HMEC

Highway Materials Engineering Course

PARTICIPANT WORKBOOK

Soils and Foundations

MODULE

B

U.S.Department of Transportation
Federal Highway Administration

Table of Contents

About This Workbook

This workbook has been developed as a resource for participants. This workbook can be used during the training session to follow along with the instructor and take notes, as well as for reference after the module has ended.

Course Overview

The Federal Highway Administration (FHWA) Highway Materials Engineering Course (HMEC) is a comprehensive multi-week training event that consists of eight content “modules” that provide students with the knowledge to develop materials specifications and guidance, make effective acceptance decisions, and design, construct, and maintain assets with a long service life. Modules range in duration for the number of days they take to complete. The modules are:

- Module A: Quality Assurance
- Module B: Soils and Foundations
- Module C: Steel, Welding, and Coatings
- Module D: Aggregates for Transportation Construction Projects
- Module E: Mechanistic Empirical Pavement Design Guide
- Module F: Asphalt Materials and Paving Mixtures
- Module G: Portland Cement Concrete
- Module H: Evaluating Recycled Materials for Beneficial Uses in Transportation

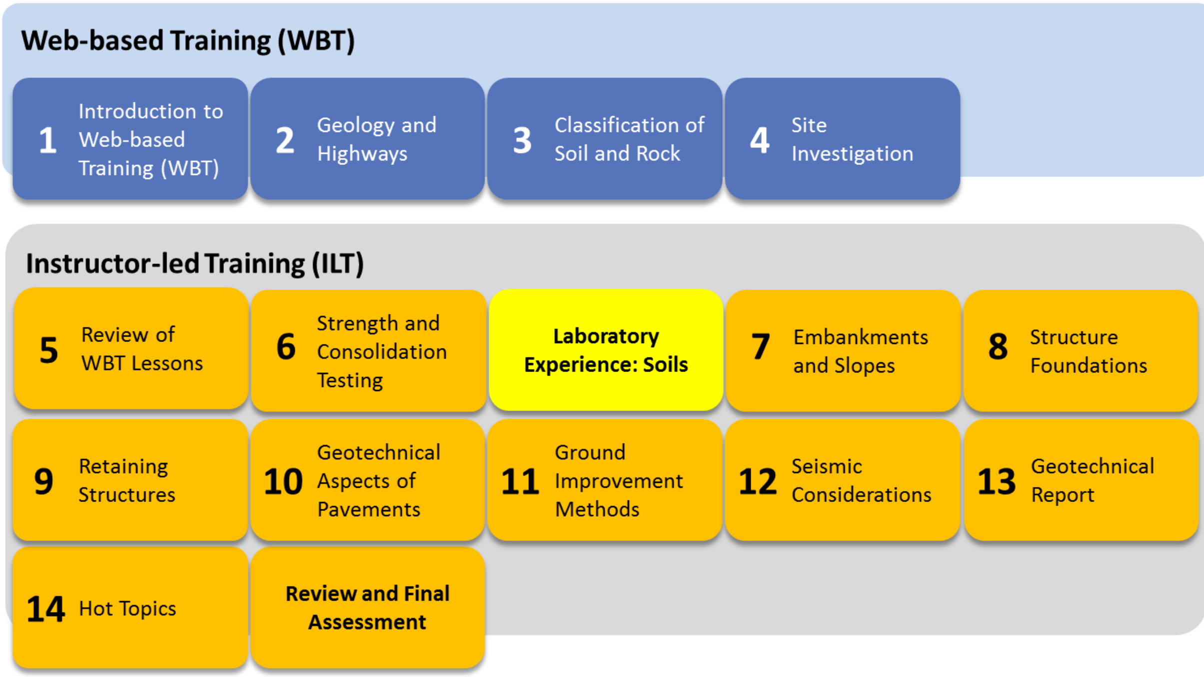
Introduction

Module B: Soils and Foundations is the second module in the FHWA HMEC.

Module B is designed to focus on the group process required for geotechnical issues in the real world, with an emphasis on the role of the materials engineer. Materials engineers should be asking questions, collaborating, and viewing other perspectives in all aspects of geotechnical work. They should know when to ask for help and ask for it when needed.

Module B Overview

Below is a visual overview of all of the lessons covered in this module:



Module Goals

The goals for this module are as follows:

- Recognize the essential geotechnical processes required to design, construct, and maintain a highway system
- Recognize the impact of soil and rock material properties on the design, construction, and maintenance of the geotechnical aspects of highway projects
- Identify the minimum acceptable level of geotechnical effort needed for design and construction of a successful highway project
- Recognize site conditions that indicate the need for further investigation by geotechnical engineers
- Identify potential solutions to geotechnical concerns and participate in the process of determining the best solution

Learning Outcomes

Lesson 1: Introduction

- LO 1.1: Identify important geotechnical features on a typical transportation project
- LO 1.2: State the importance of testing, theory, and experience as applied to soils and foundations
- LO 1.3: Recall the various aspects of a project that require geotechnical involvement

Lesson 2: Geology and Highways

- LO 2.1: Explain the origins of soil and rock materials
- LO 2.2: Describe the behavior of problem ground conditions when applied to highway construction

Lesson 3: Classification of Soil and Rock

- LO 3.1: Differentiate between identification, descriptions, and classification
- LO 3.2: Explain coarse-grained soil types and their basic engineering uses
- LO 3.3: Explain fine soil types and their basic engineering uses
- LO 3.4: Compare AASHTO and USCS soil classification systems, and explain the appropriate use of each
- LO 3.5: List main rock classifications and engineering properties

Lesson 4: Site Investigation

- LO 4.1: Describe the three phases of a site investigation
- LO 4.2: Explain various site exploration and sampling methods
- LO 4.3: Using given project criteria, select an appropriate sampling and site exploration plan

Lesson 5: Introduction to Instructor-led Session and Application of Web-based Training Materials

- LO 5.1: Relate areas of geotechnical involvement to the topics from the WBT
- LO 5.2: Using given project criteria, explain how an appropriate sampling and site exploration plan was selected

Lesson 6: Strength and Consolidation Testing

- LO 6.1: Explain the soil phase system
- LO 6.2: Explain the importance of sample preparation
- LO 6.3: Apply the concept of sampling and testing variation to geotechnical design
- LO 6.4: Explain the effect of water on soil strength as determined by computing and plotting total and effective overburden and water pressure
- LO 6.5 Describe consolidation and shear strength test methods and applications

Laboratory Experience: Soils

- LO Lab 1: Using several soil samples, identify the type of soil and the kinds of testing that are suitable for the soil type
- LO Lab 2: Apply the concept of consolidation to the methods used to test consolidation

Lesson 7: Embankments and Slopes

- LO 7.1: Describe the process for determining the factor of safety
- LO 7.2: Describe potential failure modes in soil and rock
- LO 7.3: Using given criteria, discuss possible solutions to slope stability problems
- LO 7.4: Using given criteria, discuss possible solutions to embankment settlement problems
- LO 7.5: List two rock slope stabilization techniques
- LO 7.6: Describe best practices in construction monitoring methods for stability and settlement

Lesson 8: Structure Foundations

- LO 8.1: Identify standard criteria used for determining foundation type
- LO 8.2: Explain the process for determining foundation type
- LO 8.3: Describe best practices in construction monitoring for structure foundations

Lesson 9: Retaining Structures

- LO 9.1: Describe the retaining wall classification system
- LO 9.2: Identify the wall types used for cut or fill applications
- LO 9.3: List criteria for wall selection
- LO 9.4: Describe best practices in construction monitoring for retaining structures

Lesson 10: Geotechnical Aspects of Pavements

- LO 10.1: List the geotechnical input needed for the design of pavements
- LO 10.2: Explain the geotechnical parameters of interest in pavement design and their effect on the performance of pavements
- LO 10.3: Describe best practices in construction monitoring for pavement subgrades
- LO 10.4: Using given criteria, select an appropriate remediation method for pavement subgrades

Lesson 11: Ground Improvement Methods

- LO 11.1 Describe available ground improvement methods
- LO 11.2 Describe how to select appropriate ground improvement methods

Lesson 12: Seismic Considerations

- LO 12.1: Describe the implications of seismic zones
- LO 12.2: Identify site conditions that indicate seismic hazards

Lesson 13: Geotechnical Report

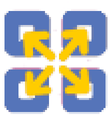
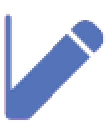
- LO 13.1: Identify the standard sections of a geotechnical report

Lesson 14: Hot Topics

- LO 14.1: Discuss current issues, emerging technologies, and trends that affect geotechnical engineering

ILT Instructor Icons

The following icons are used on the slides as a cue to the instructor and participants:

Icon	Icon Name	Typical Use
	Timer	▪ Call out the estimated time for the lesson
	Important Information	▪ Call out important information.
	Q & A	▪ Check for understanding or agreement. ▪ Survey participants. ▪ Solicit feedback.
	Breakout/Small Group Exercise	▪ Break participants into groups. ▪ Provide directions for exercise.
	Video/Sound	▪ Show a video.
	Reference	▪ Reference another document or resource.
	Links	▪ Share a Web link for additional resources.
	Whiteboard	▪ Draw or document something on a whiteboard or easel pad.

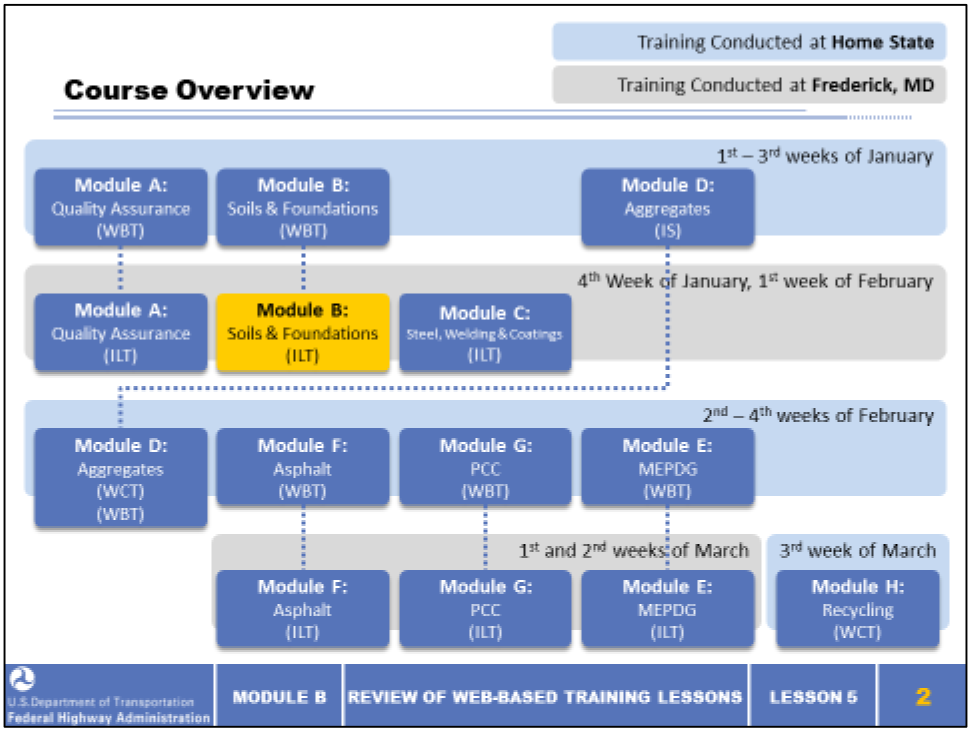
Icon	Icon Name	Typical Use
	Safety	▪ Call out important safety information.
	Common Error	▪ Call out a system or process that is often misused.

This page is intentionally blank.

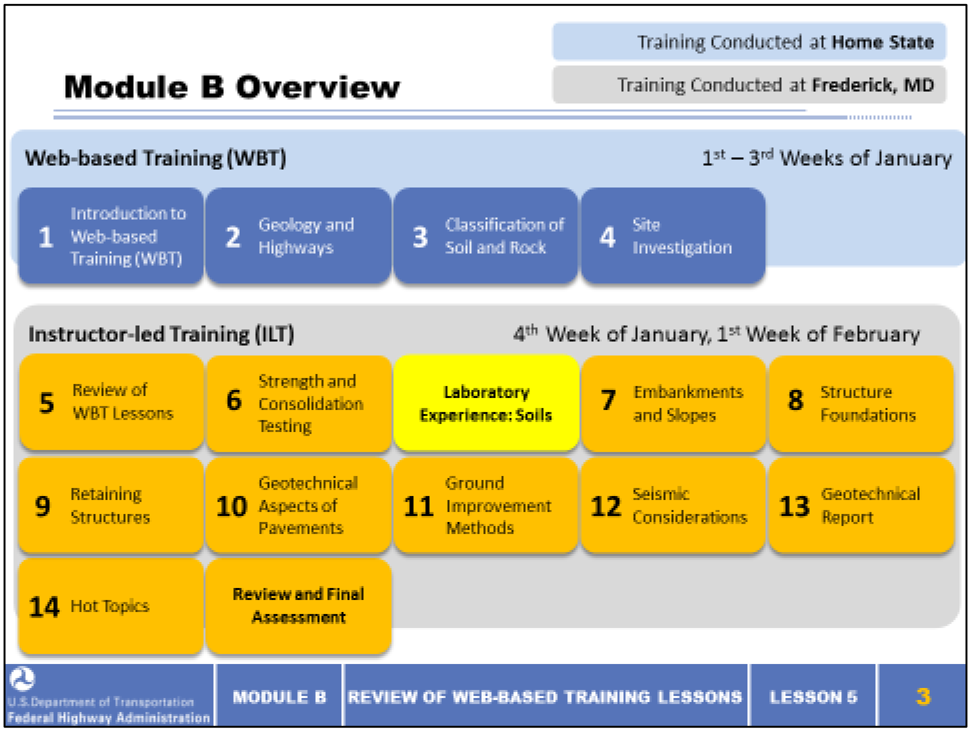
Slide 1


HMEC
Highway Materials Engineering Course
Lesson 5: Review of Web-Based Training Lessons
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2


Training Conducted at Home State
Training Conducted at Frederick, MD
Course Overview
1st – 3rd weeks of January
Module A: Quality Assurance (WBT)
Module B: Soils & Foundations (WBT)
Module D: Aggregates (IS)
4th Week of January, 1st week of February
Module A: Quality Assurance (ILT)
Module B: Soils & Foundations (ILT)
Module C: Steel, Welding & Coatings (ILT)
2nd – 4th weeks of February
Module D: Aggregates (WCT) (WBT)
Module F: Asphalt (WBT)
Module G: PCC (WBT)
Module E: MEPDG (WBT)
1st and 2nd weeks of March
3rd week of March
Module F: Asphalt (ILT)
Module G: PCC (ILT)
Module E: MEPDG (ILT)
Module H: Recycling (WCT)
U.S. Department of Transportation
Federal Highway Administration
MODULE B
REVIEW OF WEB-BASED TRAINING LESSONS
LESSON 5
2

Slide 3



There is reference material available for soils and foundations issues. Specifically you were provided a PDF of the two-volume set of the Soils and Foundations Reference Manual (FHWA_NHI-06-088 and 089). In addition, two publications: 1. Geotechnical Engineering Circular GEC-5, Evaluation of Soil and Rock Properties and 2. Subsurface Investigations - Geotechnical Site Characterization Reference Manual for NHI 132031 are two excellent resources and are available as PDF downloads on the FHWA Geotechnical Engineering Web site (https://www.fhwa.dot.gov/engineering/geotech).

____

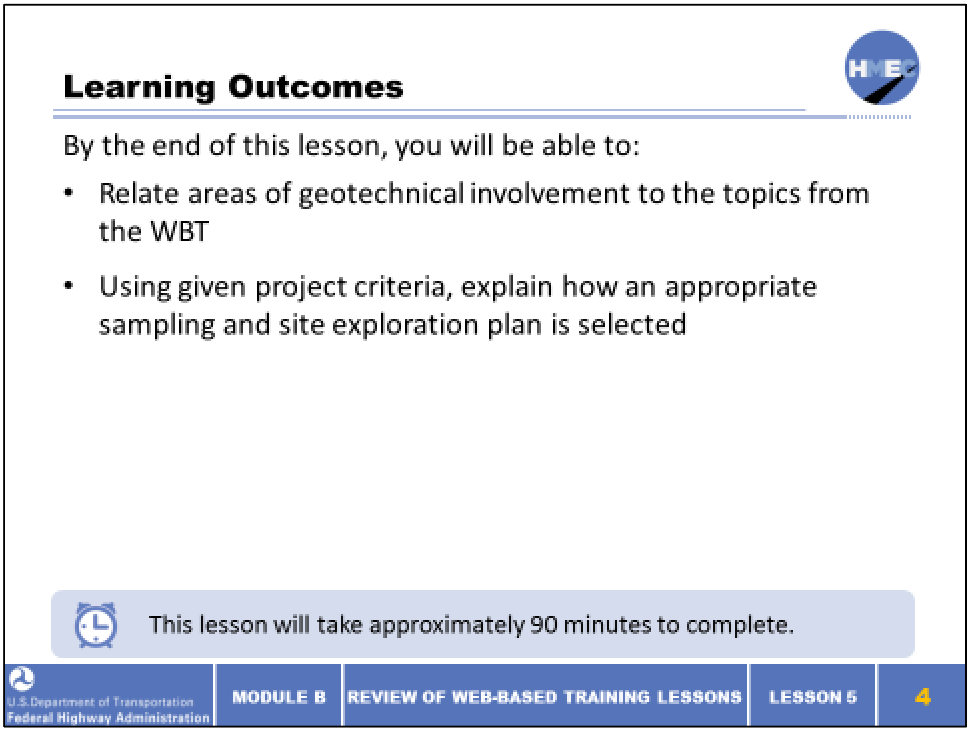
____

____

____

____

Slide 4


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Relate areas of geotechnical involvement to the topics from the WBT
• Using given project criteria, explain how an appropriate sampling and site exploration plan is selected
This lesson will take approximately 90 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
REVIEW OF WEB-BASED TRAINING LESSONS
LESSON 5
4

Slide 5

How many of you are involved in the geotechnical issues discussed in the WBT?

Slide 6

As part of your offline assignment, you were asked to list the features on the example project that required geotechnical involvement. Look at the list you developed and provide a feature that requires geotechnical involvement.

____

____

____

____

____

____

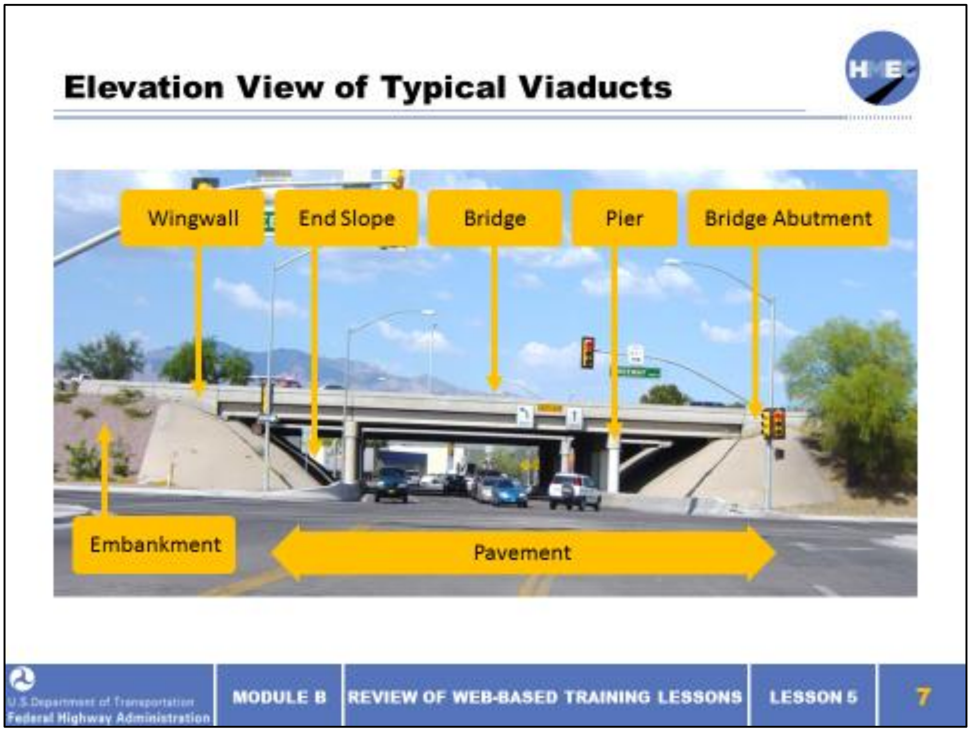
____

____

____

____

Slide 7



Compare the list you provided to a couple of slides from the WBT. Geotechnical issues may include soil and rock bearing capacities, material capacity of foundation types, selection of foundation types, settlement, slope stability, and pavement subgrade.

____

____

____

____

____

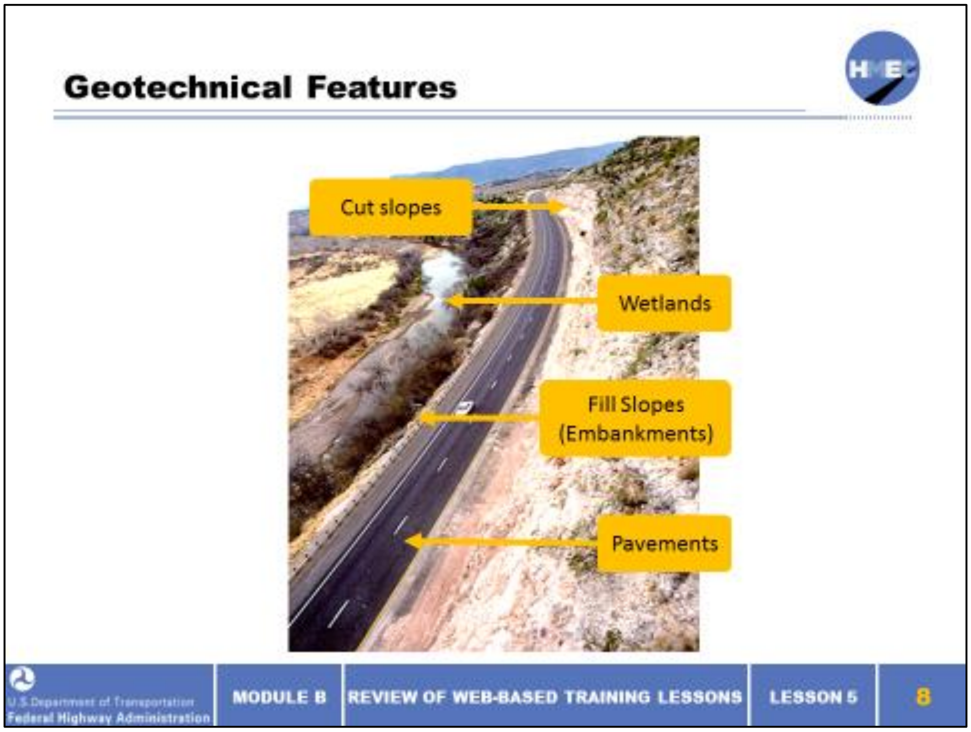
____

____

____

____

Slide 8



Geotechnical features include cut and fill slopes, pavements, and wetland mitigation. Issues to consider for these features include slope stability, groundwater, rock falls, pavement and subgrade design and stability, drainage, scour, and erosion.

Slide 9

Do you recall how these three factors interact?

The proper approach to geotechnical problems involves the combined use of testing, basic theories, and experience. Over-reliance on any one of these aspects will not produce a satisfactory design. The sole use of theory (number-crunching) may produce a wonderful design that cannot be built. Similarly, the sole use of experience (foot-stomping) may produce a design that is at the best, but is not cost effective, and at the worst, unsafe. Soil conditions at each site must be analyzed by obtaining and testing soil samples, applying basic theory to produce a preliminary result, and then tempering the result with previous experience to produce an optimal design.

