admixture stabilizers are summarized in Figure 14.4-A.

The following are guidelines regarding the application of chemical admixtures:

1.   Temperature. Chemical soil modification generally should not be conducted when the soil temperature drops below 45°F (7°C) measured 4 in (100 mm) below the ground surface. The modifier should not be mixed with frozen soils.

2.   Soil Type. When type A-6 or A-7 soils are encountered, the soil surface should be scarified prior to placement of the admixture. The admixture should be uniformly distributed using a cyclone, screw-type or pressure manifold distributor. Spreading of the admixture is usually limited to an amount that can be placed into the soil within the same work day and during acceptable wind conditions.

3.   Mixing. The admixture, soil and possibly, water is thoroughly mixed by rotary speed mixers until a homogenous layer of the blended material is obtained. Thickness of the layer should be determined during design. Admixture layer thicknesses generally range from 9 in to 16 in (225 mm to 400 mm).

4.   Compaction. Compaction of the mixture should begin as soon as practical, but generally should be started no later than the following guidelines indicate:

   a.   Cement modified soils. Begin compaction within 30 min of cement placement. Compaction should be completed within 3 hr after mixing.

   b.   Lime modified soils. Compaction should be conducted within 24 hr of mixing.

   c.   Flyash modified soils. Compaction should be conducted within 4 hr of mixing.

5.   Moisture content.   The compaction effort should be in accordance with recommendations provided in the mix design. The moisture content of the mixture should be between optimum moisture and optimum plus 2%.

6.   Traffic. Construction traffic should not be allowed on the treated soils within 72 hr of compaction.

Type	Admixture	Primary Mechanics of Stabilization	Use	Situations Best Suited	Approx. Quantity by Weight	Method of Evaluation	Change Soil Property				Construction Procedure	Limitations
							γ	LL	PL	PI		
Increase Strength by Cementing Action — Cementing Agents	Portland Cement	Principally hydration, some modification of clay minerals	Base and subbase	Sandy soils or lean clays	A-7 9%-15% to A-2 5%-9%	Durability tests, compression	Decrease[a]	Slight reduction	Increase	Decrease	Pulverize, mix, compact, cure	Organic soils
	Lime	Change water film, flocculation, chemical	Some base and subbase, shoulders	Granular materials or lean clays	2%-5%	Durability tests, compression	Decrease[a]	Varies	Increase	Decrease	Pulverize, mix, compact, cure	Freeze-thaw may be destructive
	Lime Flyash	Pozzolanic action of lime and silica, some modification of clay minerals	Some base and subbase, shoulders	Granular materials or lean clays	2%-5% lime 10%-20% flyash	Durability tests, compression	Decrease[a]	Varies	Increase	Decrease	Pulverize, mix, compact, cure	Quantity of flyash may be high
	Bitumen		Base and subbase	Granular	2%-5%							
Improve Plasticity May or May Not Increase Strength — Modifiers	Cement	Modification of clay, change water film	Improve poorly graded base and subbase	Improve existing road material, clays	½%-4%	Atterburg limits Grain size		Varies	Increase	Decrease	Pulverize, mix, compact	Strength increase may be low
	Lime	Change water film, modification of clay minerals	Improve poorly graded base and subbase	Improve existing road material, clays	½%-4%	Atterburg limits Grain size		Varies	Increase	Decrease	Pulverize, mix, compact	Strength increase may be low
	Bitumen	Retards moisture absorption	Improve poorly graded base and subbase	Improve existing road material	1%-3%	Absorption tests					Pulverize, mix, compact	Strength increase may be low
Little or No Increased Strength — Water-proofing Agents	Bitumen	Retards moisture sorption by coating soil grains	Primarily subbase	Sandy soils or poor quality base materials, some clays	4%-6%	Water sorption, compression, volume change	Decrease				Mix, cure, compact	Limited by soil plasticity
	Membranes	Prevents movement of free water and water vapor	Primarily subbase and subgrades	Soils that may be improved by compaction		Strength of natural soil	None	None	None	None	Compact to high density	Construction may be difficult
Water-retaining Agents	Calcium Chloride	Deliquescent properties, lowers freezing point, base exchange	Construction expedient, traffic binding	Graded aggregate	½%-1½%	Arbitrary	Slight increase	Slight reduction	None		Spread dry or mix with water	Leaches
	Sodium Chloride	Deliquescent properties, lowers freezing point	Construction expedient, traffic binding	Graded aggregate	½%-1½%	Arbitrary	Slight increase	Slight reduction		None	Spread dry or mix with water	Leaches
Water Retarding	Organic Cationic Compounds	Alters clay minerals to act as a hydrophobic agent	Subbases		Trace	Water sorption, compression, volume change					Mix and compact	Mixing very small quantities may be difficult