____

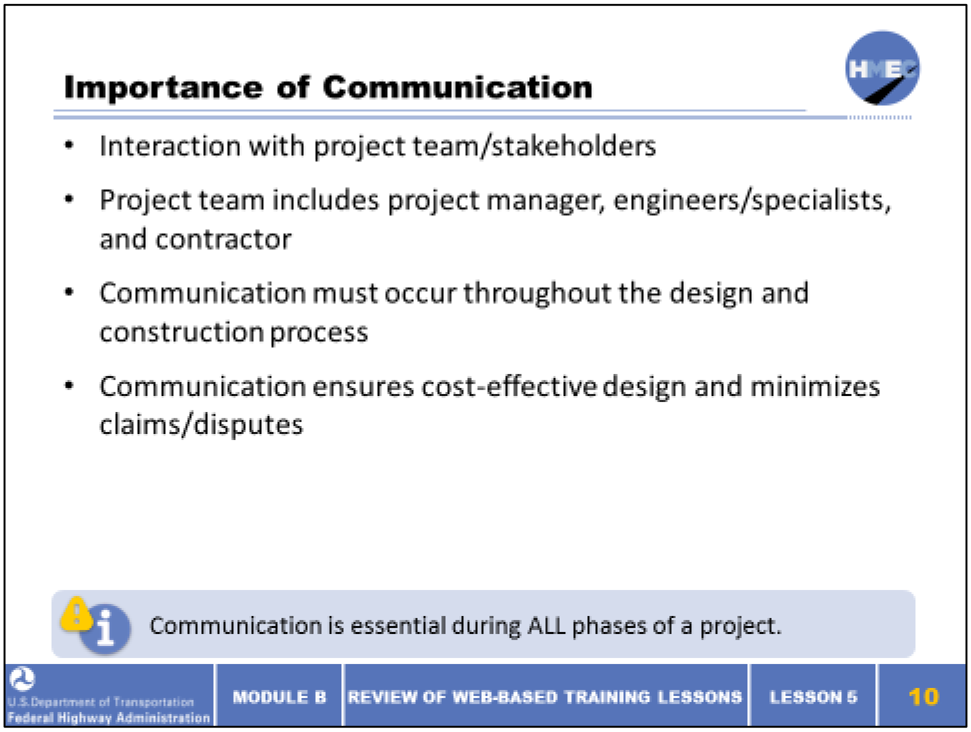
____

____

____

____

Slide 10



Communication is very important. It is how we interact with the project team and stakeholders. The project team includes the project engineer, engineers/specialists (such as the material and geotechnical engineers), and the contractor. Communication must occur throughout the design and construction process to ensure a good design and that construction is in accordance with the specifications. Communication ensures cost-effective design and minimizes claims and disputes. Remember, "Everyone communicates with someone!" Communication is essential during all phases of a project.

____

____

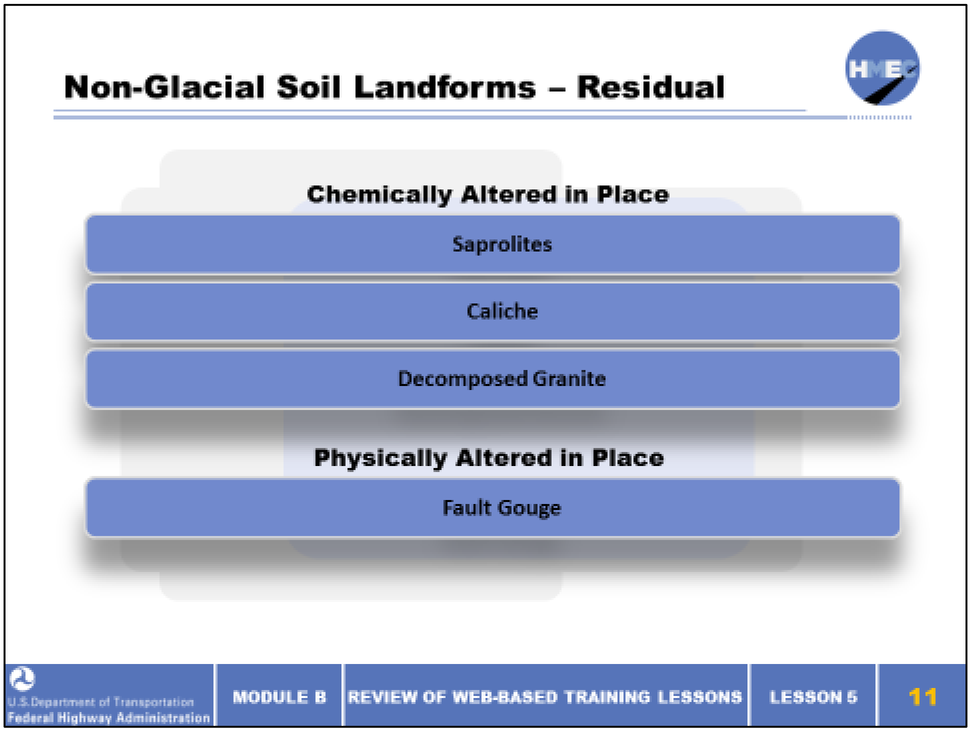
____

____

____

____

Slide 11



Landforms are a form of communication as well. As you may recall, they can give us clues as to what we might find beneath the ground surface. The two primary types of landforms are non-glacial and glacial.

Some non-glacial landforms are comprised of residual soils. Residual soils develop from the surface of bedrock through weathering and chemical action. Weathering agents include rain, wind, abrasion, heat, frost action, and root growth. Soil components reflect the composition of the parent rock. Residual soils weather in-place. Residual soils that develop by chemical weathering include saprolites, caliche, and decomposed granite. One residual soil developed through physical weathering is fault gouge.

Note that landforms can provide clues to help guide site investigations.

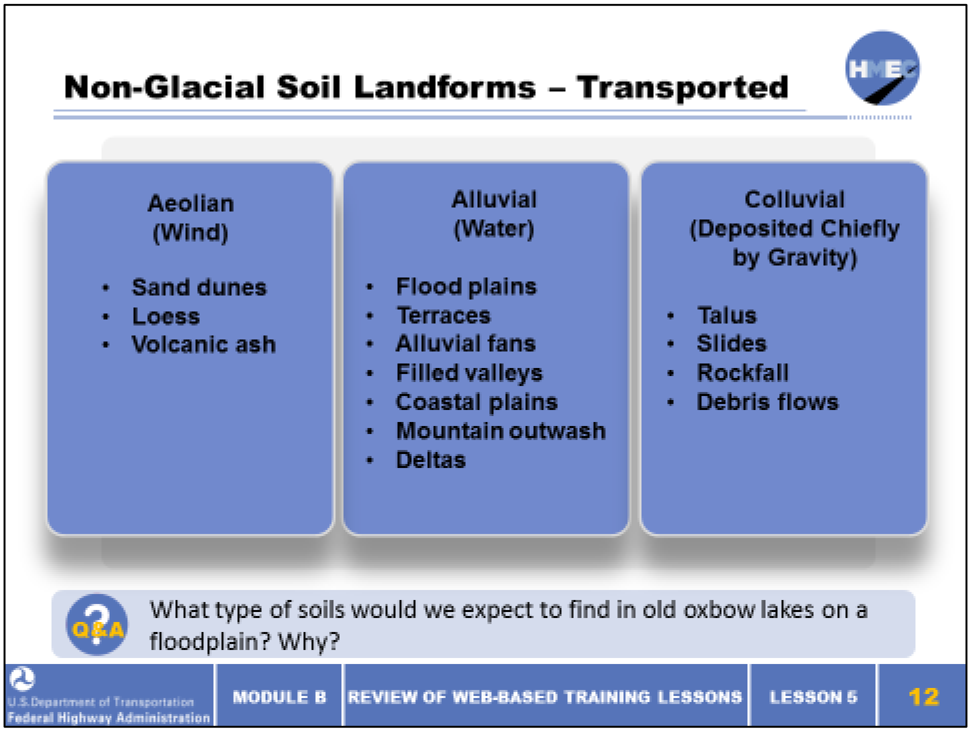
____

____

____

____

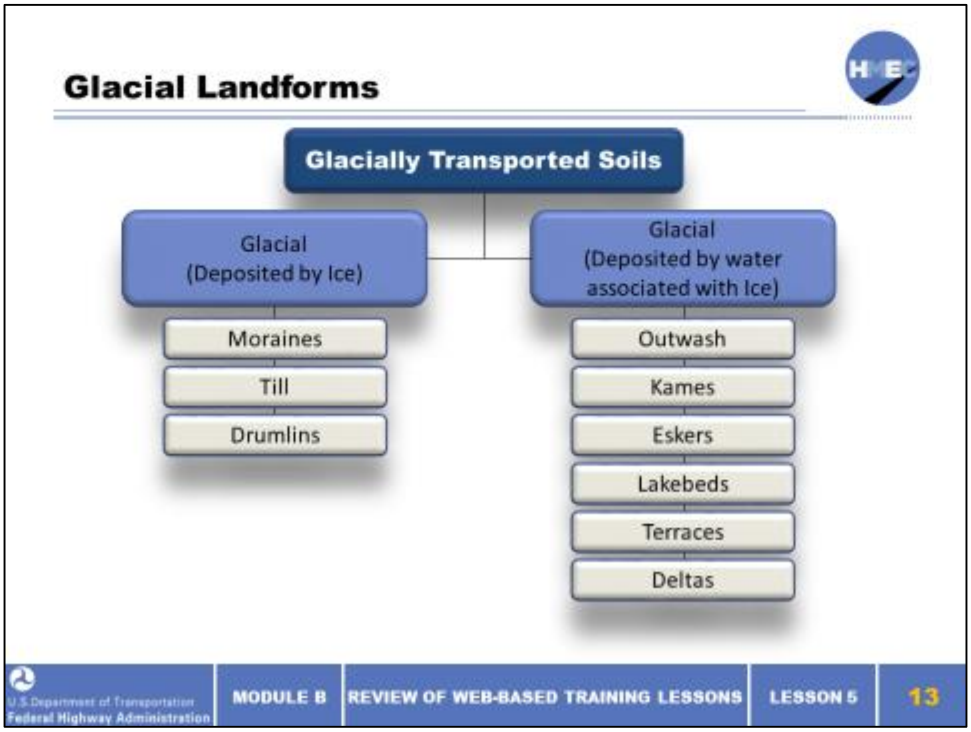
Slide 12



Many non-glacial landforms are comprised of transported soils.

Non-glacial soil landforms consisting of transported soil are created by the actions of Aeolian (wind), alluvial (water), or gravity (colluvial). The landforms created by the forces of wind include sand dunes, loess, and volcanic ash. The landforms created by the forces of water include flood plains, terraces, alluvial fans, filled valleys, coastal plains, mountain outwash, and deltas. The landforms created by the force of gravity include talus, slides, rock fall, and debris flows.

Slide 13



Climatic changes occurred millions of years ago that caused the formation of glaciers or ice sheets in present-day Canada. Continental glaciers continued to expand until, at their maximum, they covered most of the northern Midwest, the northeast, and the northwest US. As they moved over the land and then receded, they created numerous landforms. Landforms deposited by the ice included moraines, till, and drumlins. Landforms deposited by water associated with the glaciers included outwash, kames, eskers, lakebeds, terraces, and deltas.

____

____

____

____

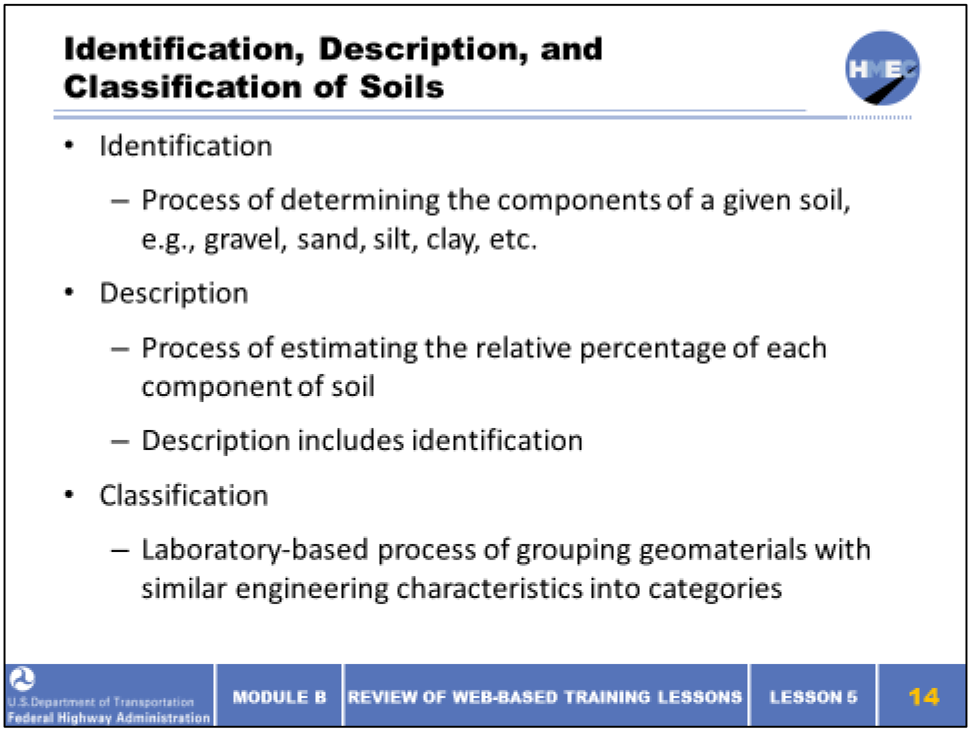
____

____

____

____

Slide 14

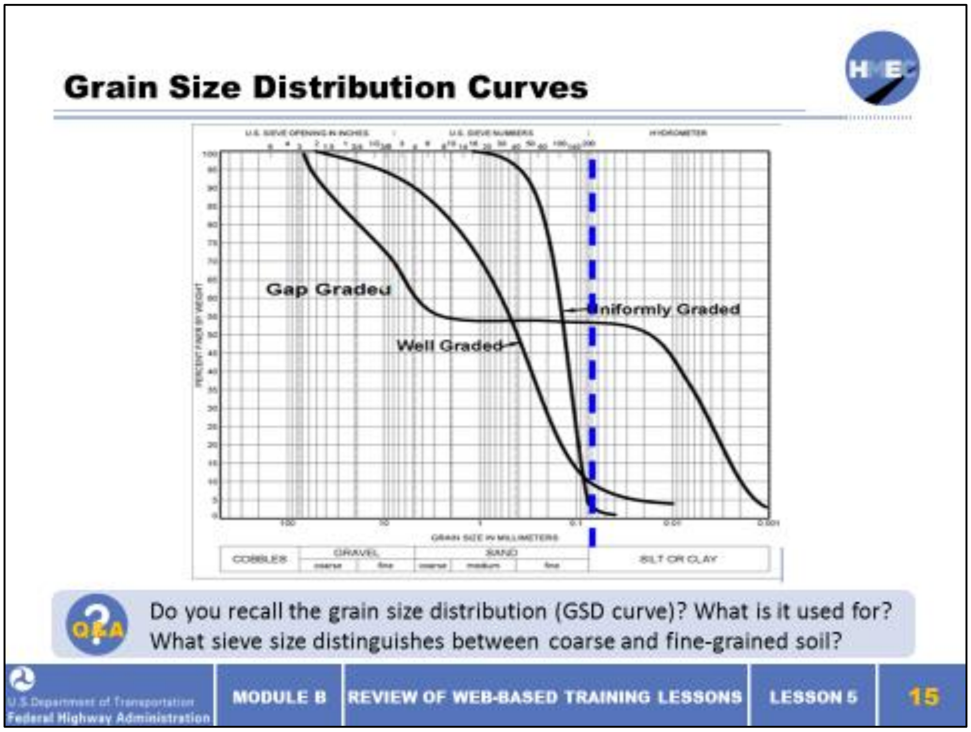


Identification is the process of determining the components of a given soil, such as gravel, sand, silt or clay. Description is the process of estimating the relative percentage of each component and preparing a word picture of the sample in accordance with ASTM D 2488: Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Identification and description are accomplished primarily by both a visual identification and the feel of the sample.

Classification, on the other hand, is a laboratory-based process of grouping soils with similar engineering characteristics into categories based on index test results; e.g., group name and symbol (ASTM D2487: Standard Practice for Classification of Soils for Engineering Purposes, AASHTO M 145).

The important distinction between classification and both identification and description is that standard AASHTO or ASTM tests must be performed to determine a soil's classification. The field personnel should only describe the soils encountered. Group symbols associated with classification should not be used in the field. It is important to send the soil samples to a laboratory for accurate identification classification by a technician experienced in soils work, as this single operation will provide the basis for later testing and soil profile development.

Slide 15



The GSD curve is a graphical representation of the particle size distribution of the material. The GSD indicates whether the material is classified as a coarse- or fine-grained soil. The GSD also allows us to determine whether the material is well graded, uniformly graded, or gap graded.

The friction angle of granular soil is dependent on the grain size and gradation of the soil. Well-graded soils will have greater particle interlock and therefore greater friction between soil grains, resulting in a higher friction angle and strength as compared to poorly-graded soils.

____

____

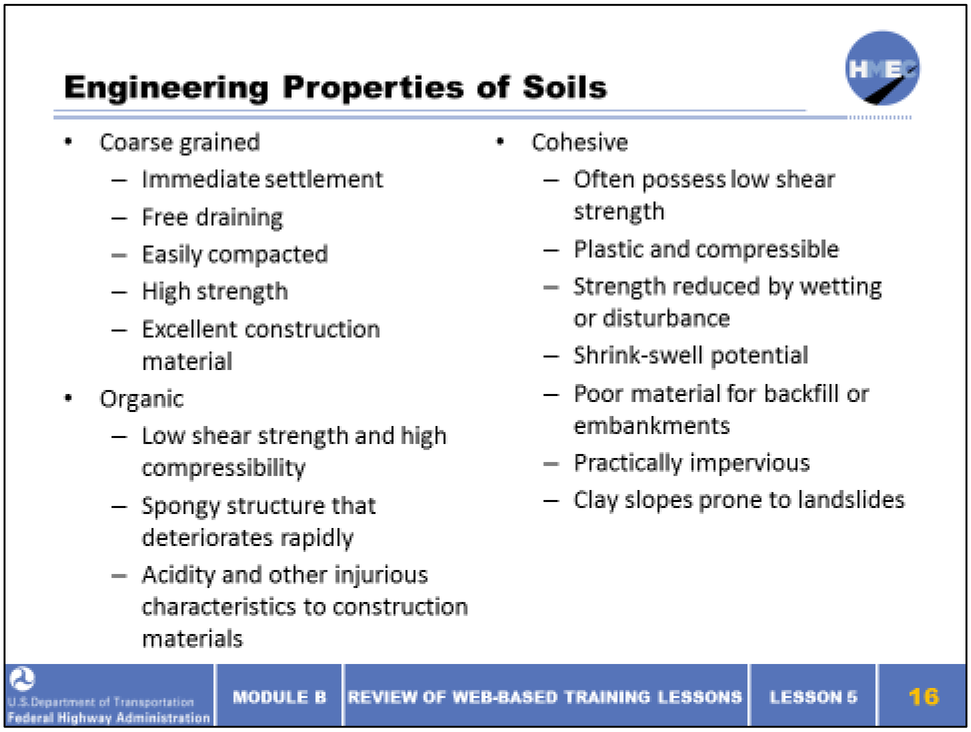
____

____

____

____

Slide 16



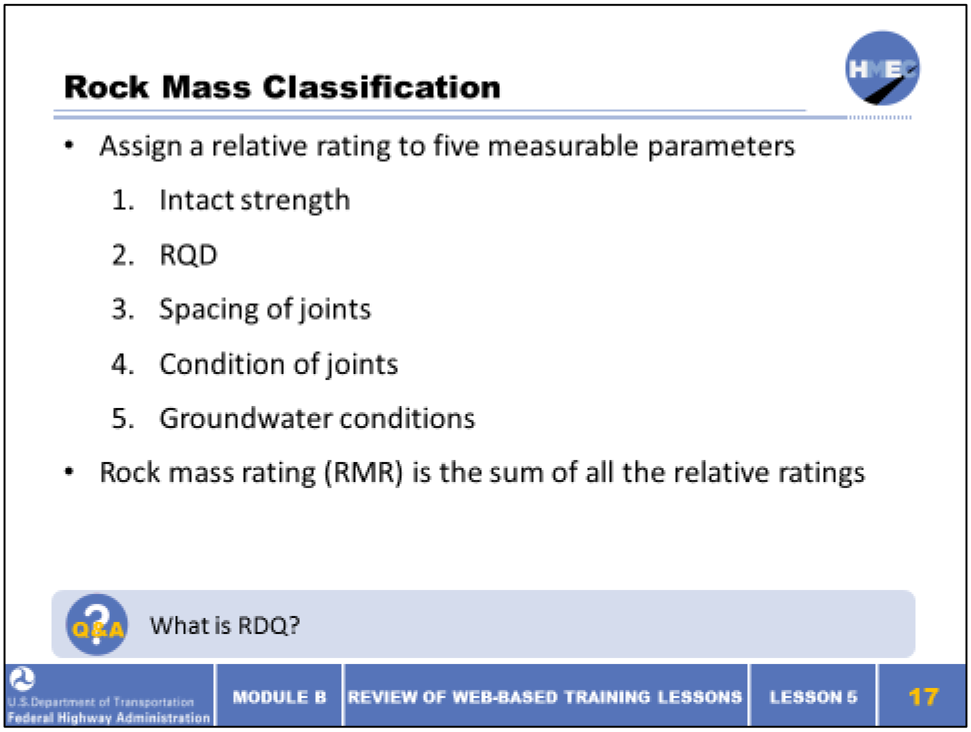
When it comes to construction, coarse-grained soils are good, cohesive soils are usually not so good, and organics are bad.

Coarse-grained or granular soils make excellent foundation material. They make very good embankment material as long as their erosion potential is properly addressed. When free draining, they make the best backfill material and are not frost susceptible. Remember that the friction angle (and therefore strength) of granular soils is strongly influenced by density. Proper compaction increases soil density, providing higher strength.

Cohesive soils (clays) have a wide range of engineering properties depending on how they were deposited and on their stress history. If clays are normally consolidated (that is, they have not been subjected to high loads in the past), they often possess low strength, are compressible, and their strength is reduced even more by wetting or disturbance. However, if they have been pre-consolidated, say by a glacier, they can be very strong and make a good foundation.

Organic soils are problematic. They have very low strength and are highly compressible. They have lots of plant matter, which will continue to decay and cause future problems, even if we deal with the immediate settlement and strength issues. Organic soils are trouble; get rid of them whenever possible.

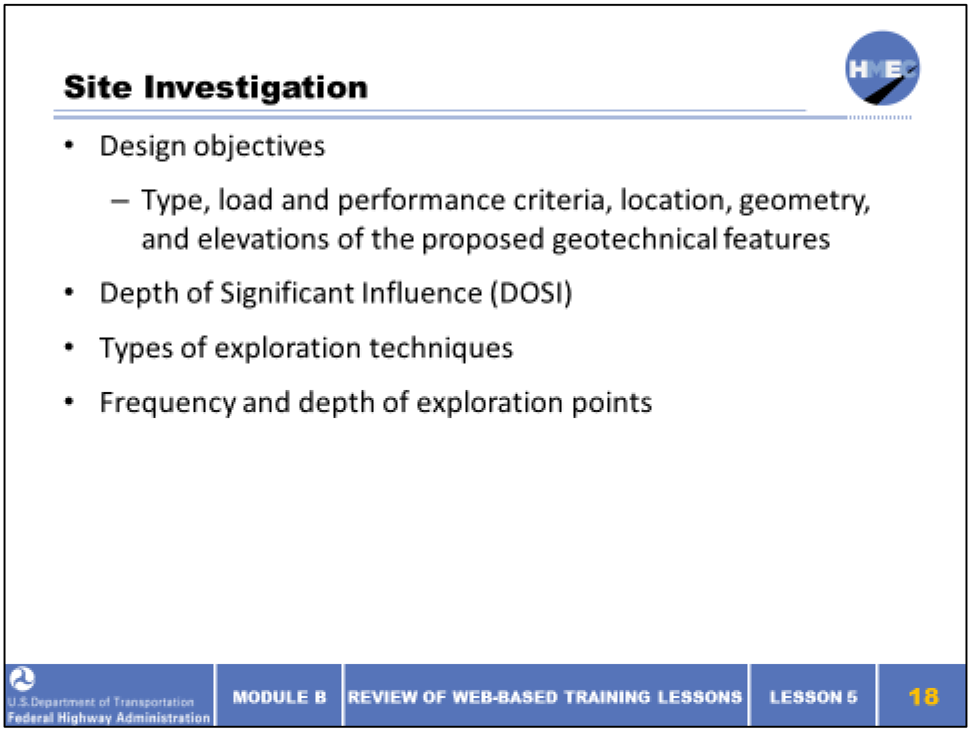
Slide 17



The performance of a rock mass is strongly influenced by its discontinuities or joints. The rock mass classification was developed to account for that influence. The strength of an intact sample (the rock core we take) is just one factor.

Because the rock mass controls behavior, the rock mass rating (RMR) was developed to estimate rock mass properties and characterize the overall quality of the rock mass. A relative rating is assigned to five measureable parameters including strength, RQD (rock quality designation), spacing of joints, condition of joints, and groundwater conditions. RMR is the sum of all the relative ratings. Various design methods have been derived around the RMR.

Slide 18

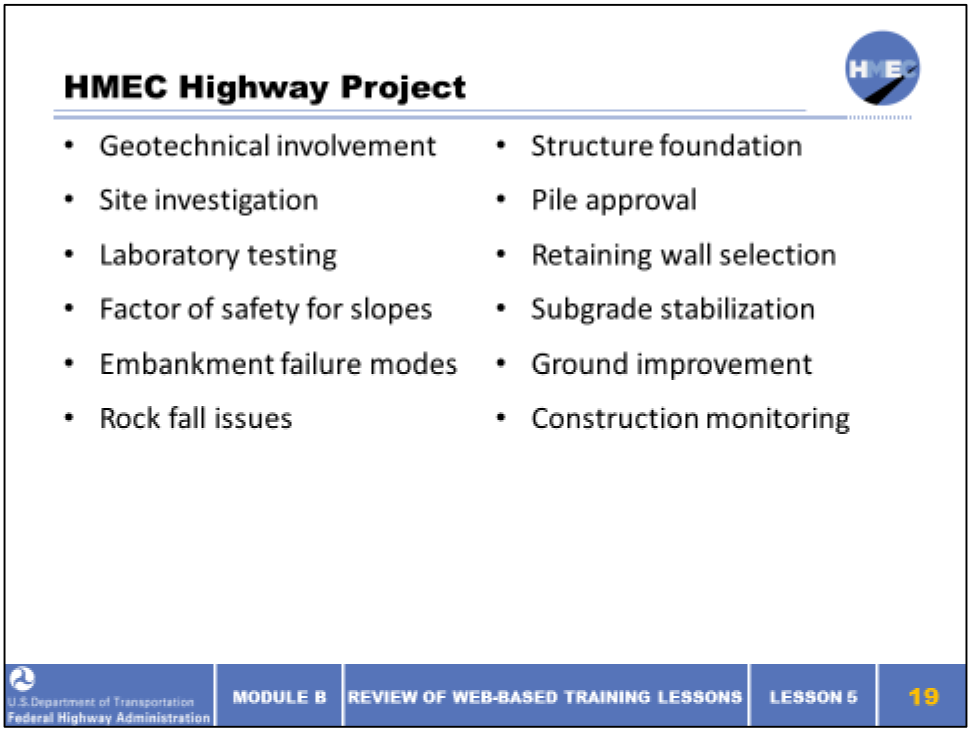


To develop a site investigation plan we need to consider several factors.

We need to consider the design objectives on our project as we develop a plan to investigate the project site. What are design objectives for the project? Does it involve new construction, rehabilitation, or maintenance? What are the loads, performance criteria, location, geometry, and elevations of the proposed geotechnical features? If we know the loads are small and some settlement is allowed, it can guide us in our investigation.

All geotechnical features influence soil behavior to significant depth (DOSI) and beyond the actual dimensions of the feature. This information can guide our investigation program regarding the location, depth, and frequency of borings and in-situ tests, as well as the type and locations of soil and rock samples.

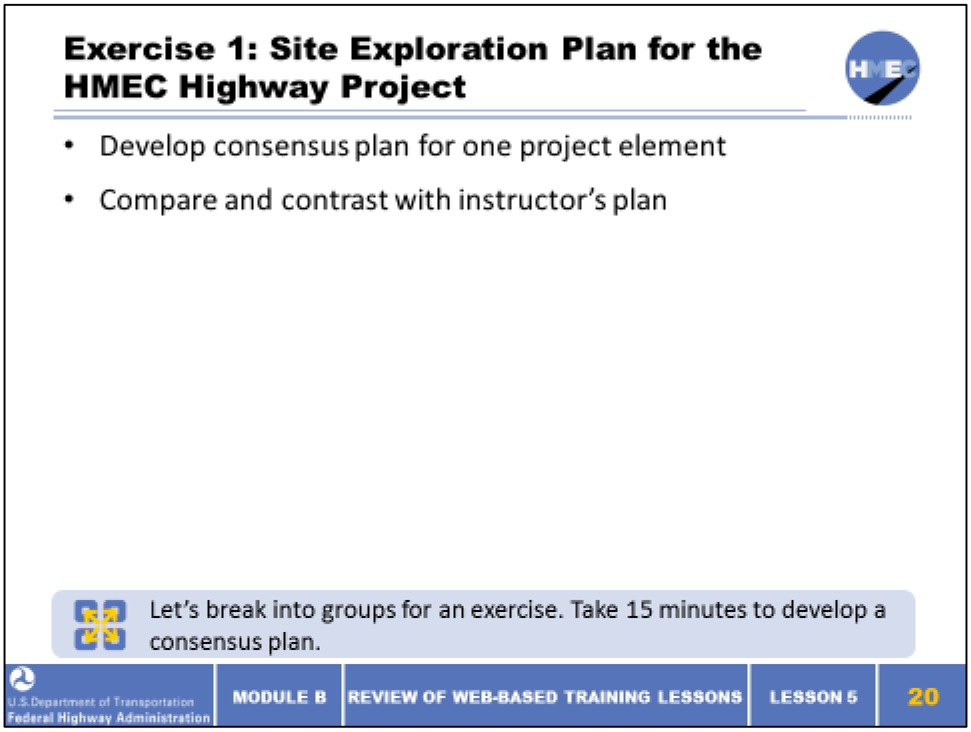
Slide 19



As you will recall from Lesson 4, we are going to use an example project to have you apply what you are learning to a realistic highway project.

This lesson will involve the site investigation portion of a project. Individually you were asked to develop a site exploration plan. We will discuss those results and compare them to our proposed solution.

Slide 20



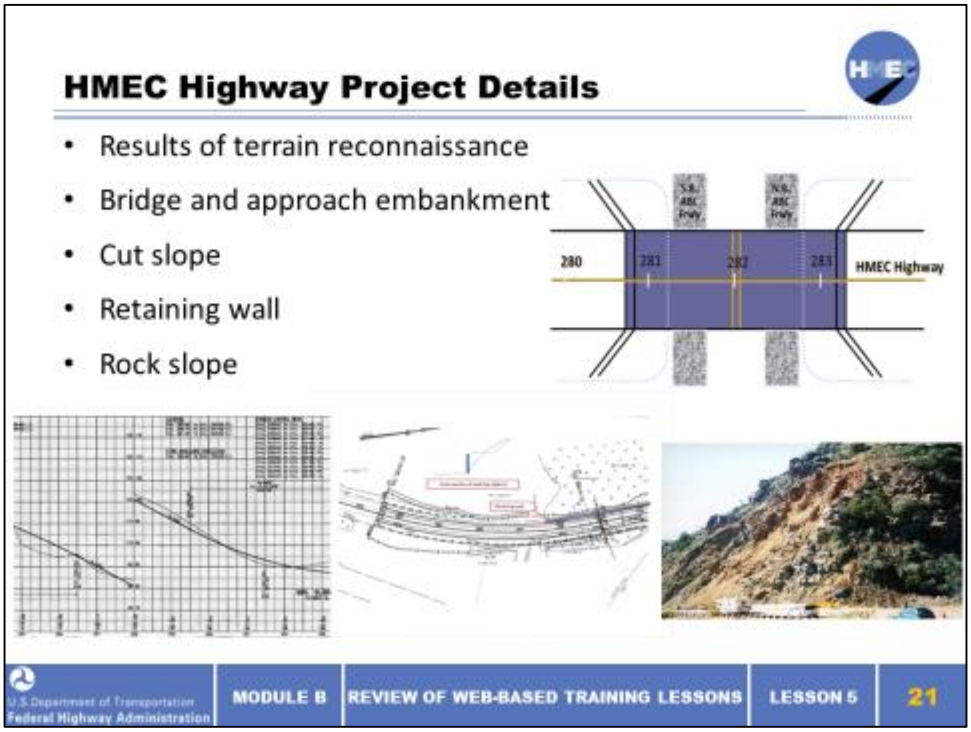
The HMEC Highway, the example project for the exercises in this manual, involves the reconstruction and upgrading of an existing county road in the upper Midwest US. The HMEC Highway project will allow you to apply what is discussed during the course to a realistic project. Although the project is fictitious, the components and issues it highlights could be encountered by you on one of your agency's projects.

You should have completed the offline exercise to develop a site exploration plan for the example project HMEC Highway. Within your group, discuss the various plans and develop a consensus exploration plan for one project element as assigned to your group. Summarize the plan on a sheet from the flip chart. Pick a representative who is willing compare and contrast your plan with the plan we will present later.

Consider the design objectives and DOSI for the geotechnical element your group is assigned. Include the exploration methods you would use, the number and location of exploration points for each method used, and the types and depths of sampling you would request. Describe how the plan may change if the investigation finds something unexpected. Also request any specialty investigation techniques or personnel that are required for the site exploration.

If you want further information, Chapter 3: Subsurface Exploration of the Soils and Foundations Reference Manual, FHWA-NHI-06-088 is an excellent reference. One of the references you have been given for this course is a pdf of the two-volume set of the Soils and Foundations Reference Manual, including the one just cited.

Slide 21



This is the information you were given to begin your site exploration plan.

You were given the results of the terrain reconnaissance. You were tasked with developing a site exploration for these four areas of the project. The bridge and approach embankment, the cut slope with subgrade, the retaining wall, and the rock slope. Full-page versions of these areas of the project can be found in your PW. The project description, the terrain reconnaissance and details of each of these four project elements are included below.

The HMEC Highway, the example project for the exercises in this manual, involves the reconstruction and upgrading of an existing county road in the upper Midwest US. The HMEC Highway project will allow you to apply what is discussed during the course to a realistic project. Although the project is fictitious, the components and issues it highlights could be encountered by you on one of your agency’s projects.

The terrain reconnaissance of the HMEC Highway project included an office review and a site inspection. Office review of available historical documents revealed that the wetlands near retaining wall at station 268 was an old lakebed. The possibility of buried soft clay deposits exist at that location. The site inspection and field reconnaissance documented that there are cattails growing near the east abutment of the bridge. Another area of potential weak soil deposits.

Overall access is easy for drilling and CPT rigs, except within the wetlands area (consider options to be used for access and sampling). A rock slope is exposed at the widening section.

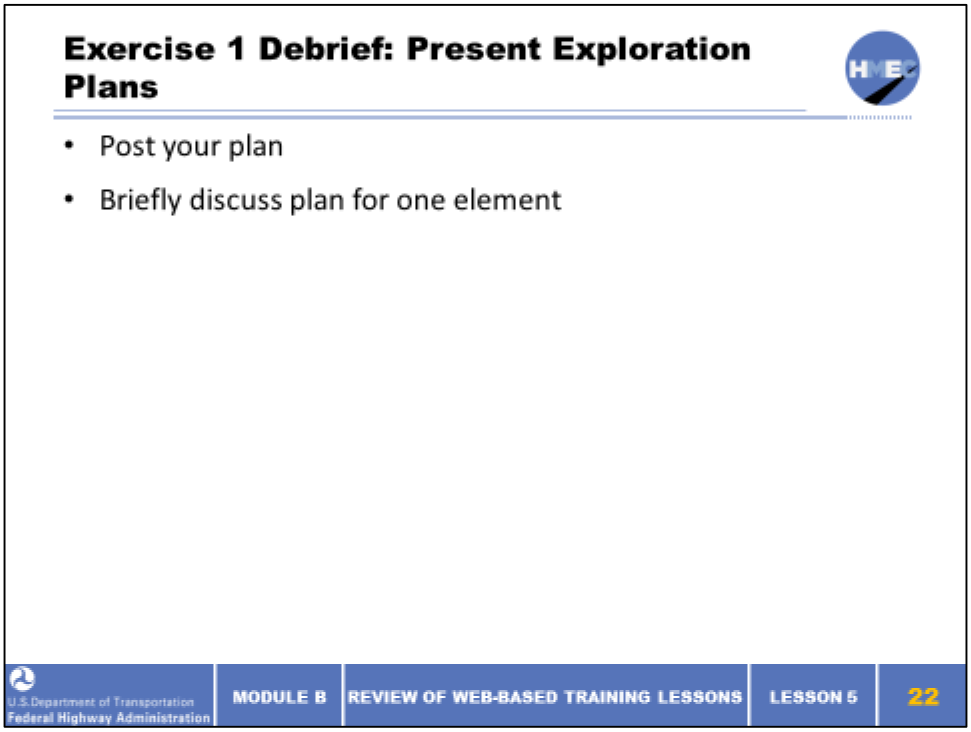
The bridge on the HMEC Highway is a two-span bridge over an existing freeway. The site investigation noted cattails growing near the east abutment, potentially soft and/or organic soils. The rock slope is at station 275, about 500 ft. west, therefore, it is unknown what type of foundations (deep or shallow) will be appropriate for this bridge. The approach embankments for the bridge are about 30 ft. high.

There will several cut slopes on the project. Only one area between station 257+50 and 260+75 will be evaluated for the exercises in this course. If suitable material is found, it is anticipated that the natural in-situ soils will be used as the pavement subgrade in cut sections.

A retaining wall with a maximum height of 25 ft. is required from station 267+70 to 270+50 to prevent a side hill embankment fill from encroaching into a wetlands area. Note the retaining wall is very close to the edge of the wetlands area.

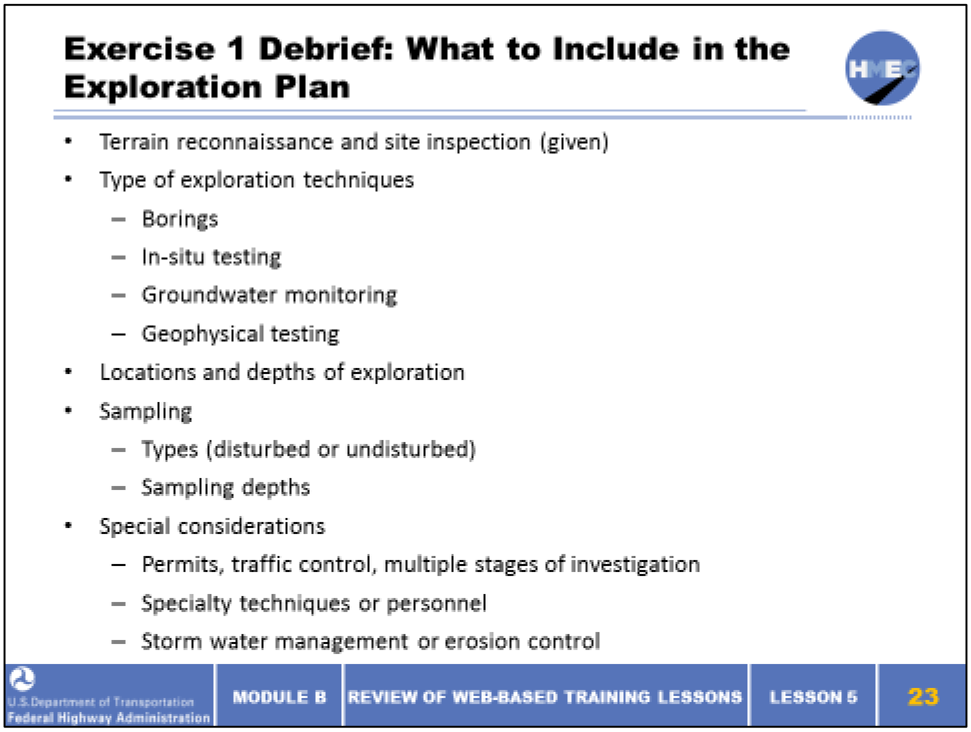
The highway will be widened into the rock slope. This section of the HMEC Highway is being widened on existing alignment. The roadway widening will be 20 ft. into the slope over a length of 500 ft. It is anticipated that the new rock slope will parallel the existing slope.

Slide 22



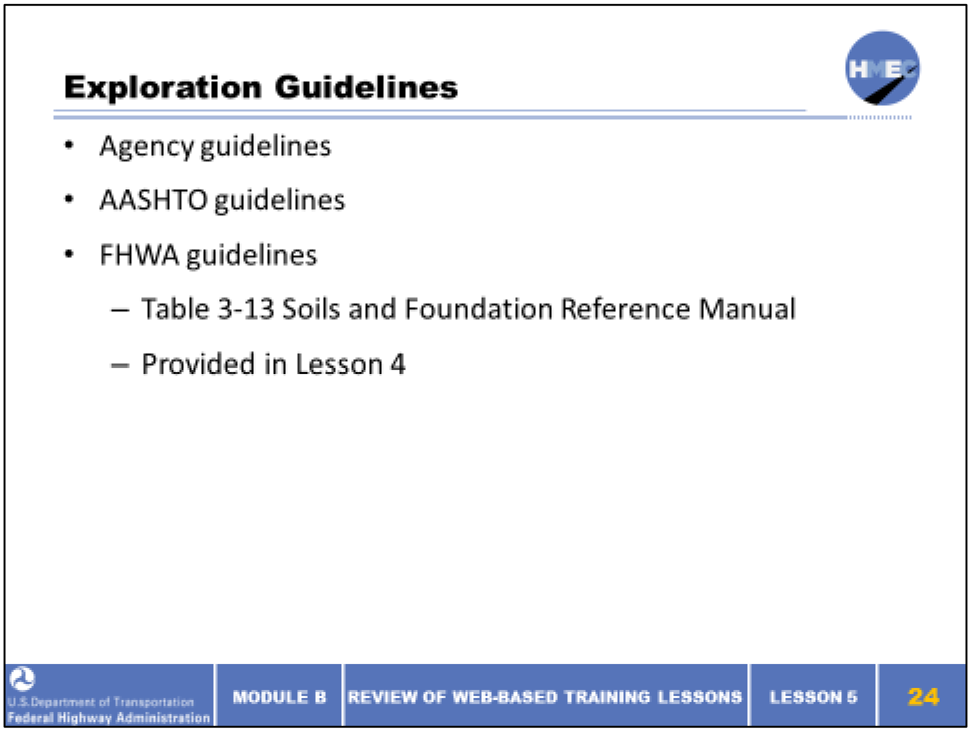
Have your group representative post your consensus exploration plan on the wall.

Slide 23



Now that we have had a look at your plans, let's discuss what we believe should be included.

Slide 24

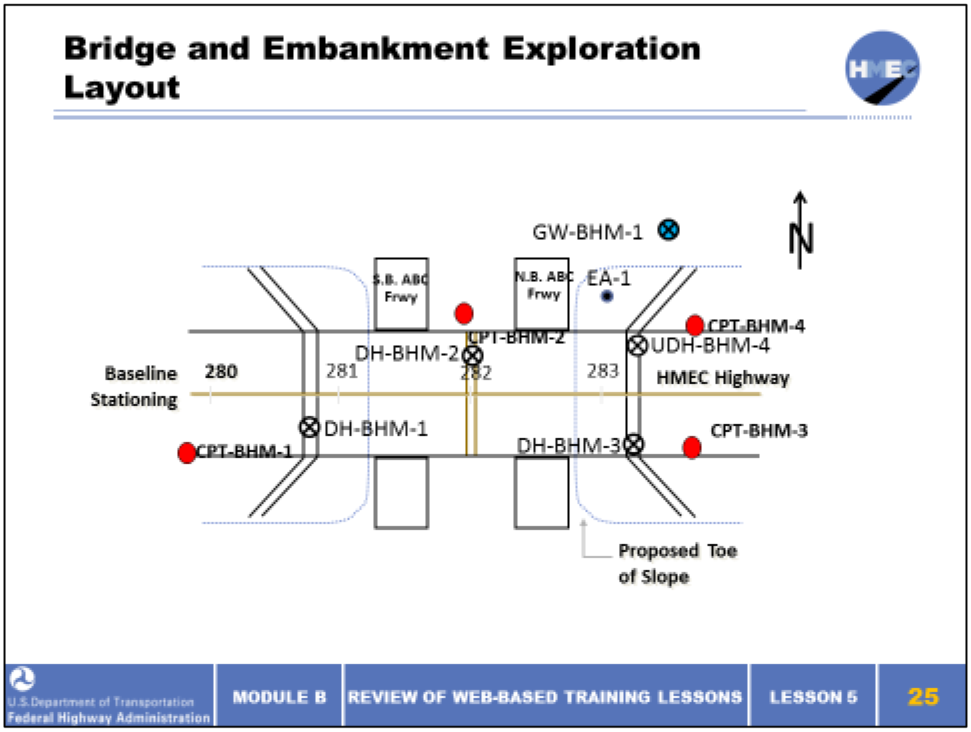


These are some guidelines that may be used to determine minimum sampling and testing during the site exploration.

Your agency may have its own guidelines. If not, they may use guidelines from AASHTO or FHWA. The guidelines referenced here apply to minimum explorations points, depths, and sampling. These are guidelines and minimums—you must decide whether or not these are sufficient for a particular project and geologic conditions.

There are also standards, typically AASHTO or ASTM, (for example standards for SPT, Shelby, CPT, etc.), which govern the procedures to be used when performing various exploration techniques and sampling. The standard procedures should always be followed; improvisation of investigative techniques may result in erroneous or misleading results, which may have serious consequences on the interpretation of the field data.

Slide 25



The Bridge you were ask to plan an exploration for is a two-lane, two-span Bridge over a freeway. The approach embankment is 30 ft. high.

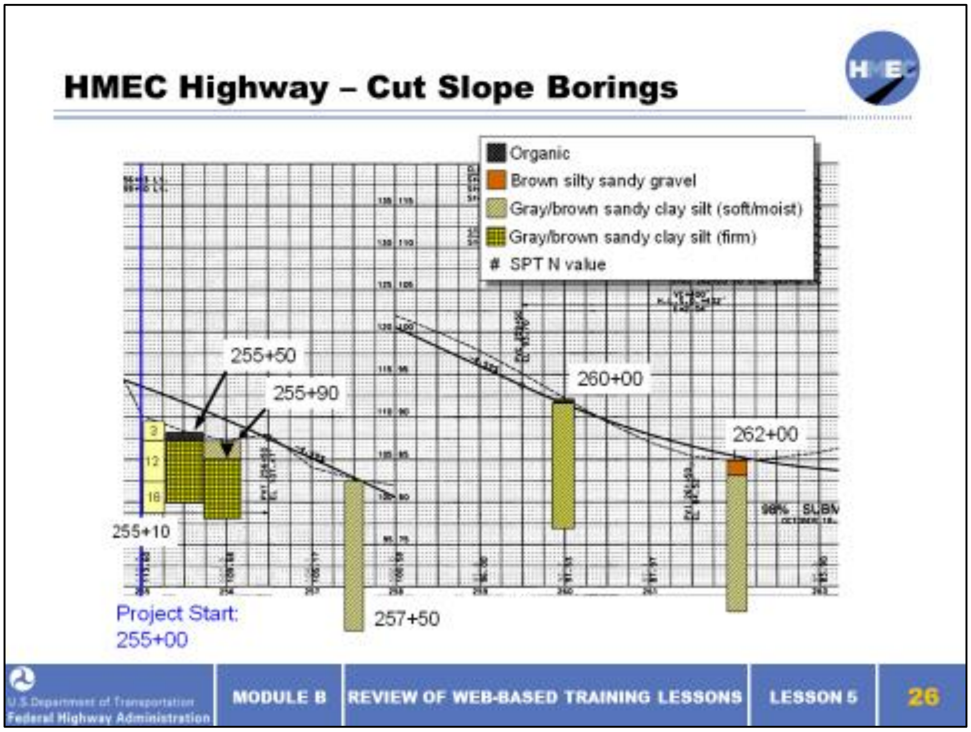
Recall that the site inspection identified an area of wet, soft ground near the east abutment. This exploration requires close coordination with the geotechnical specialist to avoid multiple trips to the field.

A bridge of this width requires at least one boring per substructure unit. Since there was soft ground under a 30-ft. high embankment at the east abutment, an additional boring along with several CPT soundings were conducted. At least one boring at the east abutment/embankment requires undisturbed sampling. We also recommended vane shear test at those borings to obtain in-situ shear strengths.

The horizontal extent of the soft ground is most easily obtained by using a hand auger at several locations. The depth of the borings will probably be controlled by the bridge exploration requirements, which is 20 ft. beyond the expected piling length or 10 ft. into rock.

The depth of borings for an embankment is two times the height of the embankment, or into a hard layer of sound foundation material. In either case, if the soft layer continues to a greater depth, the borings should be taken deeper. One groundwater monitoring well was recommended due to the potential for embankment stability issues.

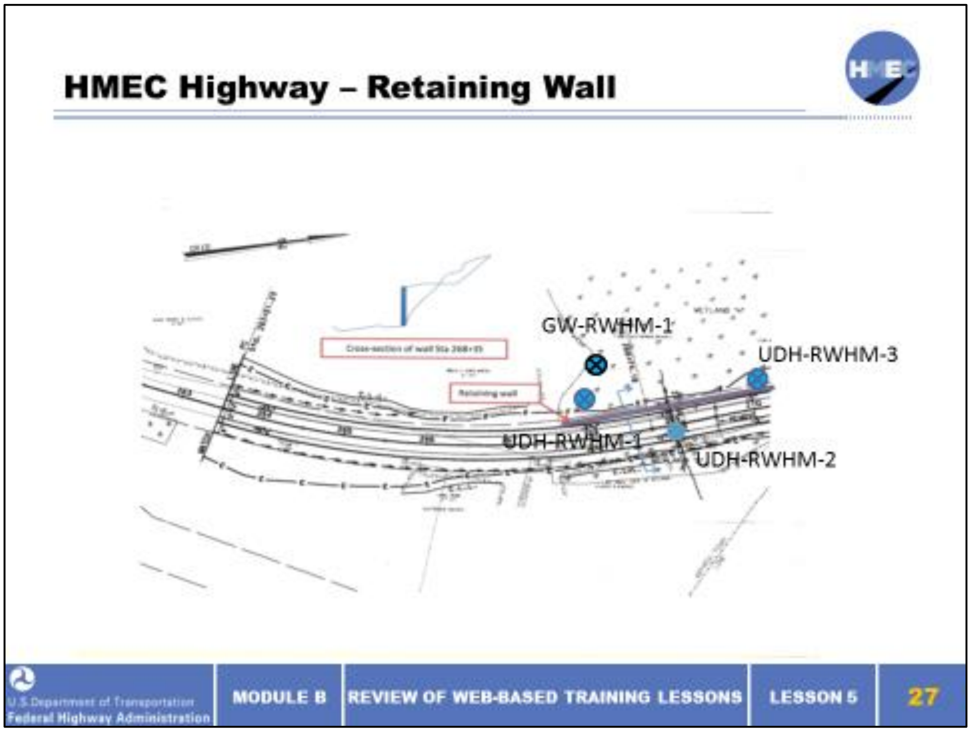
Slide 26



The cut slope we asked you to investigate on the project was about 300 ft. long; we are only interested in the cut section between 257+50 and 260+75.