[a] *In some cases a slight increase is shown for granular soils.*

Figure 14.4-A — ADMIXTURE STABILIZATION

14.5   RECLAIMED/RECYCLED ASPHALT PAVEMENT (RAP)

14.5.1   General

Reclaimed/Recycled Asphalt Pavement (RAP) is bituminous concrete material removed and reprocessed from pavements undergoing reconstruction or resurfacing.   Reclaiming the bituminous concrete may involve either cold milling a portion of the existing bituminous concrete pavement or full depth removal followed by crushing.

RAP has potential for use as granular base or subbase material in virtually all pavement types, including paved and unpaved roadways, parking areas, bicycle paths, gravel road rehabilitation, shoulders, residential driveways, trench backfill, engineered fill, pipe bedding and culvert backfill. RAP can also be used in place of aggregate or soil in embankments and some non-structural backfill situations. MDT has evaluated the use of RAP and its properties. Results of this work can be found on the Department's website.

14.5.2   Design Considerations

When considering the use of RAP for a project, review the following guidance:

1.   Blending.   The key design parameter for incorporating processed RAP into granular base material is the blending ratio of RAP to conventional aggregate that is needed to provide adequate bearing capacity. The ratio can be determined from laboratory testing of RAP aggregate blends using the R-value or CBR test methods, or by relying on previous relevant experience. It has been reported that blends of up to 40% asphalt-coated particles from RAP have been successfully incorporated into blended granular base material. RAP produced by grinding or pulverizing has a lower bearing capacity than crushed RAP because the grinding and pulverizing operation tends to generate more fines.   For load-bearing applications, granular RAP is usually blended with conventional aggregates.

2.   Design.   Conventional AASHTO pavement structural design procedures can be employed for granular base containing RAP. Review the *AASHTO Pavement Design Guide* for the thickness design of base course or subbase layers that contain RAP as a percentage of the base or subbase.

3.   RAP Portion. If the RAP is only a portion of the base or subbase material (less than 30%), the structural layer coefficient normally recommended for granular base materials (0.11 to 0.14) can be used. If the RAP constitutes a greater percentage, or even all of the base or subbase material, some adjustment of the structural layer coefficient may be considered.

14.5.3   Construction Considerations

Essentially the same equipment and procedures used to stockpile, handle and place conventional aggregates in granular base are applicable to blended granular material containing RAP. For major projects where control of engineering properties is critical, controlled blending

of the RAP with conventional granular material at a central batch plant provides better consistency than the product of in-place, full-depth processing.

Because each source of RAP will be different, random sampling and testing of the RAP stockpile needs to be performed to quantify and qualify the RAP. Samples of the stockpiled RAP should be used to determine the optimum blend of materials. Additional care is required during stockpiling and handling to avoid segregation or re-agglomerating.

To avoid agglomeration of crushed RAP, it should be blended as soon as practical with conventional aggregate (using a cold feed system) to a homogeneous mixture. However, blended material that is stockpiled for a considerable period of time, particularly in warm weather, may harden and require re-crushing and re-screening before it can be incorporated into granular base applications. Blended RAP-aggregate stockpiles should not be allowed to remain in place for extended periods in most climates because the stockpiled material is likely to become overly wet, possibly requiring some drying prior to use.

Conventional granular aggregates do not bond well with RAP or blended granular material containing RAP. Consequently, raveling can occur if thin layers of conventional aggregates are placed over material containing RAP. During placement, finish grading can be difficult because of the adhesion of asphalt in the RAP. Adequate compaction is important to avoid post-construction densification of granular base materials containing RAP.