Since the cut slope is only 300 ft. long and the maximum depth of the cut is less than 5 ft., only minimal exploration is required. The maximum spacing is 200 ft., therefore, for a 300-ft. cut slope we recommended at least two borings. The depth of the boring should be at least 15 ft. below the ditch line elevation. SPT blow counts and disturbed samples should be obtained at 5-ft. intervals.

Slide 27



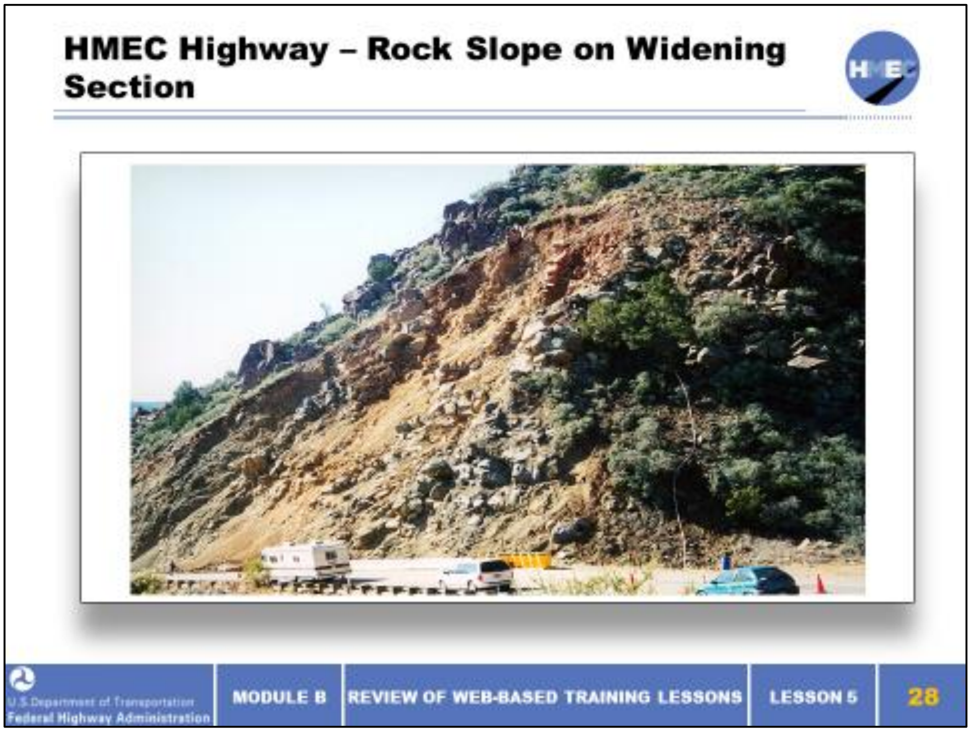
The retaining wall we asked you to investigate was on a side hill fill adjacent to a wetland between station 267+7- and 270+00.

Recall that the wetland was determined to be an old lakebed with soft clay deposits. The maximum spacing for exploration points on a retaining wall over 100 ft. long is 100 to 200 ft. Since this wall is adjacent to the wetlands, a spacing of 100 ft. was chosen. The exploration points were alternated from the front to the back of the wall. Undisturbed Shelby tube samples will be obtained at 5-ft. intervals for each clay layer. Vane shear tests will also be performed at various depths in the bottom of the borehole to obtain in-situ shear strengths. The wall is 25 ft. high, therefore the borings will continue to a depth of 50 ft.—two times the height or 10 ft. into rock, whichever is reached first. If the soft clays continue to a depth greater than 50 ft., the geotechnical engineer should be contacted. A groundwater monitoring well was recommended due to the potential for stability issues and dewatering issues during construction of the wall.

____

____

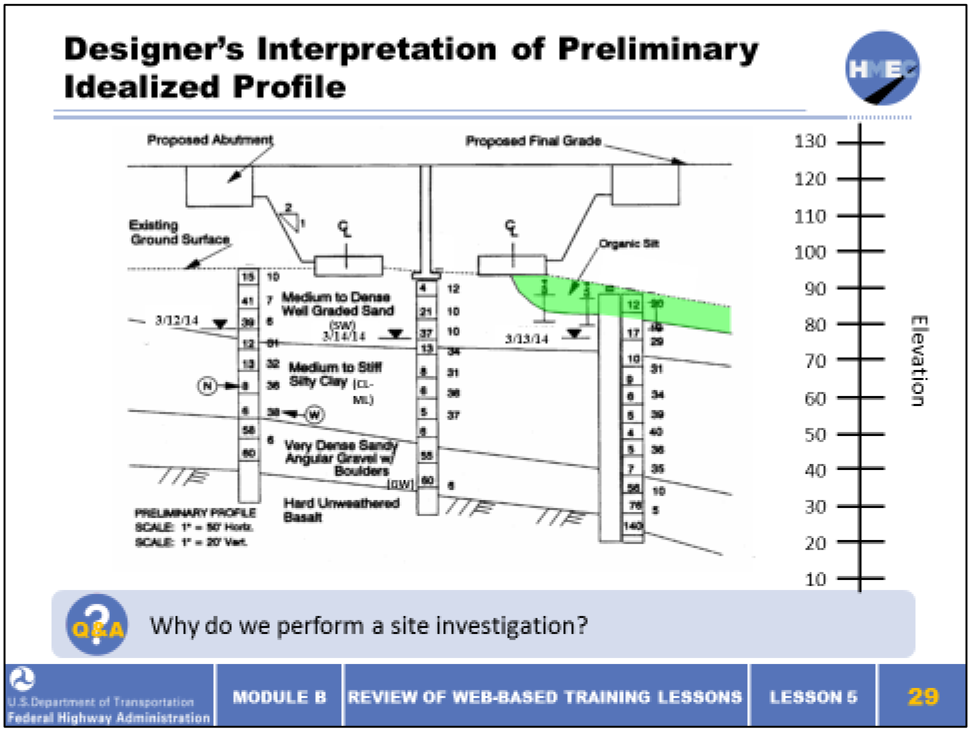
Slide 28



The rock cut slope you were asked to investigate is 500 ft. long in a section being widened along the existing alignment.

On the surface, this rock slopes appears to be a combination of weathered rock and soil, which may be unstable. It is important to involve a geotechnical or geologist specialist to inspect and map this rock slope and determine the additional exploration that is required. That may include borings and/or geophysical testing. Since this is an existing rock slope, previous project construction records and maintenance records will provide valuable information. Borings may be required on the slope to determine if the weathered rock is a surficial deposit or if it continues to a greater depth. If needed, the borings would need to extend beyond the planned excavation depth of 20 ft.

Slide 29

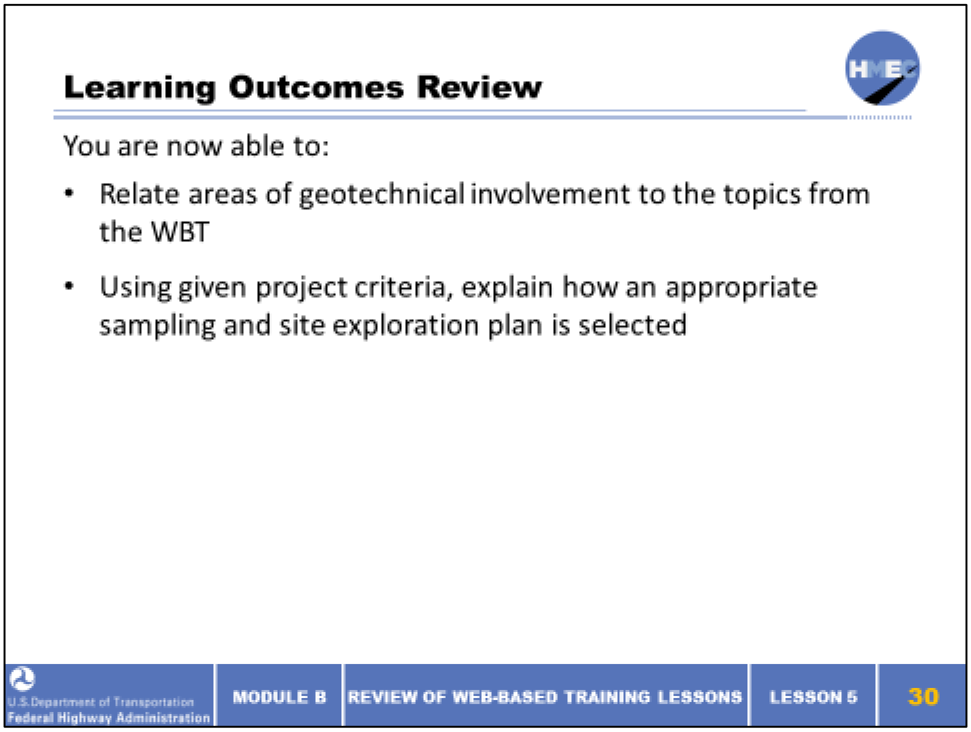


When the site investigation is completed, we will use all of the information obtained by the various exploration techniques to generate a subsurface profile for the project.

This is an example of the subsurface profile generated from our proposed exploration plan for the bridge/embankment site.

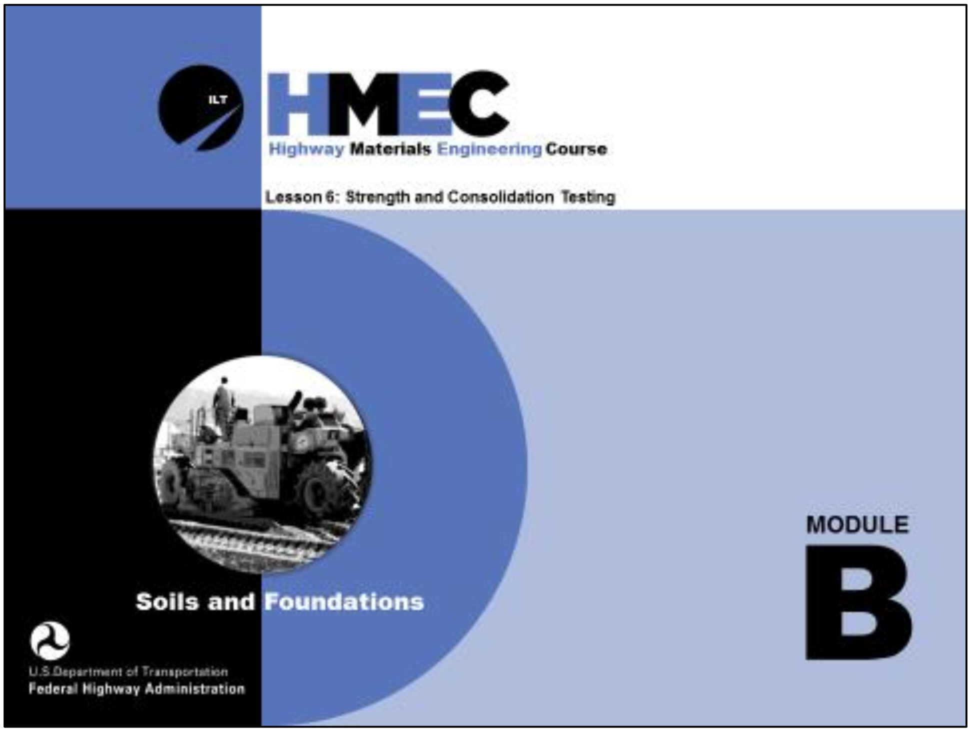
Why does moisture drop from 38 to 6? Depth beneath water table doesn't matter. What is the material and density that will control moisture content?

Slide 30

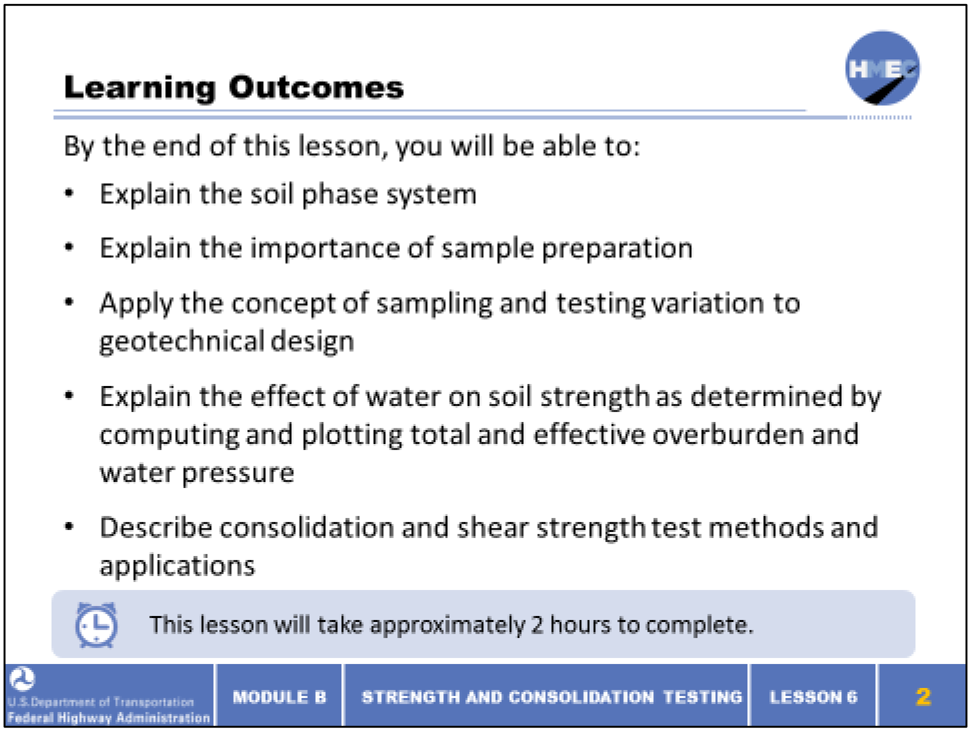

Learning Outcomes Review
You are now able to:
• Relate areas of geotechnical involvement to the topics from the WBT
• Using given project criteria, explain how an appropriate sampling and site exploration plan is selected
U.S. Department of Transportation
Federal Highway Administration
MODULE B
REVIEW OF WEB-BASED TRAINING LESSONS
LESSON 5
30

This page is intentionally blank.

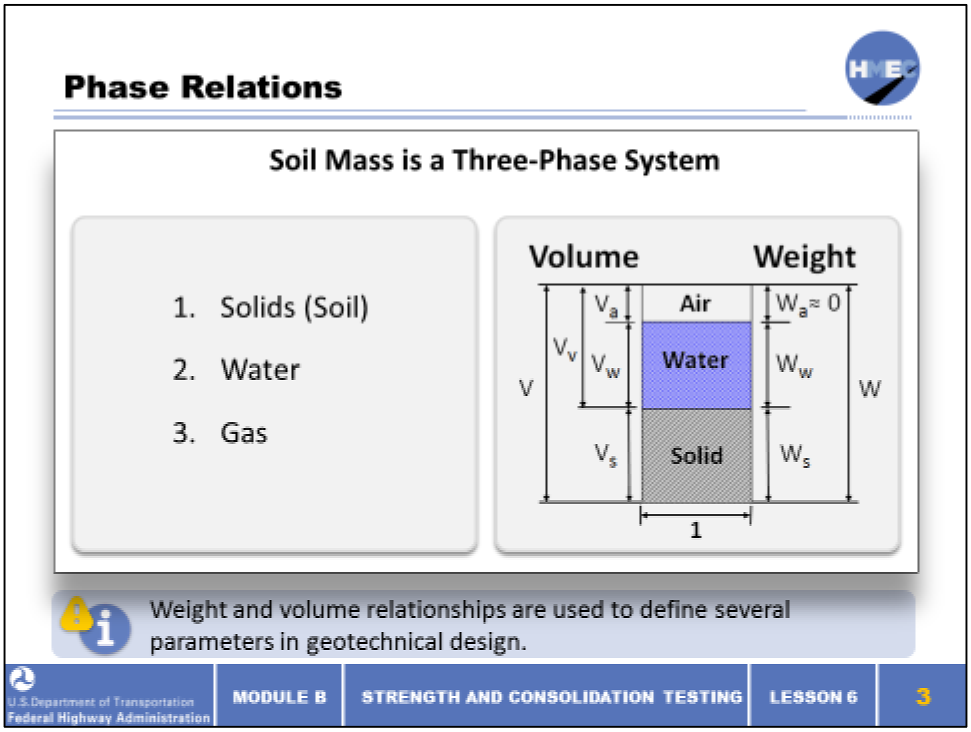
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 6: Strength and Consolidation Testing
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Explain the soil phase system
• Explain the importance of sample preparation
• Apply the concept of sampling and testing variation to geotechnical design
• Explain the effect of water on soil strength as determined by computing and plotting total and effective overburden and water pressure
• Describe consolidation and shear strength test methods and applications
This lesson will take approximately 2 hours to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
2

Slide 3



Define terms, volume of solids, and volume of water. Soil behavior is controlled by the interaction of these three phases. Due to the three-phase composition of soils, complex states of stresses and strains may exist in a soil mass. Proper quantification of these states of stress, and their corresponding strains, is a key factor in the design and construction of transportation facilities. The soil is simplistically rearranged in terms of air, water, and solid particles in an attempt to quantify their volumes and weights.

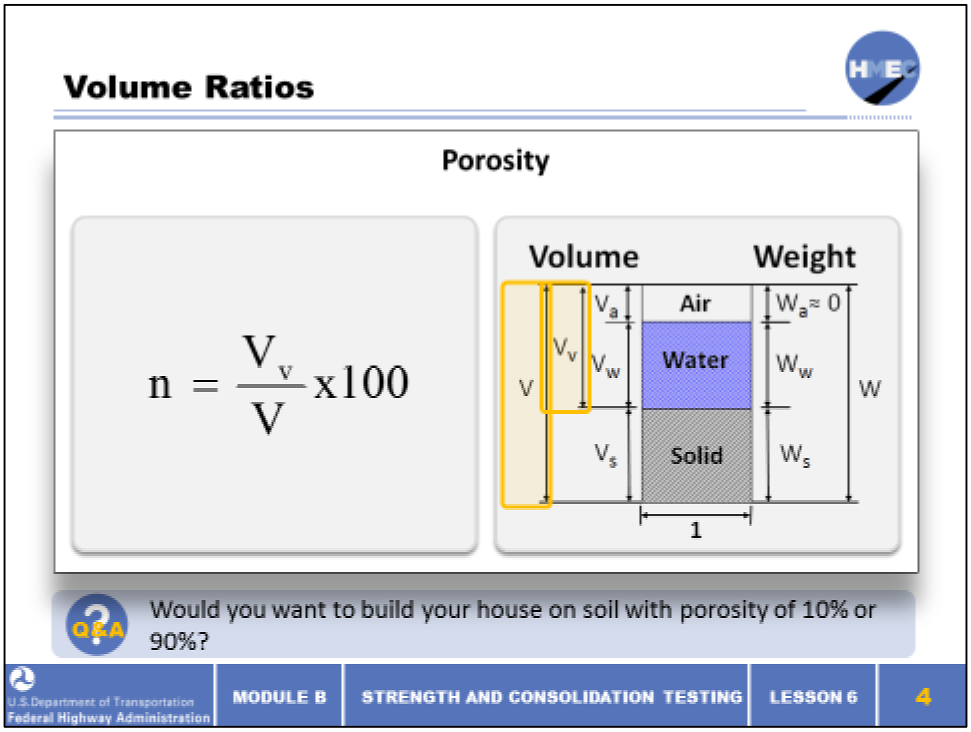
We use the weight and volume relationships to define several parameters that that are used in geotechnical design.

The first step in quantification of the stresses and strains in soils is to characterize the distribution of the three phases of the soil mass and determine their inter-relationships. The inter-relationships of the weights and volumes of the different phases are important since they not only help define the physical make-up of a soil but also help determine the in-situ geostatic stresses, i.e., the states of stress in the soil mass due only to the soils self-weight.

____

____

Slide 4



One of those parameters is a measure of how much of the soil mass is void space.

A parameter used to express the volume of the voids in a given soil mass can be obtained from the ratio of the volume of voids, V_v, to the total volume, V. This ratio is referred to as porosity, n, and is expressed as a percentage as shown on the slide. As a soil mass is compressed, the volume of voids, V_v, and the total volume, V, decrease. Thus, the value of the porosity changes. Since both the numerator and denominator in the equation change at the same time, it is difficult to quantify soil compression, e.g., settlement or consolidation, as a function of porosity.

____

____

____

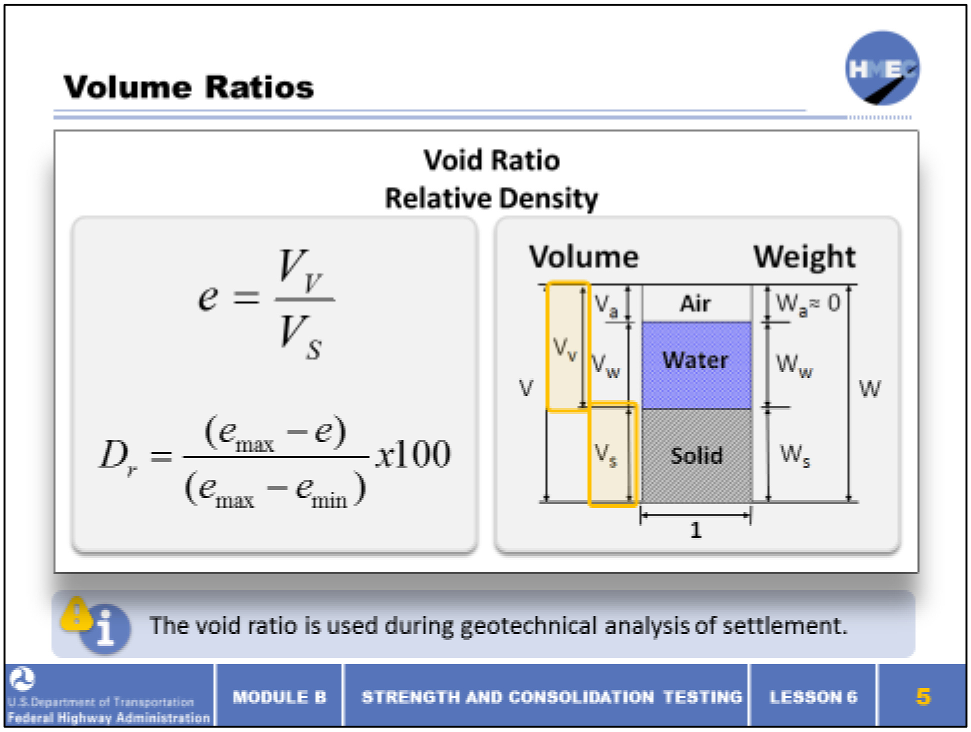
____

____

____

____

Slide 5



In soil mechanics, the volume of voids, V_v (V_a plus V_w) is expressed in relation to a quantity, such as the volume of solids, V_s, that remains unchanging during consolidation or compression. This is done by the introduction of a quantity known as void ratio, e, which is expressed in decimal form as shown by the equation for e. Unlike the porosity, the void ratio can have values greater than 1. That would mean that the soil has more void volume than solids volume, which would suggest that the soil is “loose” or “soft.” Therefore, in general, the smaller the value of the void ratio, the denser the soil. As a practicality, for a given type of coarse-grained soil, such as sand, there is a minimum and maximum void ratio. These values can be used to evaluate the relative density, D_r (%), of that soil at any intermediate void ratio as shown by the equation for relative density.