14.5.4   Quality Control

The same laboratory and field test procedures used for quality control of conventional aggregate are appropriate for granular base/subbase containing RAP. Testing of moisture content and compacted density using nuclear gauges is affected by the presence of RAP. Both parameters tend to be overestimated because of the presence of hydrogen ions in the asphalt cement contributing to the total count. To avoid this problem, compaction of granular base containing RAP should be carried out using a control strip. Laboratory moisture checks should be completed to calibrate nuclear density gauge moisture content readings.

14.6   SUBGRADE UNDERCUT (DIG-OUTS)

14.6.1   General

Long-term performance of the pavement riding surface is affected by the characteristics of the subgrade. Desirable properties that the subgrade should possess include:

- strength,
- drainage,
- ease of compaction,
- permanency of compaction and
- permanency of strength.

Soil is a highly variable material; the interrelationship of soil texture, density, moisture content and strength are complex, and in particular, behavior under repeated loads is difficult to evaluate. Because subgrade soil varies considerably, it is necessary to conduct a thorough study of the in-situ soils, and from this, determine the design of the pavement. At some project sites, the in-situ soil will not possess all of the desirable properties listed previously. In these cases, the subgrade soils will require overexcavation and replacement with suitable granular fill (typically select A-1-a soil). The Department refers to these areas as undercuts or dig-outs.

14.6.2   Geotechnical Design Considerations

Because of the complexity of the problem, it is not practical to set down rules that will be suitable for all cases. The purpose of this section is to provide general principles that can be incorporated as part of the investigation, analysis and pavement design process.

Some soils are undesirable under nearly all conditions and should be removed if practical prior to construction of the subgrade. These include the following:

- soils that contain large quantities of mica;

- soils that have a high organic content;

- highly expansive soils or soils that have high swell potential;

- volcanic soils (including volcanic ash and andosols);

- A-7-5 and A-7-6 soils with high group indices. These soils are especially problematic in cut/fill transition zones and locations where subgrade drainage is not well developed, although they are sometimes used; and

- saturated or nearly saturated soils that cannot be excavated using the same equipment and methods as for unclassified excavations.

Subgrade areas requiring overexcavation may be identified during preconstruction. This would typically occur during the geotechnical investigation phase of a project. In this circumstance, the project plans will specifically indicate the location and depth of required sub-excavation, and

the specifications will describe the requirements for removal and disposal of unsuitable material as well as backfill requirements.

Preconstruction subsurface investigations will not successfully locate 100% of all unsuitable subgrade soils. When unexpected unsuitable soils are encountered during construction, these soils will be overexcavated (also called dig-out situations) at the direction of the project engineer. Dig-outs generally extend to a maximum depth of 2 ft to 5 ft (0.6 m to 1.5 m) below top of subgrade (bottom of base course elevation), depending on the thickness and consistency of the soft material. For example, peat might be excavated to 5 ft (1.5 m) or greater; whereas, medium stiff clay may require only 2 ft (0.6 m) of excavation. The decision on the amount of dig-out is usually made in the field by the MDT field inspector in consultation with the project geotechnical specialist. Dig-outs are typically backfilled in compacted lifts using A-1-a soils or imported granular fill.

14.7   FROST SUSCEPTIBILITY

14.7.1   General

As ground temperatures decrease to temperatures below 32°F (0°C), water within a soil mass freezes and undergoes a 9% volumetric increase as a result of the water-to-ice phase change. Lenses of frozen water form in this zone and draw additional moisture upward through capillary forces, which, in turn, freeze and expand. The movement or displacement associated with this process is termed frost heave.

The depth of frost penetration and the amount of heave depend on many factors including soil type, soil permeability, moisture content, elevation and climatic conditions. The seasonal frost layer is generally described as the top layer of soil in which temperatures fluctuate above and below 32°F (0°C) during the year. For example, in Bozeman, Montana the maximum depth of frost ranges from about 2 ft to 3.3 ft (0.6 m to 1 m) and in some select areas of Montana can be up to 5 ft (1.5 m) or more.