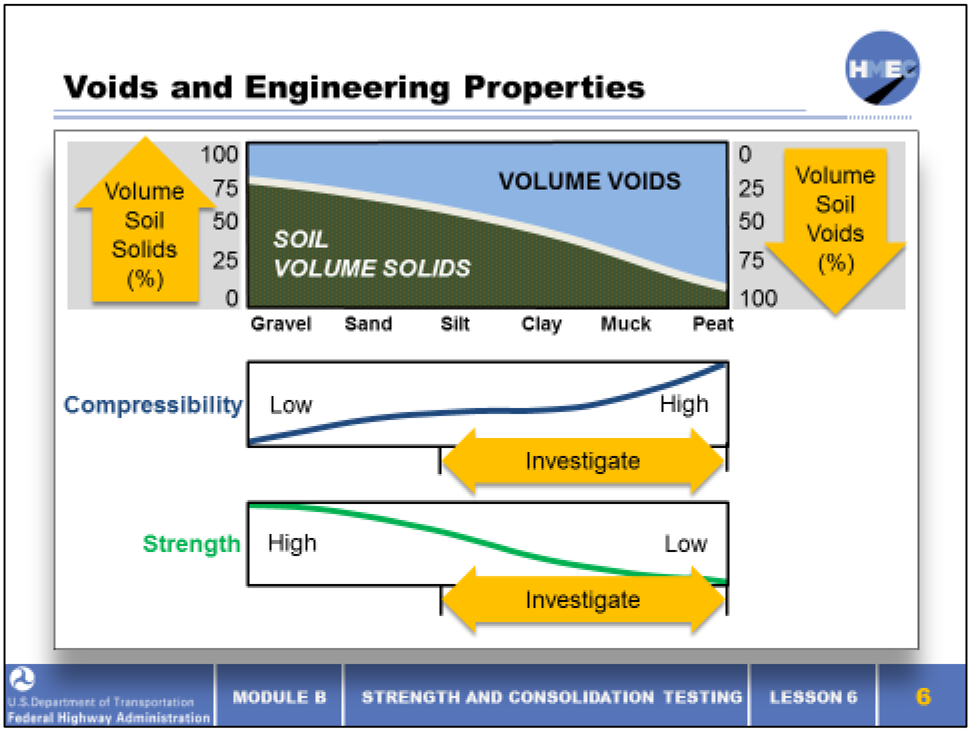
The void ratio is used during geotechnical analysis of settlement. Relative density is used to estimate the friction angle of granular soils.

___

___

___

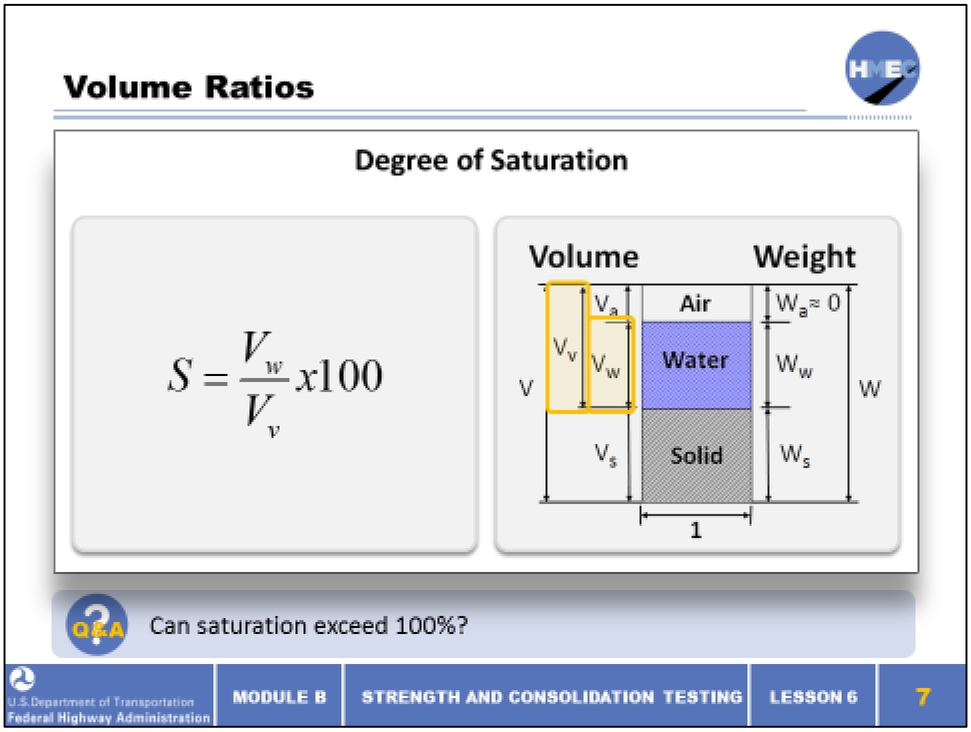
Slide 6



In the graphic, as the voids increase, the strength goes down and the compressibility goes up. The horizontal scale shows various material types, including gravel, sand, silt, clay, muck, and peat.

____

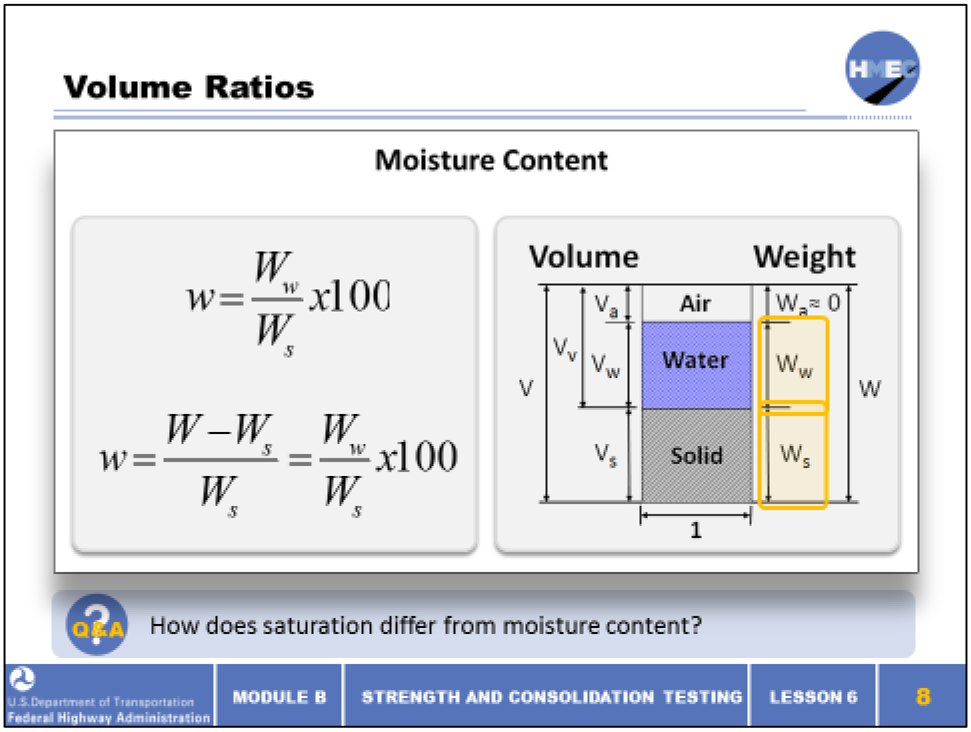
Slide 7



In the case of a saturated soil, all the voids (i.e., soil pore spaces) are filled with water, $V_v = V_w$. While this condition is true for many soils below the groundwater table or below standing bodies of water such as rivers, lakes, or oceans (and for some fine-grained soils above the groundwater table due to capillary action), the condition of most soils above the groundwater table is better represented by consideration of all three phases where voids are occupied by both air and water. To express the amount of void space occupied by water as a percentage of the total volume of voids, the term degree of saturation, S, is used as shown by the equation for S.

Note that saturation is a volume relationship and differs from moisture content, which is a weight relationship.

Slide 8



The moisture content test is one of the simplest and least expensive laboratory tests to run. Moisture content is determined based on the ratio of the weight of water to the weight of dry soil. To determine the moisture content, the soil must be dried, typically using an oven in the laboratory. If moisture content must be determined in the field, techniques such as speedy moisture, field drying, and resistance between electrical probes, are used.

____

____

____

____

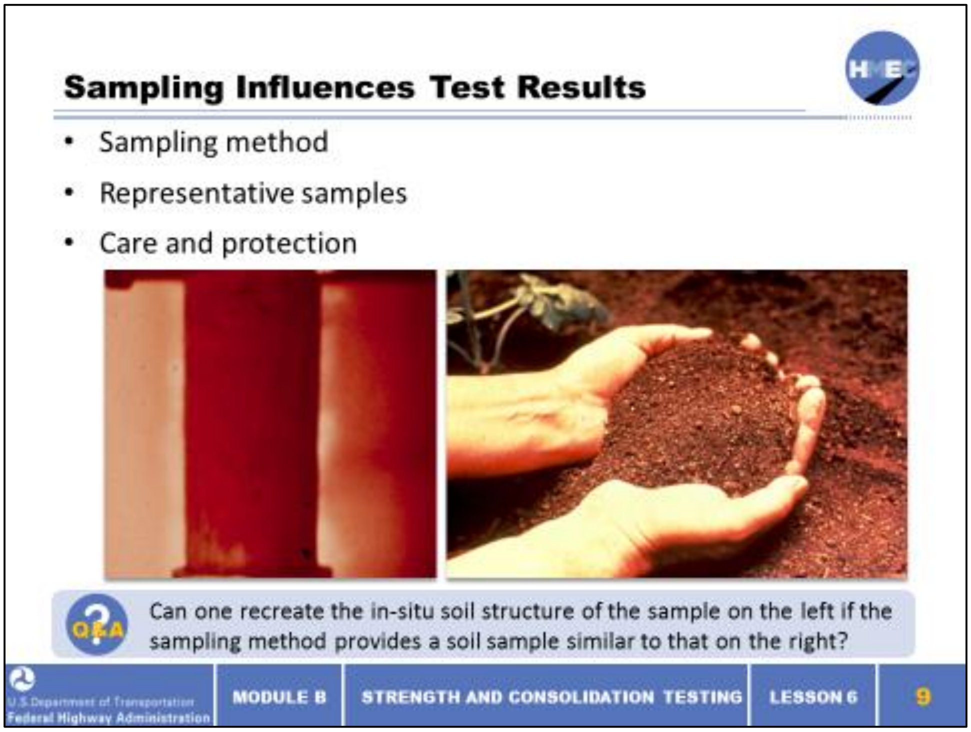
____

____

____

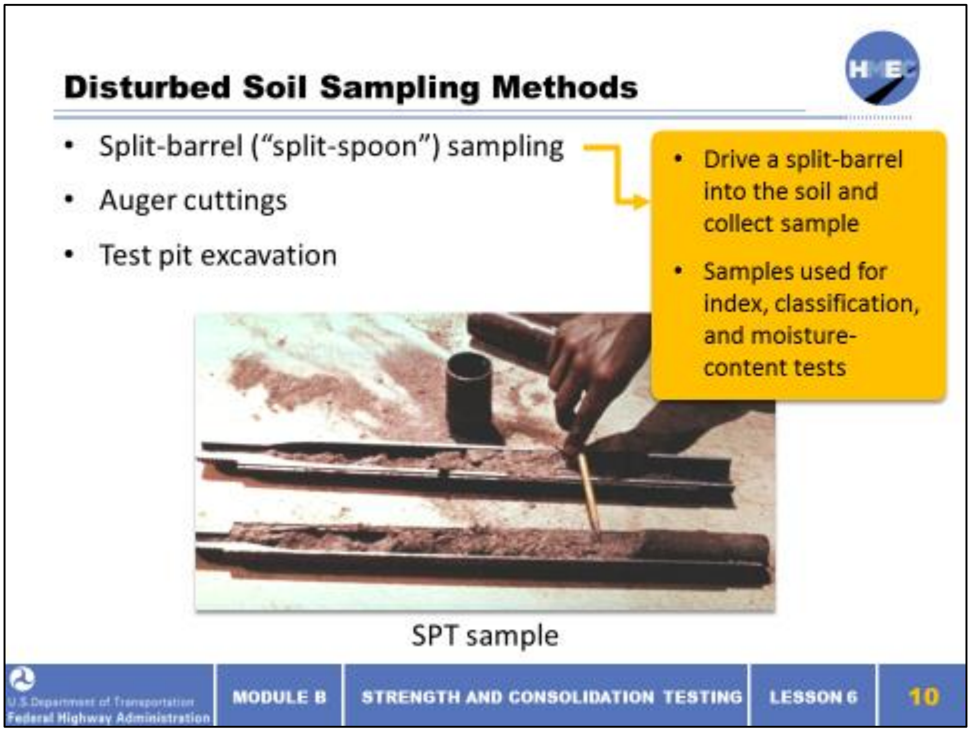
____

Slide 9



Laboratory testing of soil samples recovered during subsurface explorations is the most common technique to obtain engineering properties necessary for design. A laboratory-testing program consists of index tests to obtain general information on material consistency, (i.e., to classify the soil), and performance tests to measure specific properties (e.g., shear strength, compressibility, hydraulic conductivity) for design and constructability assessments.

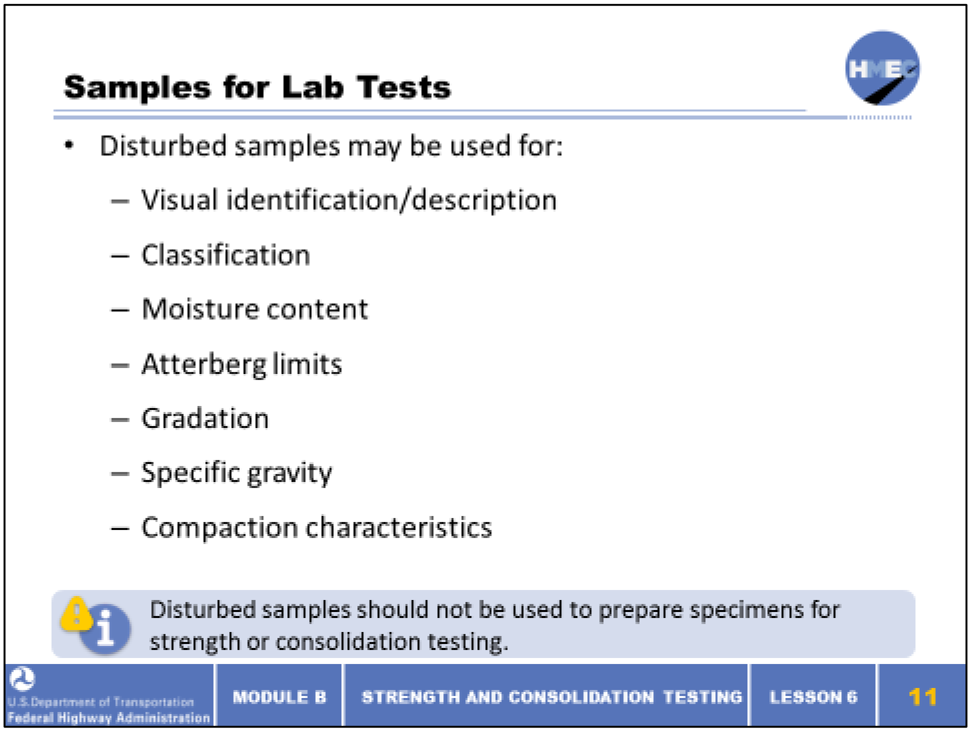
Slide 10



The most common soil samples are those obtained from split-barrel sampling, auger cuttings, and/or test pits. These are referred to as disturbed samples because the soil matrix or structure has been completely destroyed in the case of auger cuttings and excavations, or severely altered in the case of standard penetration test (SPT) samples.

Even though these samples are disturbed, they are still valuable and the tests we perform on them provide useful results. Therefore, it is critical that the samples be representative and handled properly.

Slide 11



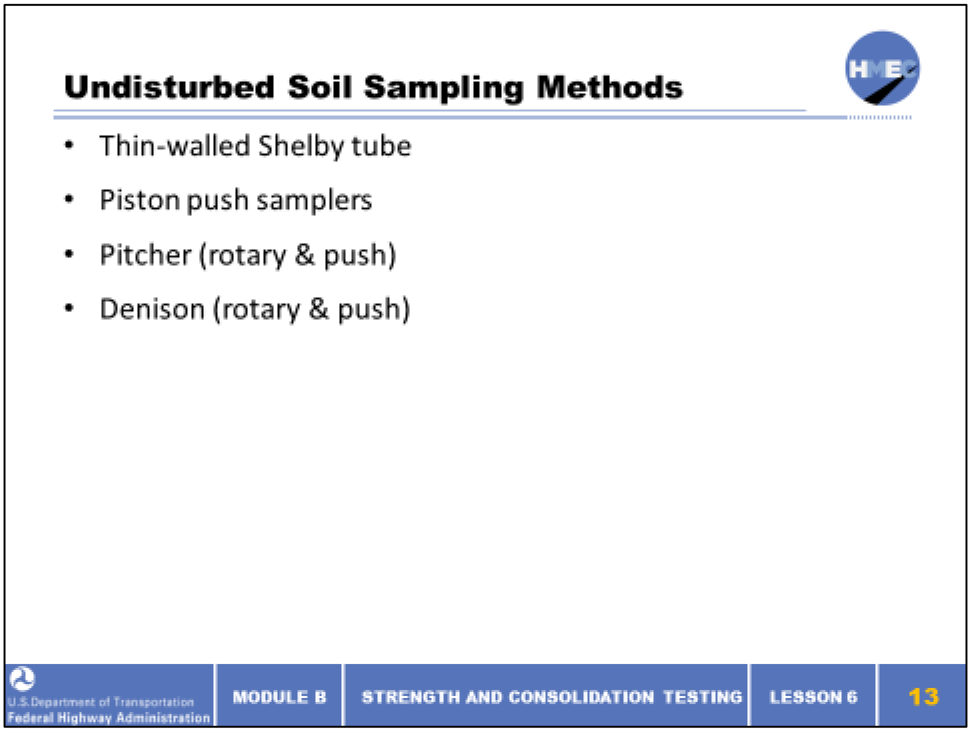
You can actually learn quite a lot from testing disturbed samples. All of these tests—classification, moisture content, specific gravity, Atterberg limits, gradation, and compaction characteristics—will tell you a lot about the soil behavior under load and whether it is suitable for the design or construction application you have in mind. Disturbed samples should not be used to prepare specimens for strength or consolidation testing.

However, it is critical that even though these samples are disturbed that they be properly handled and tested.

Slide 12

The sample must be from a single strata and its location must be properly identified on the sample container or the test results will be meaningless at best and misleading at worst. If one of the tests will be moisture content, the sample must be placed in a water-tight container. If the sample can potentially degrade when in contact with air, it must be placed in an air-tight container. This is a problem with some shales. If the sample is taken from cuttings or excavations, it is important to obtain sample that has not been segregated by the sampling process.

Slide 13

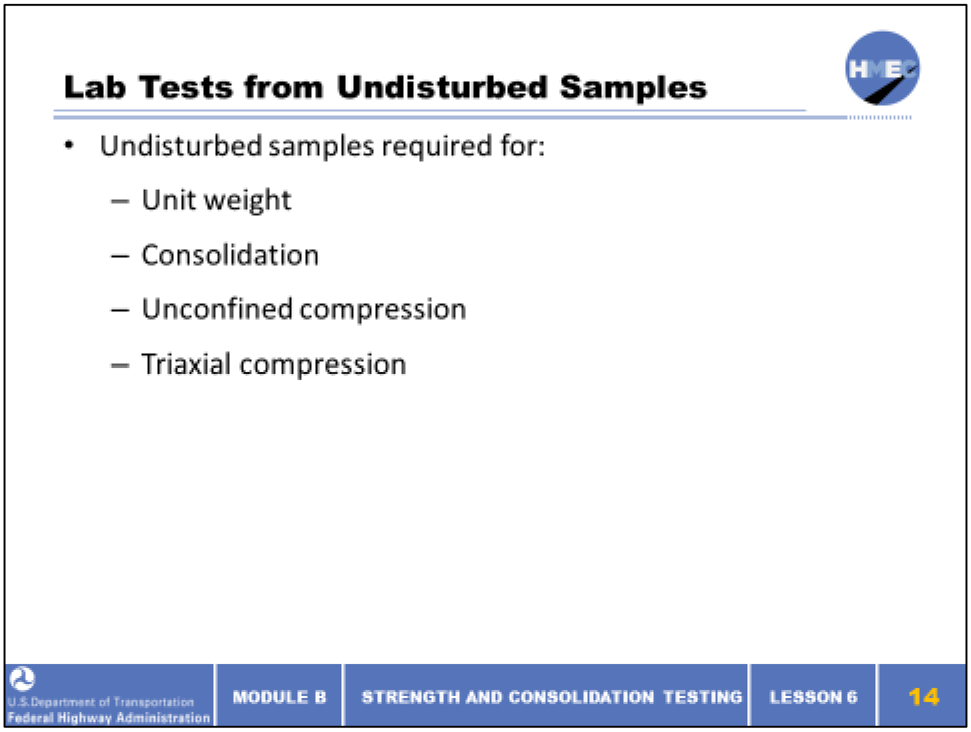


Disturbed samples are useful for index tests; however, to conduct performance tests, such as consolidation and strength testing, we must have undisturbed samples.

There are four methods for obtaining undisturbed samples. Due to cost and ease of use, the thin-walled Shelby tube is the most commonly used sampler for obtaining relatively undisturbed samples of soft to stiff fine-grained soils. Don't spend much time describing these methods. This section is about the importance of undisturbed samples, not the method.

• Shelby tube: Simplest device for undisturbed samples; boring should be clean before sampler is lowered; little waste area in sampler; not suitable for hard, dense or gravelly soils.
• Piston push sampler: Piston at end of sampler prevents entry of fluid and contaminating material requires heavy drill rig with hydraulic drill head; samples generally less disturbed compared with Shelby tube; not suitable for hard, dense, or gravelly soil.
• Pitcher: Differs from Denison in that inner tube projection is spring controlled; often ineffective in cohesion-less soils.
• Denison: Inner tube face projects beyond outer tube, which rotates; amount of projection can be adjusted; generally takes good samples; not suitable for loose sands and soft clays.

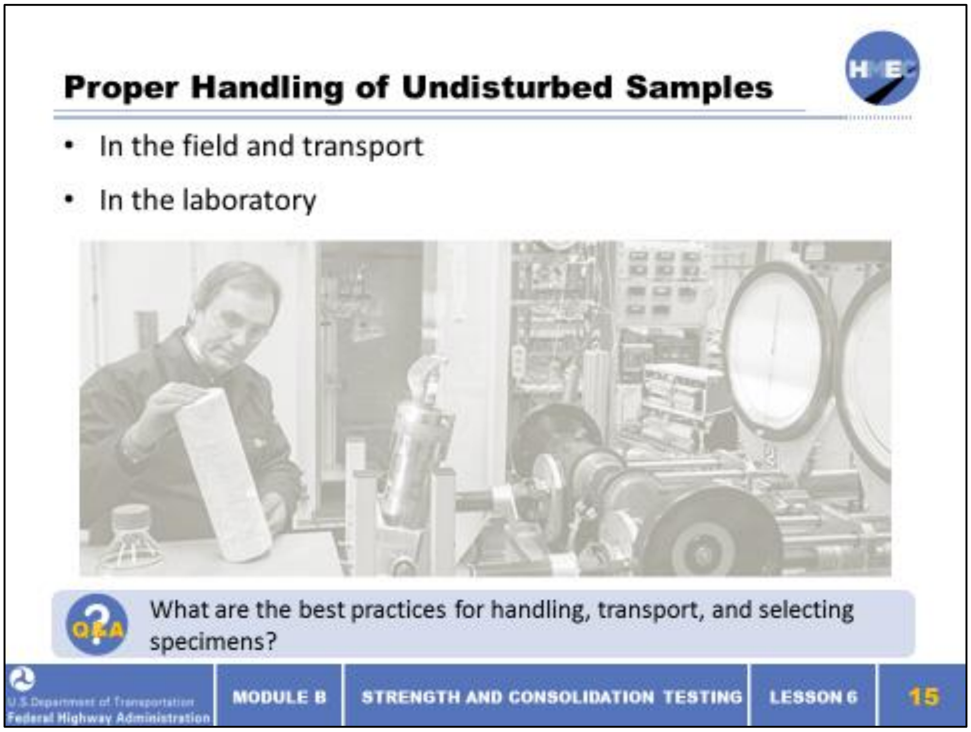
Slide 14



Undisturbed soil samples are required for performing laboratory strength and consolidation tests on cohesive soils having consistencies ranging from soft to stiff. High-quality samples for such tests are particularly important for approach embankments and for structural foundations and wall systems that may stress the compressible strata. In reality, it is impossible to retrieve truly undisturbed samples since changes in the state of stress in the sample occur upon sampling and removal of the sample from depth. The goal of high-quality undisturbed sampling is to minimize the potential for: 1. alteration of the soil structure, 2. changes in moisture content or void ratio, and 3. changes in chemical composition of the soil.

The unit weight can also be obtained from an undisturbed tube sample.

Slide 15



Subsurface sampling is expensive and time-consuming, so it makes sense to obtain all information possible from the investigative process and the retrieved samples. After expending the money and effort to obtain subsurface information, the samples should not be subjected to unacceptable temperature, rough handling, shoddy packaging, or harsh transportation methods. Any type of mishandling of a sample may make the sample useless for testing or logging, and the investigation may have to be repeated.

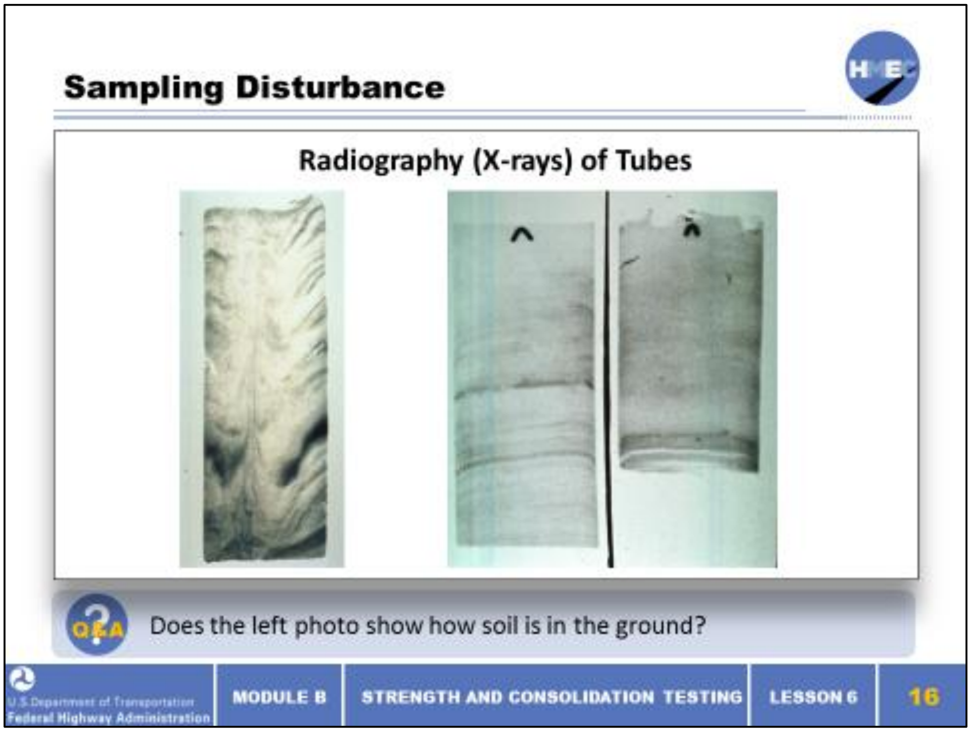
Undisturbed soil samples should be transported and stored so that that the moisture content is maintained as close as possible to the natural conditions (AASHTO T 207, ASTM D4220 and 5079). The end of the sample tubes should be sealed. Samples should not be placed, even temporarily, in direct sunlight. Undisturbed soil samples should be stored in an upright position with the top side of the sample up.

As storage time increases, moisture will migrate within a tube. Potential for disturbance and moisture migration within the sample will increase with time, and samples tested after 30 days should be noted on the laboratory data sheet.

Careless handling of undisturbed soil samples may cause major disturbances that could lead to serious design and construction consequences.

The selection of representative specimens for testing is one of the most important aspects of sampling and testing procedures. Selected specimens must be representative of the formation or deposit being investigated.

Slide 16



This slide shows radiography images of a highly disturbed Shelby tube sample and a relatively undisturbed sample.

As a result of drilling, sampling, sample extrusion, and trimming to form a specimen for testing, nominally undisturbed specimens from samples obtained using methods discussed will become disturbed. These processes change the effective stress condition in the soil sample; that is, the effective stress in the soil at the time after a sample is trimmed and prepared for testing is different than that of the same soil in the ground.

____

____

____

____

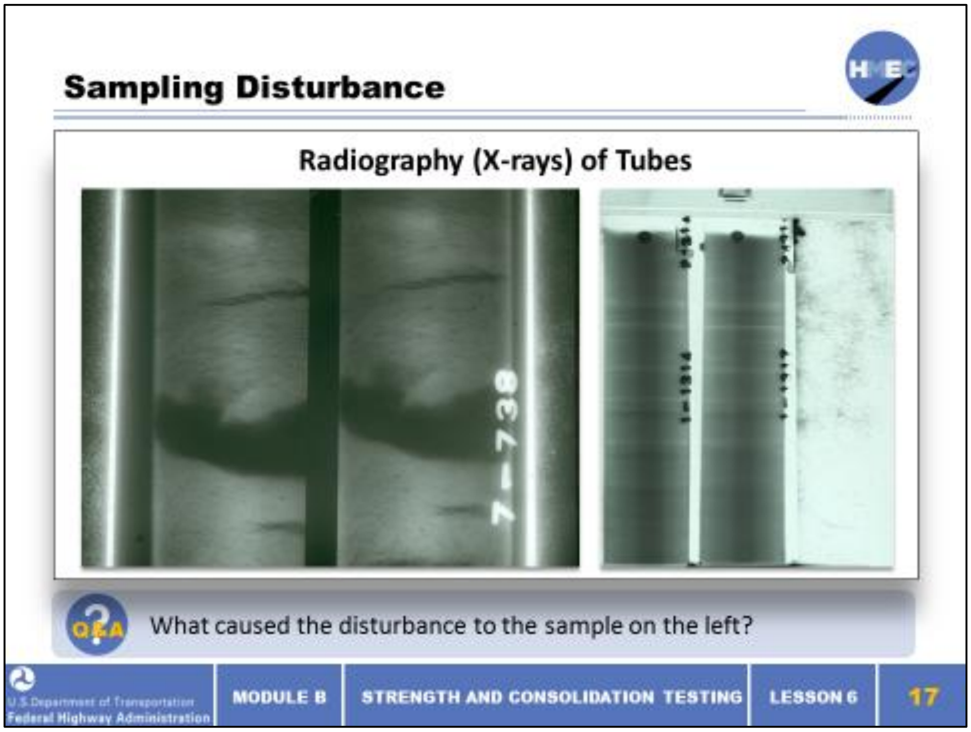
____

____

____

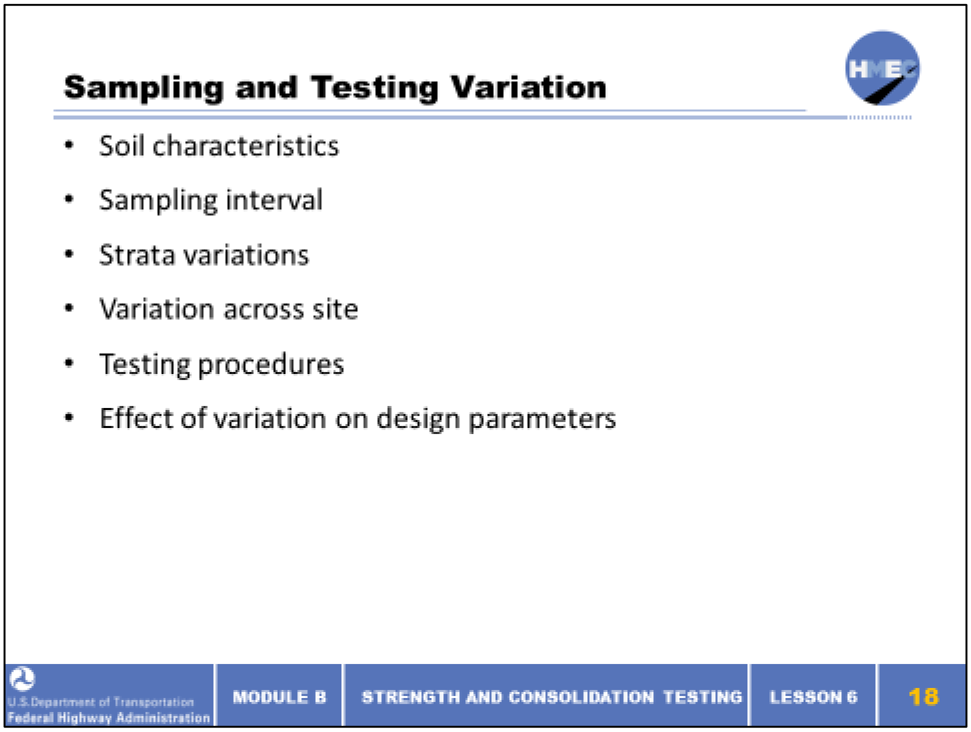
____

Slide 17



Here are two more radiography images of a highly disturbed Shelby tube sample and a relatively undisturbed sample.

Slide 18



Soil is a heterogeneous material, even across a small project site. That variation may be multiplied many times across a large project site. Soil often varies horizontally and vertically. The engineering properties of the soil even within a strata of the same soil may also vary horizontally and vertically. With soil being such a highly variable material, you can see why it is important to minimize the variations that we can control (those due to sampling and testing).

By proper selection of the sampling locations and intervals, enough data points can be obtained to allow us to determine a representative design parameter for a strata of soil that varies both horizontally and vertically.

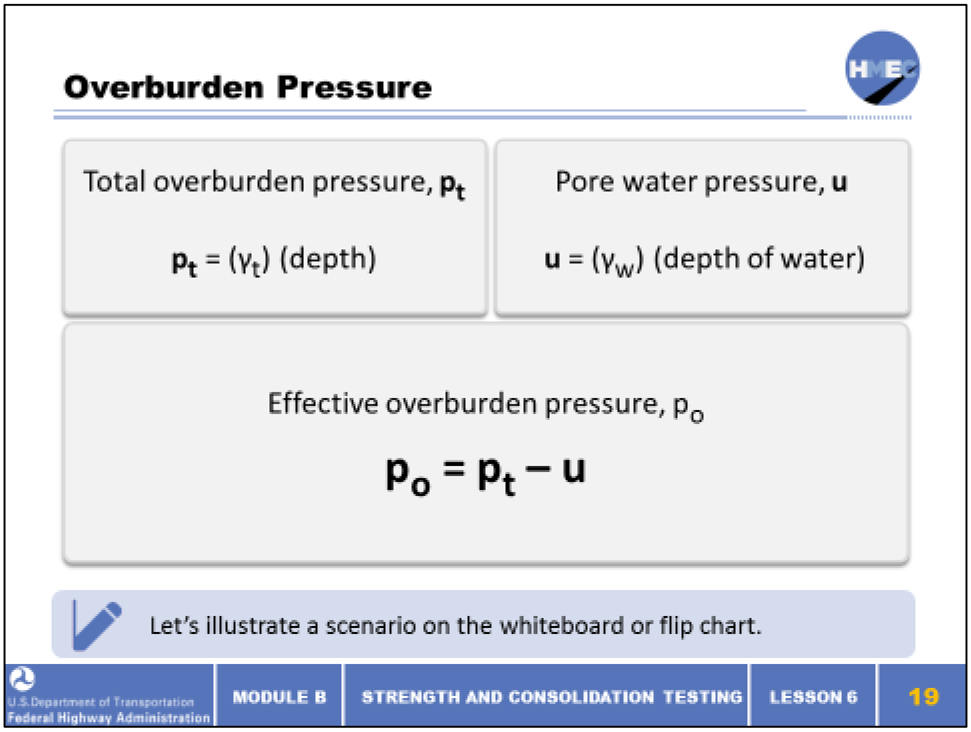
Strict adherence to ASTM or AASHTO sampling procedures by well-trained personnel will minimize disturbance and variation in samples that are obtained and transported to the laboratory for testing.

Once the sample is received by the laboratory, additional ASTM and AASHTO procedures for handling, storage, and testing must also be followed by well-trained laboratory technicians.

The natural variation of soil will have a strong influence on the selection of appropriate design parameters. If additional variation is caused by sampling and testing issues, the designer may be forced to select overly conservative design parameters because of uncertainties that cannot be rationally resolved.

Sampling and testing procedures should include quality control review to ensure variations that can be controlled are controlled. Precision and bias are two measures of sampling and testing variation that can be monitored.

Slide 19



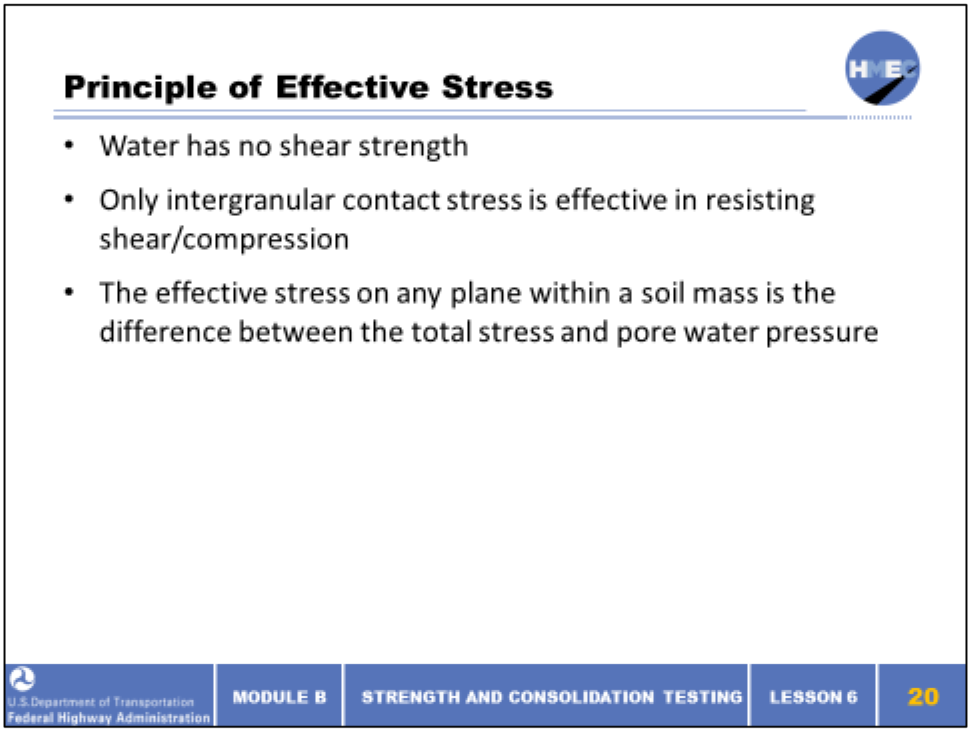
Soils existing at a distance below ground are affected by the weight of the soil above that depth. The influence of this weight, known generally as overburden, causes a state of stress to exist, which is unique at that depth, for that soil. This state of stress is commonly referred to as the overburden, in-situ, or geostatic state of stress.

The total overburden pressure, p_t, is found by multiplying the total unit weight of each soil layer by the corresponding layer thickness and continuously summing the results with depth.

The effective overburden pressure, p_o, at any depth is determined by accumulating the weights of all layers above that depth with consideration of the water level conditions at the site. If the layer is above the water table, $p_o = p_t$. If the layer is below the water table, subtract the pore water pressure at the point of interest. In the case of capillary action and negative pore pressure, effective stress will be greater than the total stress.

When a soil sample is removed from the ground, as during the field exploration phase of a project, that in-situ state of stress is relieved as all confinement of the sample has been removed. In laboratory testing, it is important to reestablish the in-situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied.

Slide 20

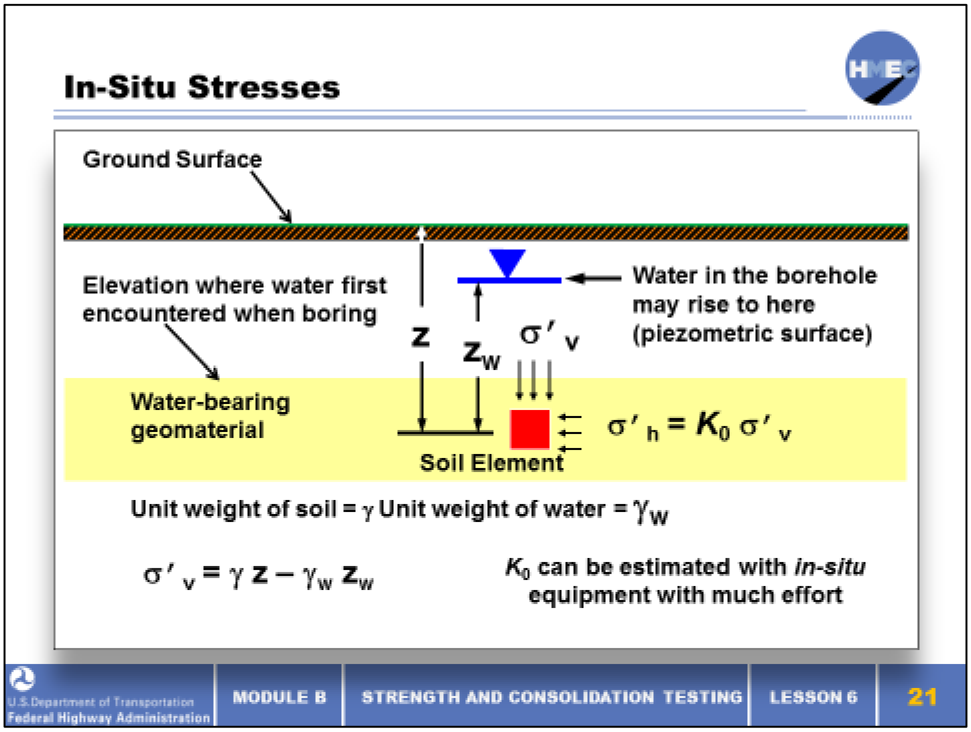


Water may fill the void spaces in the soil structure, but it has no shear strength, therefore, when the soil mass is loaded, excess pore pressure will build up in the pore water and it will begin to drain. How quickly it drains is a function of the soil type and its permeability. The contact stress between soil particles (effective stress) is what resists the shearing and compression caused by the applied load. As the pore water drains, the excess pore water pressure dissipates and the intergranular stress increases.

This phenomena is why some clays may continue to settle for months or even years. The load is initially carried by the intergranular stress and the pore water pressure. As the pore water drains (albeit very slowly), the intergranular stress increases causing the soil particles to adjust into a more dense configuration, which results in continued settlement.

The effective stress at any depth can be calculated by subtracting the pore water pressure from the total stress at that depth.

Slide 21



Let's discuss K_o is important in the design of many geotechnical features. It is an element of soil below the ground. Its in-situ stress is determined by its depth, the weight of soil above it (overburden), the depth and weight of water above it, and the buoyancy of water. Water may rise above initial elevation in the borehole.

The critical points are definition of σ'_v (vertical effective stress) and σ'_h (horizontal effective stress), which control the in-situ state of stress.

Later we will discuss confining stress used in lab tests. They are used to simulate the in-situ stresses shown here.

____

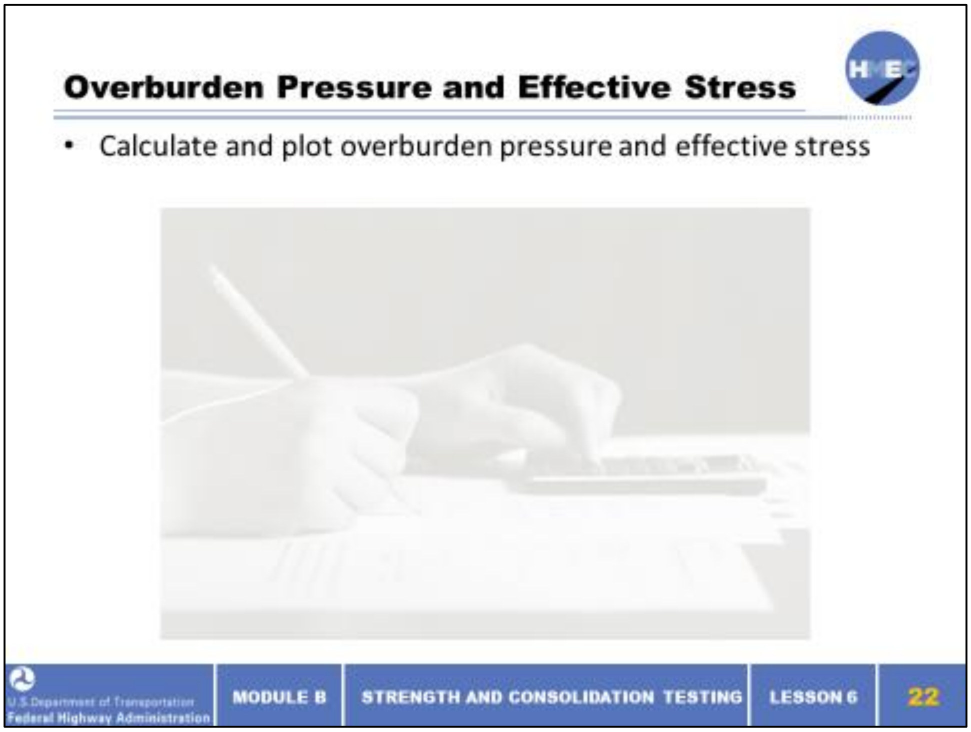
____

____

____

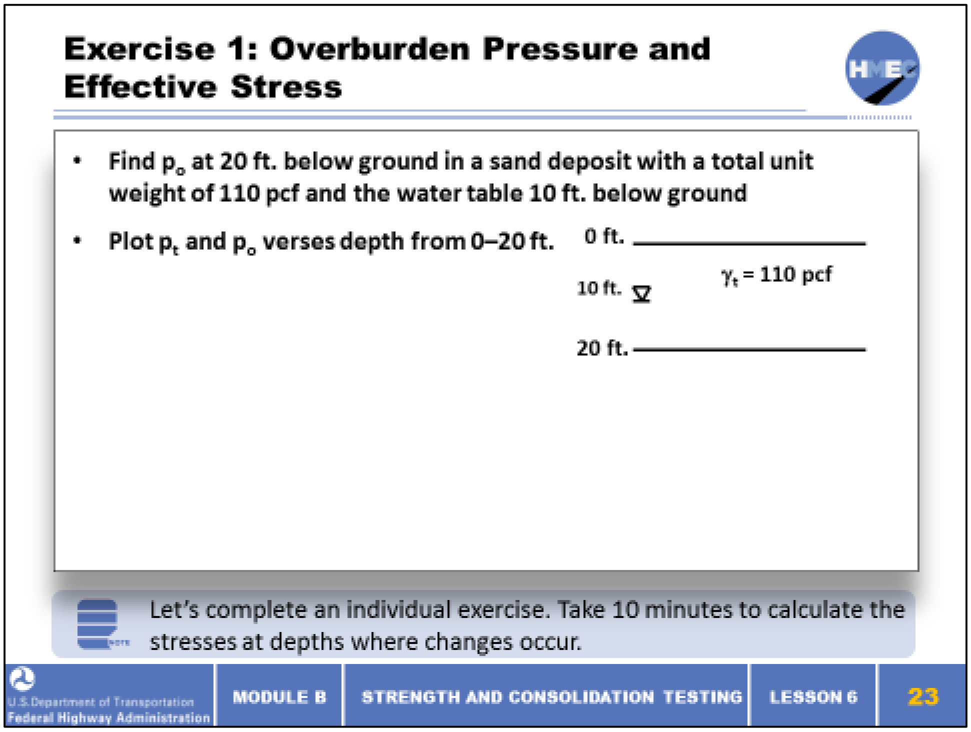
____

Slide 22

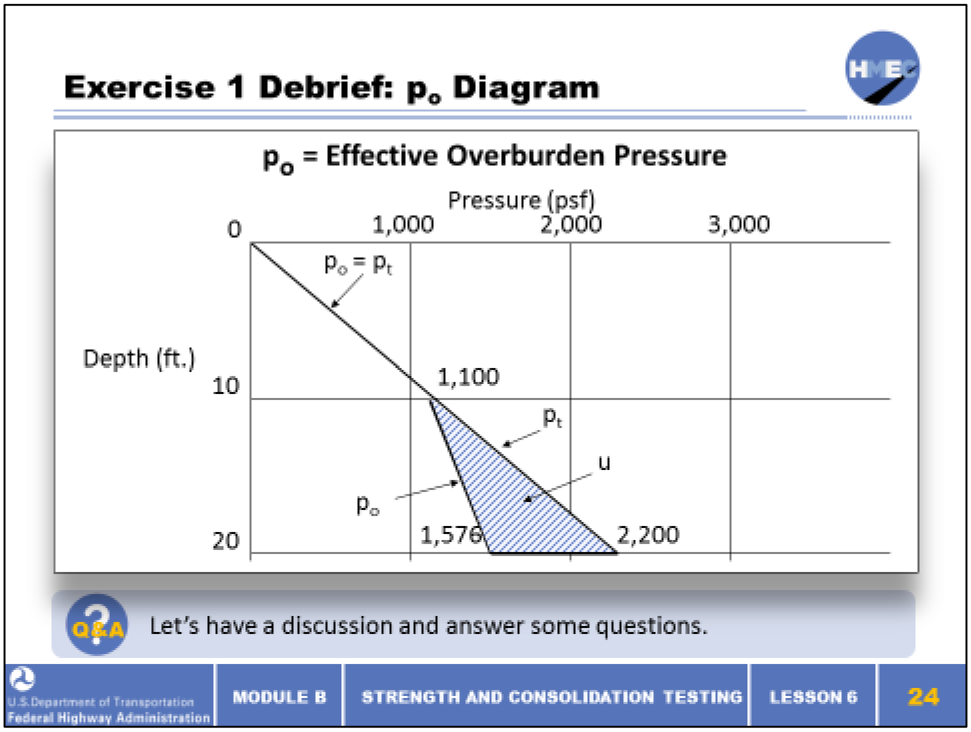


The concepts of overburden pressure and effective stress are important geotechnical concepts. They are used in design, but they are also used during certain laboratory testing to recreate in-situ stress conditions when a sample is tested.