As temperatures warm in the spring, thawing commences from above and below the frozen layer. As shown in Figure 14.7-A, during thaw, segregated water may not effectively drain from the soil because the surrounding frozen ground is relatively impermeable. Consequently, the subgrade becomes temporarily saturated with water, which reduces the bearing capacity (strength) of the soil for supporting vehicular traffic or other loads. This process is commonly referred to as spring thaw or thaw weakening. During spring thaw, paved roads on top of frost-susceptible soils may experience a loss of 50% or more of the normal bearing capacity, while gravel-surfaced roads on frost-susceptible soil may experience bearing capacity losses in excess of 70%.

Early studies by Casagrande and others focused on identifying conditions in which frost action may occur, and characterizing soils that may be frost susceptible. Results of their research are usually presented in the form of the following three basic requirements:
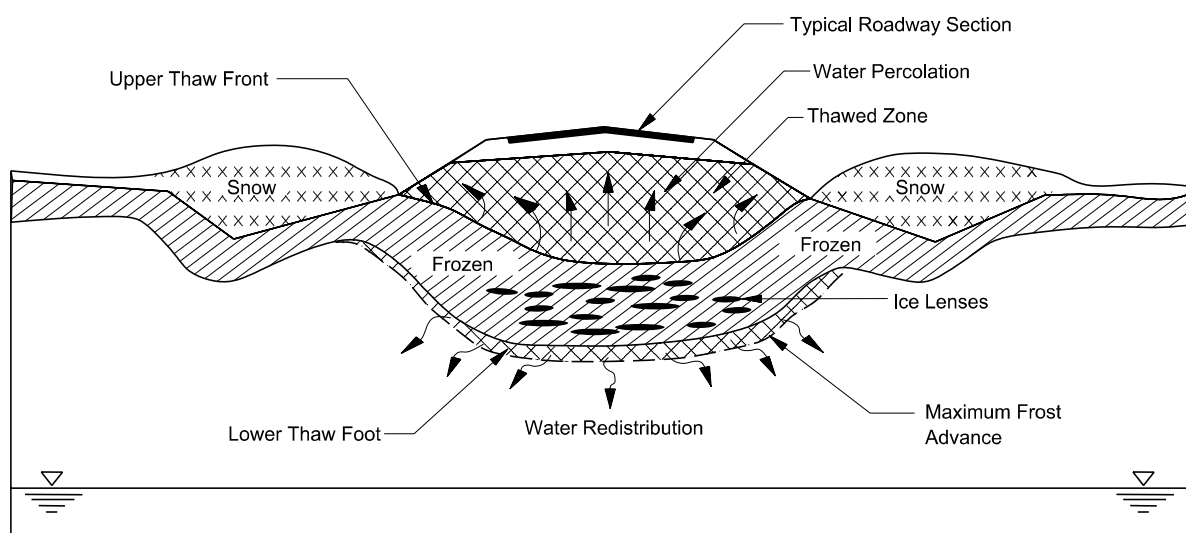

Figure 14.7-A ⎯ SCHEMATIC CROSS-SECTION OF ROADWAY DURING THAW

- temperatures below freezing,
- source of water close enough to supply capillary water, and
- frost-susceptible soil based on grain size distribution.

Unfortunately, these general guidelines comprise the extent of frost heave knowledge for many practitioners. Catastrophic failures of engineering structures (e.g., buildings, bridges, embankments, roadways) are fortunately quite rare. However, damage attributable to frost action is readily apparent in many areas of North America. Shortcomings in current design methodologies likely can be attributed to the complex physical, rheologic and geomorphologic mechanisms that occur when a heterogeneous mass of soil and water is exposed to repeated cycles of freezing and thawing.

Various equations and computer software have been developed to describe and evaluate the effects of frost heave on soils (e.g., "The Segregation Potential of a Freezing Soil," *Canadian Geotechnical Journal*, Vol. 18, 1981; *Frozen Ground* Engineering (Andersland and Ladanyi, 2004)).

14.7.2   **Frost-Susceptible Soils**

A frost-susceptible soil is defined in terms of its frost-heaving and thaw-weakening behavior. Both can cause considerable damage to engineering structures and roadways, the former during freezing and the latter during thawing. Frost heave is not necessary for thaw weakening. Some clay soils develop segregated ice while exhibiting little or no heave. Shrinkage or consolidation of layers adjacent to an ice lens cancels the heave normally associated with ice segregation, particularly where the water supply is limited and soil permeability is low. Frost-susceptibility index tests permit evaluation of the potential for frost heaving and thaw weakening of subgrade soils and unbound base course materials.