Slide 23

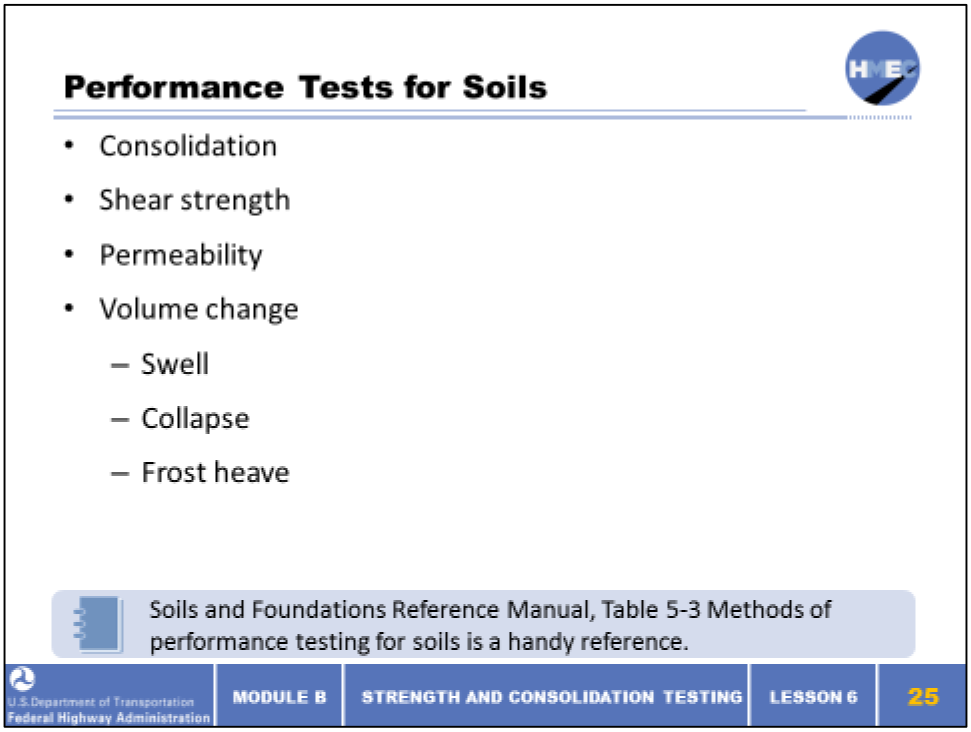

Exercise 1: Overburden Pressure and Effective Stress
• Find p_o at 20 ft. below ground in a sand deposit with a total unit weight of 110 pcf and the water table 10 ft. below ground
• Plot p_t and p_o verses depth from 0–20 ft.
0 ft.
10 ft.
γ_t = 110 pcf
20 ft.
Let's complete an individual exercise. Take 10 minutes to calculate the stresses at depths where changes occur.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
23

Slide 24



Exercise 1 Debrief: p_o Diagram
HMEC
p_o = Effective Overburden Pressure
Pressure (psf)
0
1,000
2,000
3,000
Depth (ft.)
10
20
$p_o = p_t$
1,100
p_t
u
p_o
1,576
2,200
Let's have a discussion and answer some questions.
Q&A
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
24

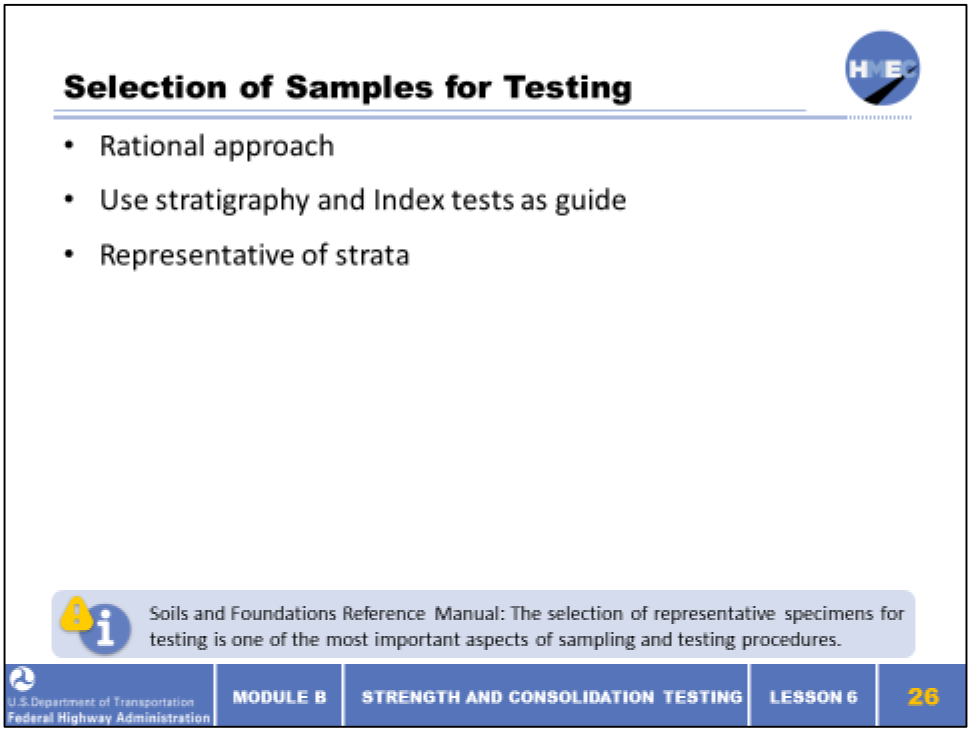
Slide 25



These tests will be performed on undisturbed samples, most of which have had their in-situ state of stress reestablished as part of the test.

Design soil properties for deformation, shear strength, and permeability characteristics are evaluated using laboratory-testing methods. By comparison to most index tests, performance tests are usually more costly and time-consuming. The results, however, provide specific data regarding engineering performance.

Slide 26



The stratigraphy and index tests can guide the selection of sample selection. Chose specimens that represent the strata, not just the worst or the best. Samples with discontinuities and intrusions may fail prematurely in the laboratory. The first inclination would be to test these samples. However, if these features are small and randomly located, they may not necessarily cause such failures in the field.

Note that several tests must be performed at each depth being evaluated, therefore, they will need multiple samples to select enough specimens for testing.

Slide 27

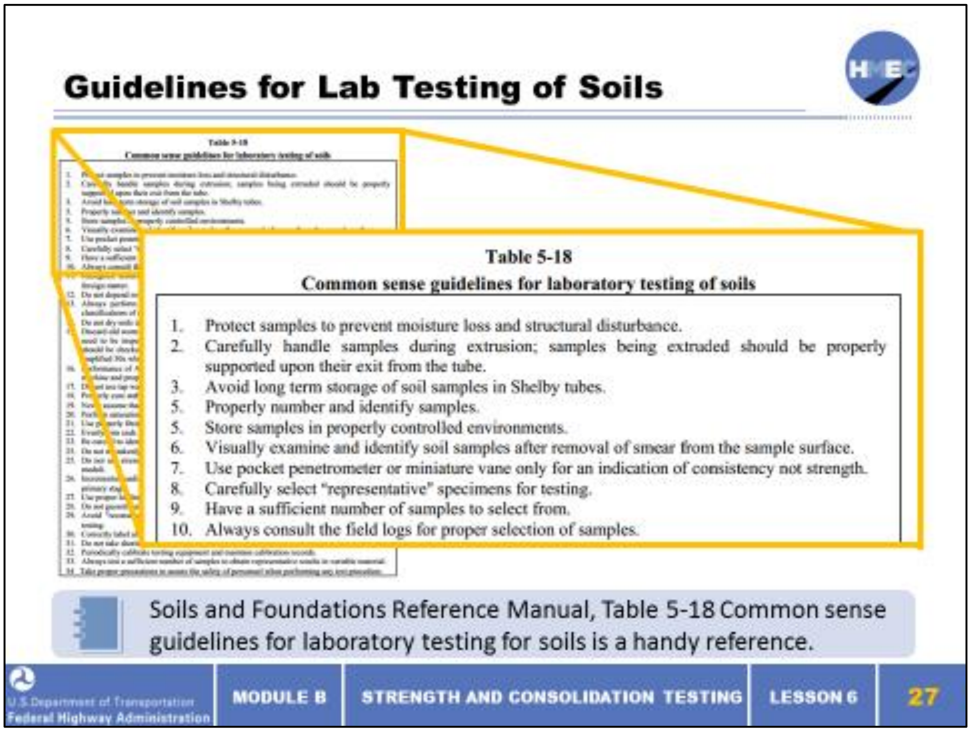


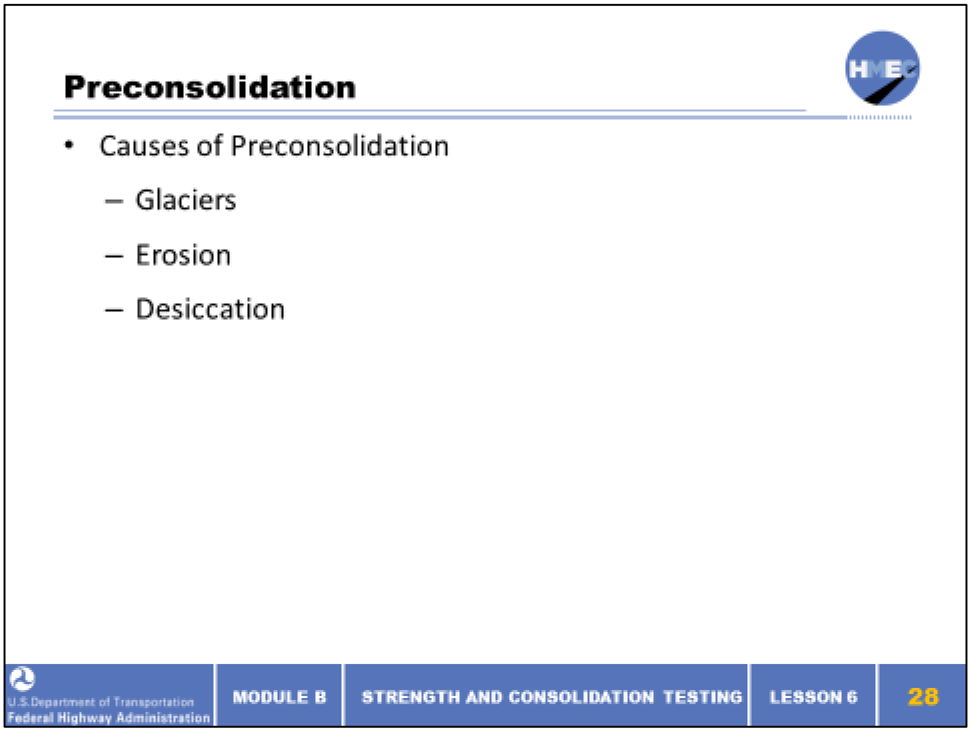
Table 5-18
Common sense guidelines for laboratory testing of soils

1. Protect samples to prevent moisture loss and structural disturbance.
2. Carefully handle samples during extrusion; samples being extruded should be properly supported upon their exit from the tube.
3. Avoid long term storage of soil samples in Shelby tubes.
5. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of consistency not strength.
8. Carefully select a representative @ specimens for testing.
9. Have a sufficient number of samples to select from.
10. Always consult the field logs for proper selection of samples.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
15. Do not dry soils in overheated or under heated ovens.
15. Discard old worn-out equipment; old sieves for example, particularly fine (<No. 40) mesh ones need to be inspected and replaced often; worn compaction molds or compaction hammers should be

checked and replaced if needed. An error in the volume of a compaction mold is amplified 30x when translated to unit volume.
16. Performance of Atterberg limits tests requires carefully adjusted drop height of the liquid limit machine and proper rolling of plastic limit specimens.
17. Do not use tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.
19. Never assume that all samples are saturated as received.
20. Perform saturation by applying properly staged back pressures of adequate magnitude.
21. Use properly fitting o-rings, membranes, etc. in triaxial or permeability tests.
22. Evenly trim ends and sides of undisturbed samples.
23. Be careful to identify and report slickensides and natural fissures.
25. Do not mistakenly identify failures due slickensides as shear failures.
25. Do not use stress-strain curves from unconfined compression test results to determine elastic moduli.
26. Incremental loading of consolidation tests should be performed only after the completion of the primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label all laboratory test specimens.
31. Do not take shortcuts by using non-standard equipment or non-standard test procedures.
32. Periodically calibrate testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.
34 Take proper precautions to assure the safety of personnel when performing any test procedure.

____
____
____
____
____
____
____
____
____
____
____

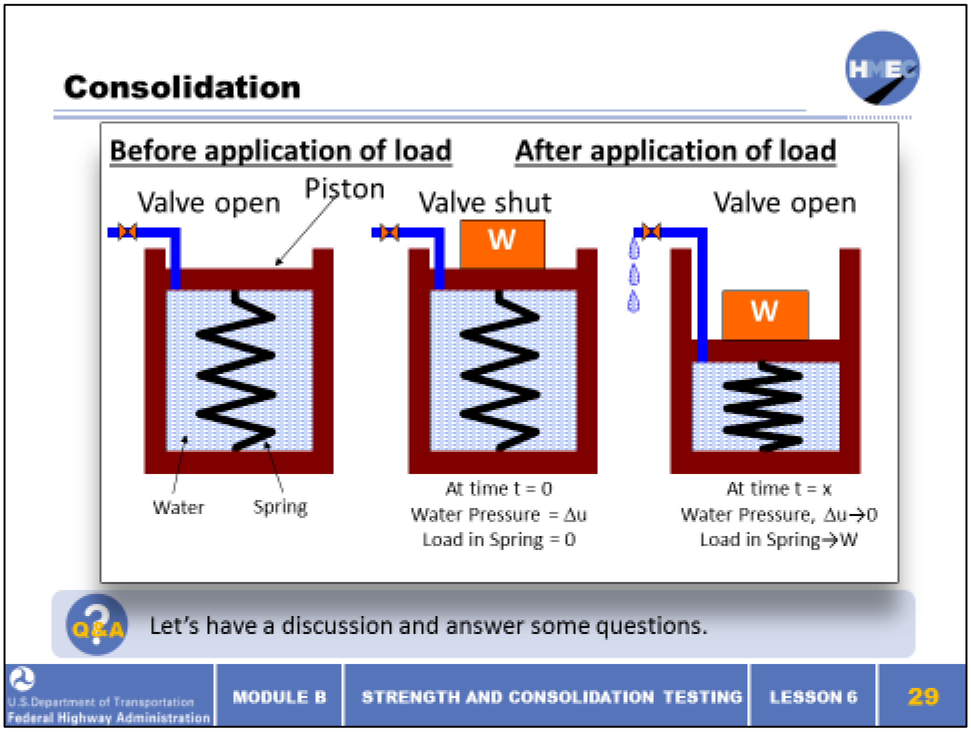
Slide 28



Some natural deposits experienced compression in geologic history due to the weight of glaciers, the weight of overlying soil that has been eroded, or due to desiccation. Since their void ratios were substantially reduced in the past by these processes, these soils are less compressible today. Such soils are called “preconsolidated” or “over consolidated” since they have been subjected to greater stresses in the past than exist at present.

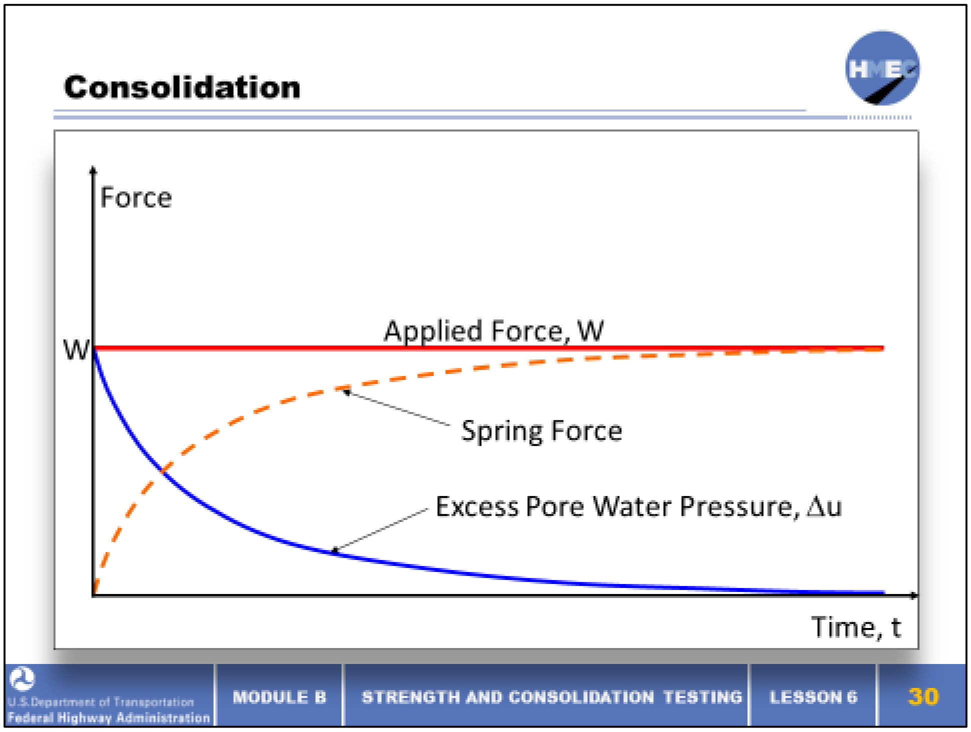
Preconsolidated soil has a “memory” of that past consolidation stress, which affects its performance when subjected to loads from the projects (embankments, spread footings, etc.). The soil will be stronger and settle less until current loads exceed that past consolidation stress.

Slide 29



When a load is initially applied to soil, the increased stress is carried by the pore water until the pore water begins to drain. This illustration should help visualize that process.

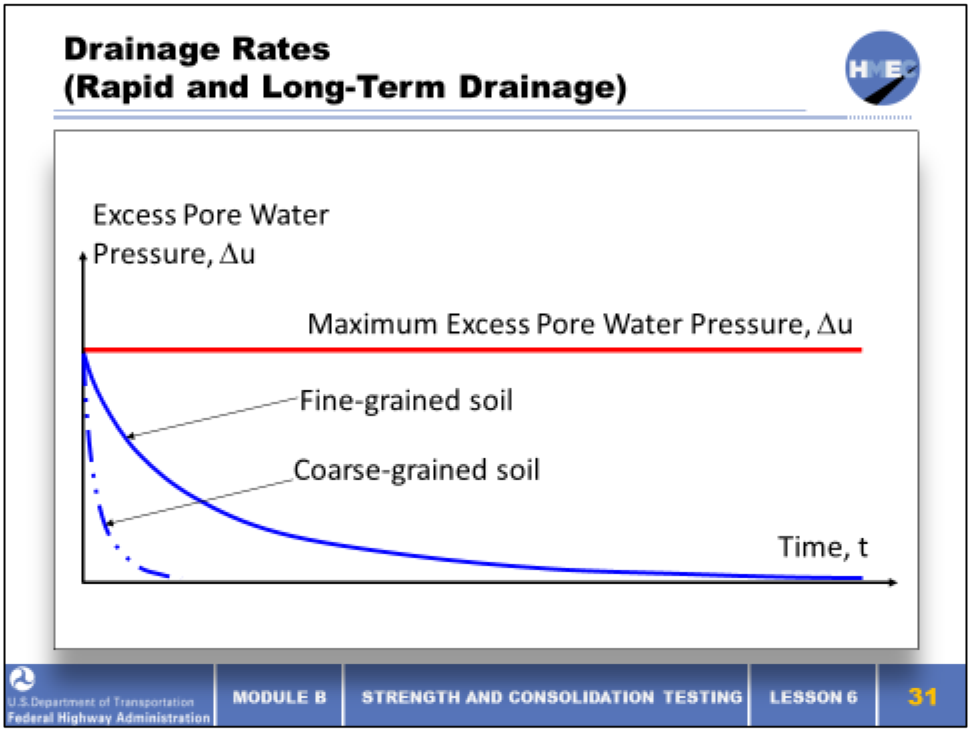
Slide 30



This slide provides another illustration of how stress from an applied load is transferred to the soil intergranular contact stress from the excess pore water pressure.

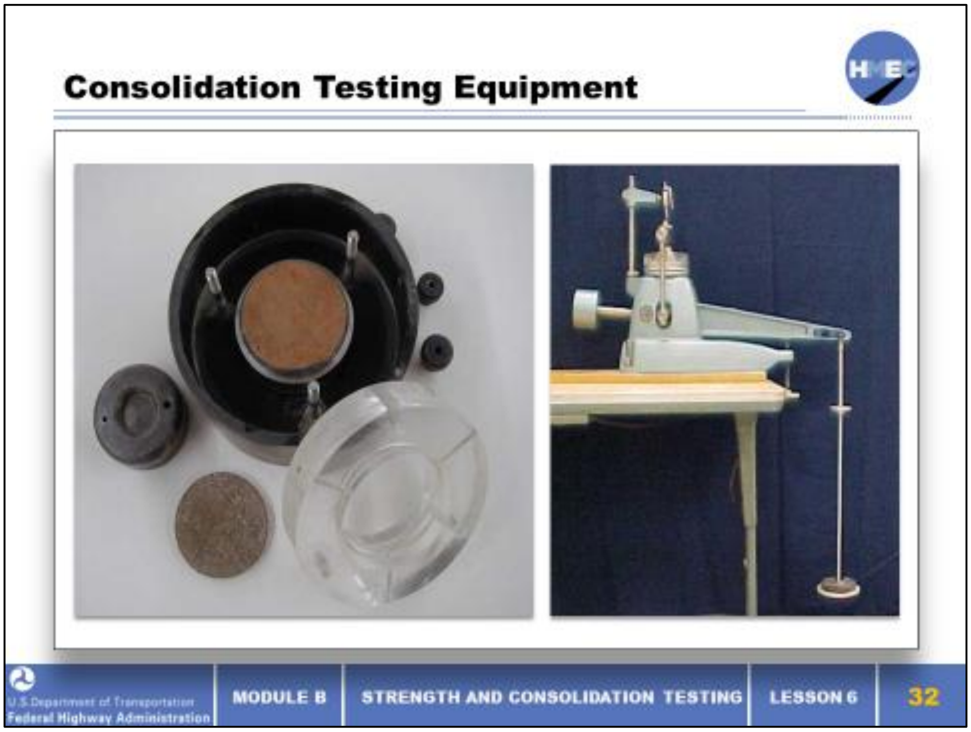
Hydrostatic pore water pressure will still exist after consolidation under a given stress increment is complete. In consolidation process, when water pressure goes up, the effective stress goes down and vice versa, i.e., when water pressure goes down, the effective stress goes up.

Slide 31



Permeability is so high and drainage is so fast that most sands and gravels consolidate immediately. Basically they consolidate during construction; that is, during application of the load. On the other hand, clays can take months or years for the excess pore water pressure to dissipate and the consolidation to occur.

Slide 32



What test can we perform to allow us to estimate consolidation and drainage properties of the soil? The one-dimensional consolidation properties of soil test. (AASHTO T 216 or ASTM D2435)

The oedometer (or one-dimensional consolidometer) is the primary laboratory equipment used to evaluate consolidation and settlement potential of cohesive soils. Results from oedometer tests are also used to assess the rate of consolidation (t_{50} or t_{100}), creep characteristics (C_a), stress history (p_c'), and swell potential. A consolidation test is typically performed on high quality undisturbed samples obtained from the deposit to evaluate settlement potential of in-situ foundation soils, however, recompacted materials can also be tested to assess the settlement performance of compacted fills.

The loading sequence selected for a consolidation test will depend on the type of soil being tested and the particular application (e.g., embankment, shallow foundation) being considered for the project. The range of applied loads for the test should well exceed the effective stresses that are required for settlement analyses. This range should cover the smallest and largest effective stresses anticipated in the field and will depend on depth, foundation loads, and excavations.

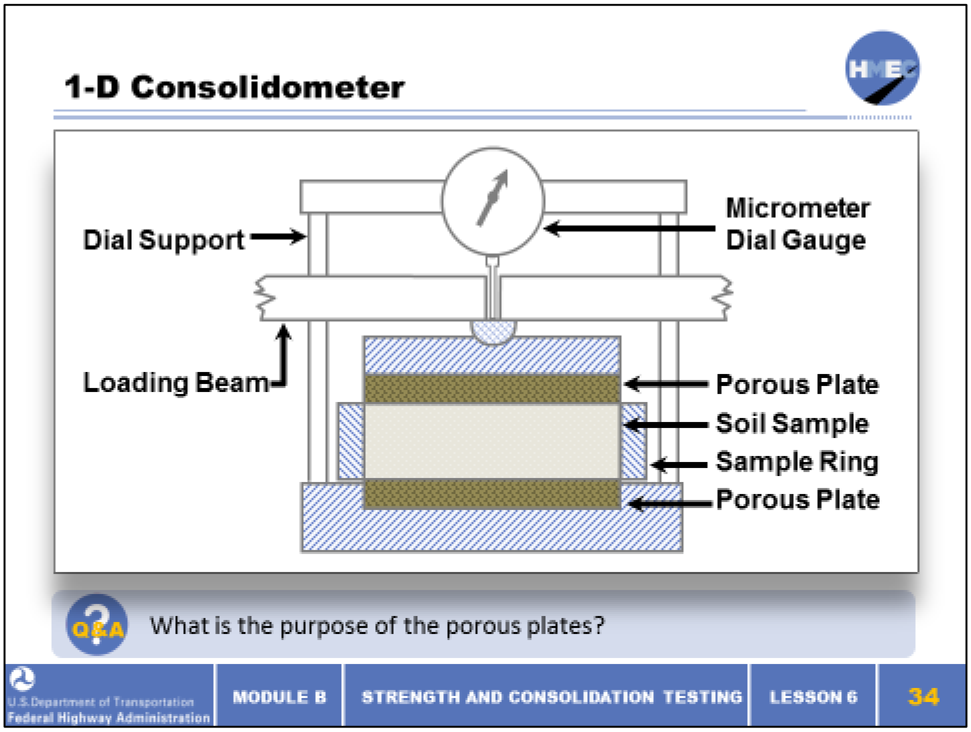
Point out that the weighted lever arm consolidometer is the most common consolidation equipment in most labs, but automated oedometer equipment is becoming more commonplace.