The most frost-susceptible soils are

- silt,
- very fine sand,
- silty sand,
- low plasticity clay, and
- varved and banded sediments.

In terms of the USCS system, the following soil types have the greatest potential for frost susceptibility: ML, MH, SM, SP-SM, SC, CL and CL-ML. In terms of the AASHTO system, the following soil types have the greatest potential for frost susceptibility: all soils within the A-2, A-4, A-5, A-6 and A-7 categories. The frost susceptibility soil classification system are provided in Figure 14.7-B is useful for general guidelines. This table and additional frost susceptibility criteria are available in *Frozen Ground Engineering* (Andersland and Ladanyi, 2004).

Frost Susceptibility[1]	Frost Group	Kind of Soil	Amount Finer Than 0.001 in (0.02 mm) (wt %)	Typical Soil Type Under USCS[2]
Negligible to low	NFS[3]	Gravels	0 – 1.5	GW, GP
		Sands	0 – 3	SW, SP
Possibly	PFS[4]	Gravels	1.5 – 3	GW, GP
		Sands	3 – 10	SW, SP
Low to medium	S1	Gravels	3 – 6	GW, GP, GW-GM, GP-GM
Very low to high	S2	Sands	3 – 6	SW, SP, SW-SM, SP-SM
Very low to high	F1	Gravels	6 – 10	GM, GW-GM, GP-GM
Medium to high	F2	Gravels	10 – 20	GM, GM-GC, GW-GM, GP-GM
Very low to very high		Sands	6 – 15	SM, SW-SM, SP-SM
Medium to high	F3	Gravels	> 20	GM, GC
Low to high		Sands, except very fine silty sands	> 15	SM, SC
Very low to very high		Clays, I_p > 12	—	CL, CH
Low to very high	F4	All silts	—	ML, MH
Very low to very high		Very fine silty sands	> 15	SM
Low to very high		Clays, I_p > 12	—	CL, CL-ML
Very low to very high		Varved clays and other fine-grained banded sediments	—	CL and ML; CL, ML and SM; CL, CH and ML; CL, CH, ML and SM

1. Based on laboratory frost-heave tests.

2. G-gravel; S-sand; M-silt; C-clay; W-well graded; H-high plasticity; L-low plasticity.

3. Non-frost-susceptible.

4. Requires laboratory frost-heave test to determine frost susceptibility.

**Figure 14.7-B — US ARMY CORPS OF ENGINEERS
FROST DESIGN SOIL CLASSIFICATION SYSTEM**

14.7.3   Geotechnical Design Considerations

Subfreezing temperatures and frost-related problems are possible on all highway construction in Montana. Frost heave and roadbed swelling are important environmental considerations in pavement design because of the potential effect on the rate of serviceability loss. Frost heave refers to localized volume changes that occur in the subgrade soils as moisture collects, freezes into ice lenses and produces permanent distortions in the pavement surface. The effects of frost heave and swelling soils can be decreased by employing one of the following measures, or a combination thereof:

1.   Excavate the frost-susceptible soil within the zone of frost penetration and replace with non-frost-susceptible soil.

2.   Provide an adequate drainage system that will remove and redirect water from the frost zone, and serve as a capillary break to minimize migration of segregation water to frozen areas.

3.   Provide a layer of non-frost-susceptible material thick enough to insulate the subgrade from frost penetration. This not only protects against frost heave, but may also significantly reduce or even eliminate the thaw-weakening that occurs in the subgrade soil during early spring.

A common practice in Montana is to remove frost-susceptible soil and replace with selected non-susceptible material. Where frost-susceptible soils are too extensive for economical removal, they may be covered with a sufficient depth of suitable material to modify the detrimental effects of freezing and thawing. Methods for evaluating the consequences of frost heave and seasonal thaw-weakening and procedures for determining the environmental serviceability loss are provided in the *AASHTO Guide for Design of Pavement Structures*.