Slide 33

A weighted lever arm provides a robust, relatively simple system for consolidation testing, however, because data are generally recorded manually, it is difficult to expedite testing or vary the loading schedule since data reduction cannot typically be performed in real time. The constant rate of strain (CRS) version mitigates some of those issues.

The CRS version of the consolidation test (ASTM D4186) applies the loading continuously and measures stress and pore pressures by transducers in real time, thereby reducing testing times from approximately one week by IL oedometer to about one day by a CRS consolidometer. While expediting the testing time duration, the CRS consolidation test requires special instrumentation and equipment that is not normally available in State DOT laboratories. A discussion of the CRS and other consolidation methods are described in Head (1986) and Lowe et al. (1969).

Slide 34



This schematic of the consolidation test helps visualize the test set-up. With automated systems there will be linear variable differential transducers (LVDTs), etc.

____

____

____

____

____

____

____

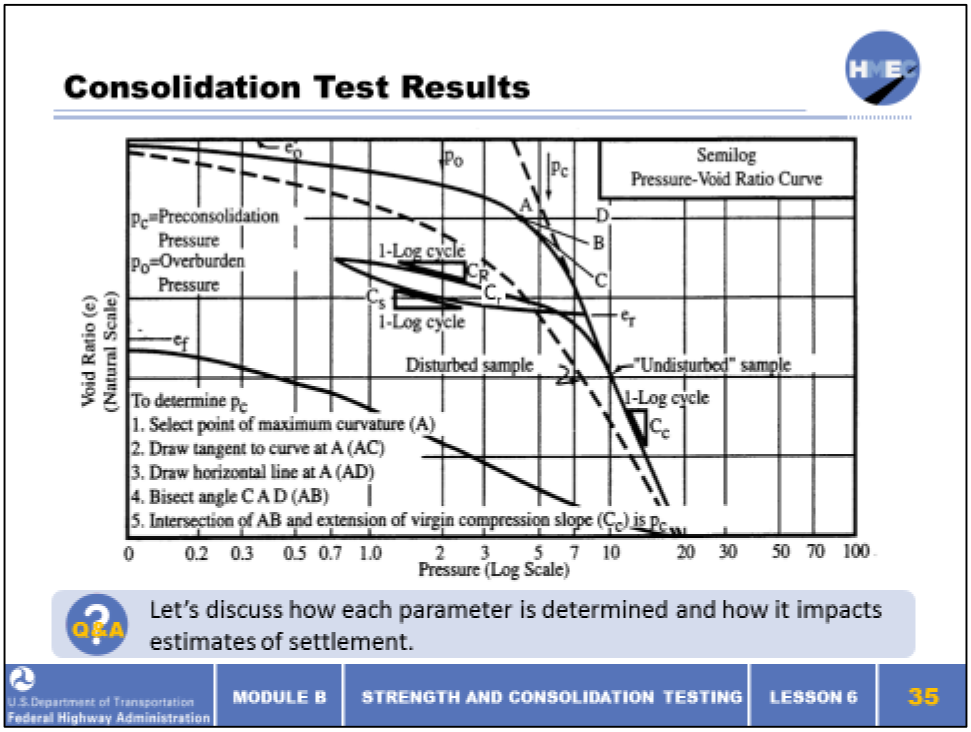
____

____

____

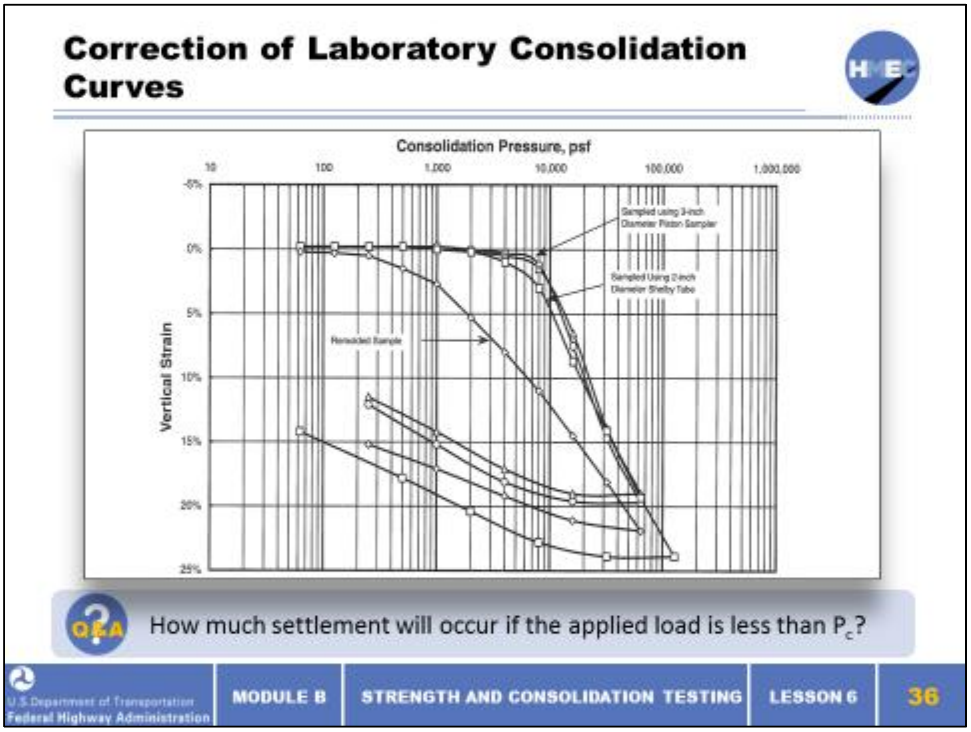
____

Slide 35



This slide shows the results from the consolidation test. The results are used to estimate settlement and the time of consolidation (how long does it take for the settlement to occur).

Slide 36

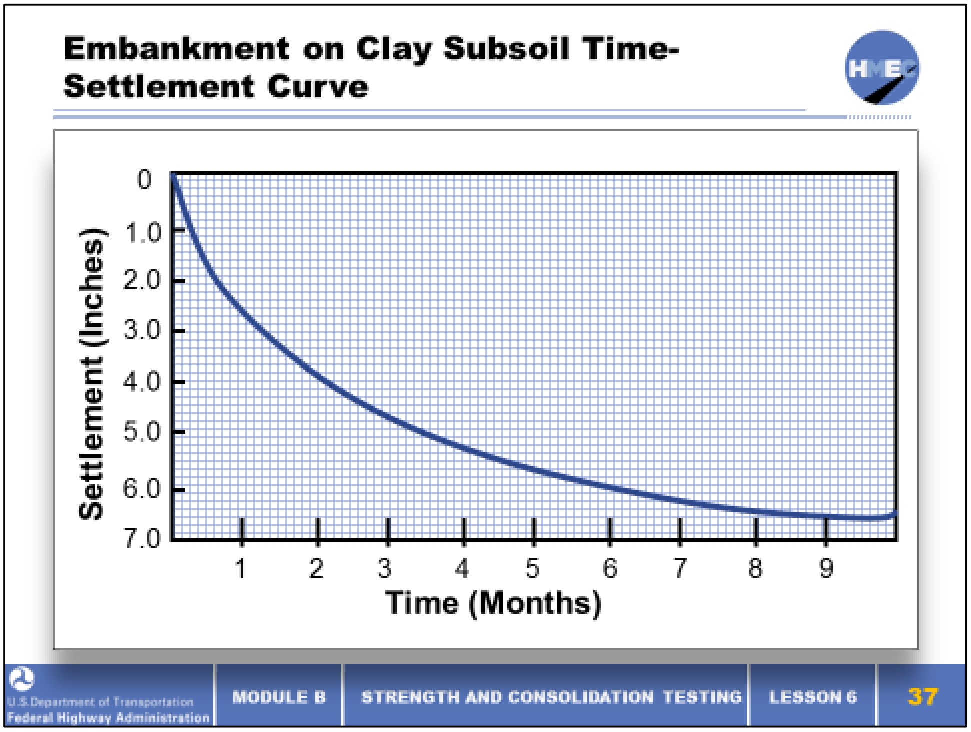


Most samples that are brought to the lab and tested have some level of disturbance. Test results need to be evaluated and corrected if necessary.

The disturbance of the sample has the greatest influence on preconsolidation pressure, P_c. This is critical because P_c defines whether the clay is normally consolidated, over consolidated, or under-consolidated.

This plot uses vertical strain instead of void ratio. It provides a better illustration of the amount of settlement that occurs on each segment of the curve, initial versus virgin. This can be used to discuss what happens to settlement estimates if P_c is underestimated due to sample disturbance or poor test procedures.

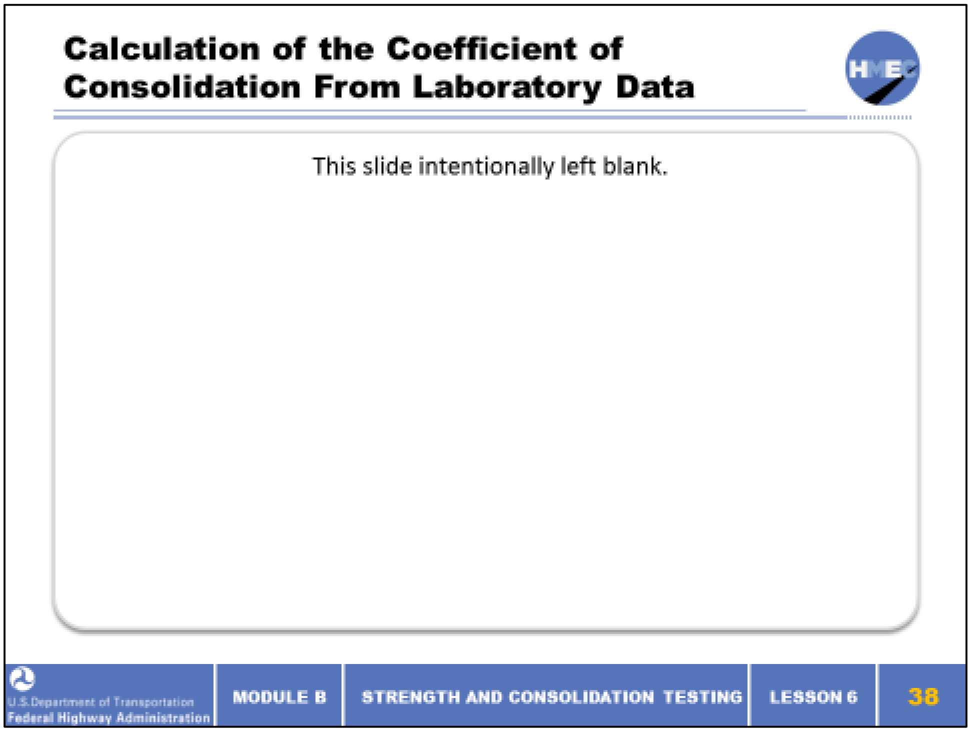
Slide 37



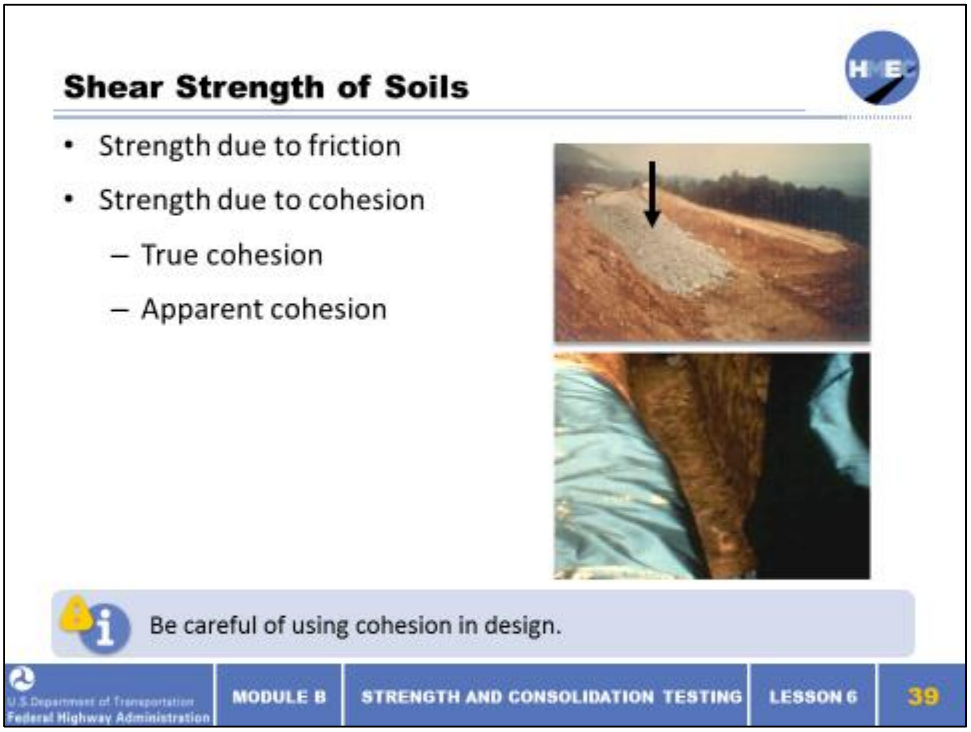
The data on the time it takes to consolidate the sample (coefficient of vertical consolidation, C_v from the consolidation test can be used to generate a plot to depict how much settlement will occur over a given time period.

It is important to note that c_v is *not a constant*, but varies with both the level of stress and degree of consolidation. However, it is typically determined for the appropriate load range for the site and used as a constant.

Slide 38


Calculation of the Coefficient of
Consolidation From Laboratory Data
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
38

Slide 39



Shear strength is another of the design parameters determined by laboratory performance tests. Shear strength in soils is due to friction between soil particles and/or cohesion. Friction is due to the resistance of soil particles as they slide past each other. Cohesion can either be true cohesion or apparent cohesion. Recall that true cohesion is due to either the electrochemical bond between soil particles or chemical cementation. True cohesion is typically quite small and is often ignored. Apparent cohesion can be quite large and it occurs in partially saturated fine-grained soils due to the negative pore pressure from surface tension in the water acting on the soil particles.

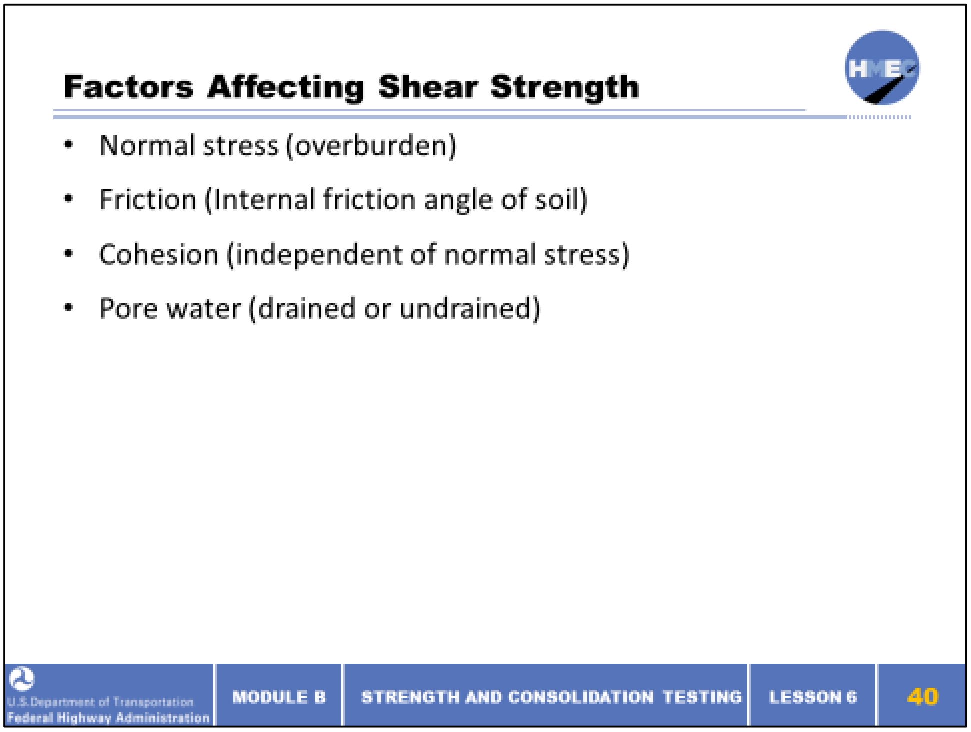
The top photograph shows a purely frictional soil where all of the shear strength is due to intergranular friction between the coarse-grained material. The bottom photograph shows a predominantly cohesive soil where the shear strength is due to either true or apparent cohesion. Unless the complete soil sample is composed of colloidal particles, true cohesion due to interparticle attraction cannot be relied upon. The cohesion shown here is most likely apparent cohesion, which has resulted in many trench collapses and deaths when the apparent cohesion is lost due to a change in the saturation level and/or vibrations.

There are some mixed soils that derive strength from both intergranular friction and cohesion (bonding). Such soils are commonly denoted as “c-ϕ soils.” Since cohesion cannot be defined with confidence, its contribution to long-term shear strength in c-ϕ soils is often disregarded or greatly minimized by using only a small value such as 100 to 500 psf in design.

Note that shear is not a fundamental or unique property of soil, but instead, a specific behavioral response to a certain set of loading conditions.

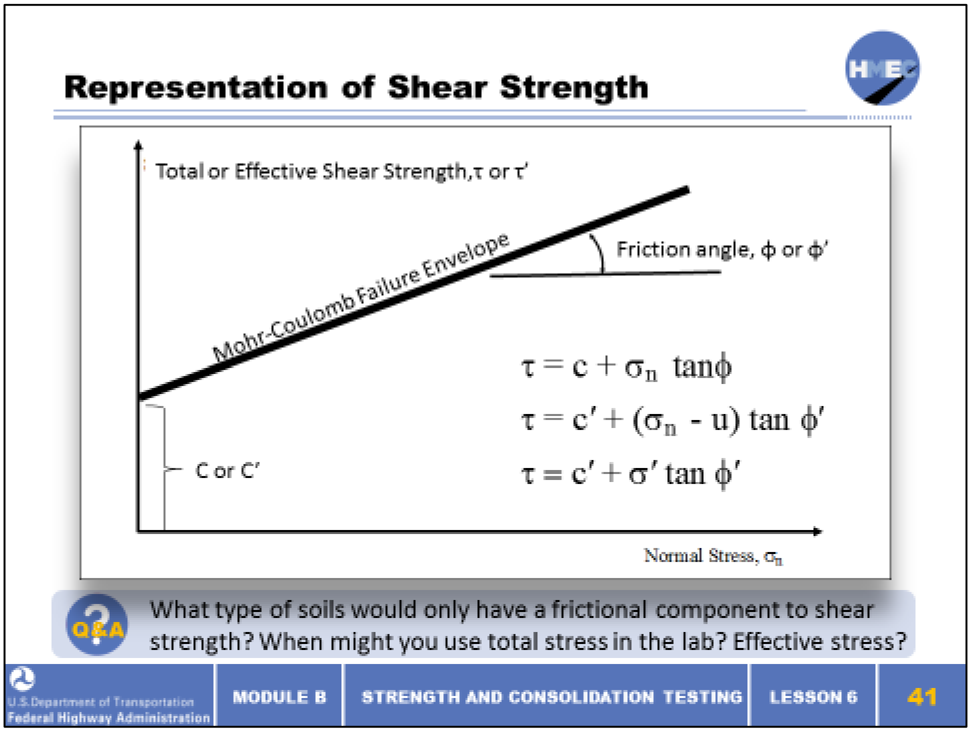
The shear strength soil is influenced by many factors including the effective stress state, mineralogy, packing arrangement of the soil particles, soil hydraulic conductivity, and rate of loading, stress history, sensitivity, and other variables. As a result, the shear strength of soil is not a unique property.

Slide 40



Many factors influence the shear strength of soil. Shear strength due to friction is dependent on the normal force pushing down on the particles that are sliding past each other and the roughness of the particles (internal friction angle of the soil). The normal force is the weight of soil (overburden pressure) and loads on the soil. We need friction angle and normal stress to determine shear strength. This component of shear strength increases with increasing friction angle and increasing normal stress.

Slide 41



Here is a graphical representation of the two components of shear strength: friction angle and cohesion. This is known as the Mohr-Coulomb (M-C) diagram and is plotted based on the results of laboratory performance tests, such as unconfined compression (UC), unconsolidated undrained (UU), consolidated undrained (CU), consolidated drained (CD), or direct shear.

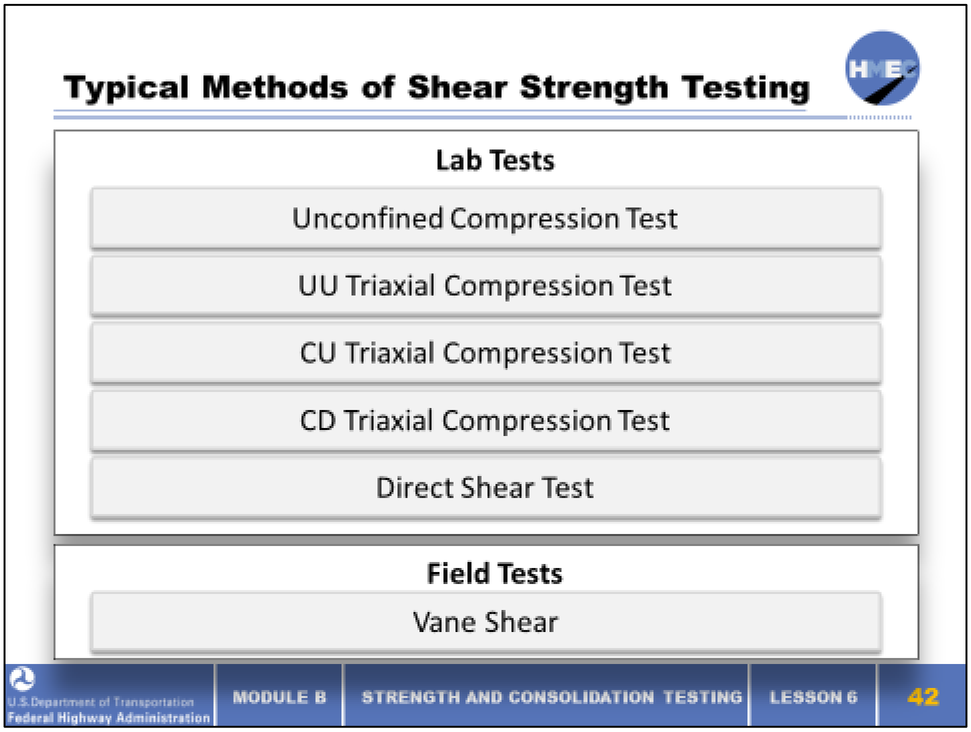
The M-C plot may look different depending on the test performed and whether the soil is cohesive, granular, or a mixture.

Note that C and C' and ϕ and ϕ' being shown on the same graph is only for simplicity. They are not the same and each parameter will be a result of the appropriate test.

The first equation represents the undrained or total stress condition. This is the condition before pore water has drained and the loads are carried by the soil particle contact and the excess pore water pressure. The second and third equations represent a drained or effective stress condition. This is the condition after pore water has drained and the loads are carried by the soil particle contact.

The maximum shear stress at failure is determined for at least three different soil samples subjected to different confining pressures (we'll talk about that later) and the results plotted to generate an M-C failure envelope. The slope of that failure line is the tangent of the internal angle of friction for the soil. Using the friction angle and the normal stress, the frictional component of the shear strength can be determined. The shear strength where the failure line intercepts the shear strength (Y) axis is the cohesion component of shear strength for the soil. As indicated by the equations, those components are added together to obtain the shear strength of the soil.

Slide 42



We will not discuss the vane shear test. However, it can provide both peak and residual shear strength values for cohesive materials as well as valuable correlation data for laboratory tests.

These tests are normally performed on fine-grained soils.

Because undisturbed samples of granular soils are very difficult to obtain, laboratory strength tests are rarely performed to determine their shear strength. Typically, the strength for granular soil is assigned on the basis of in-situ tests such as SPT or CPT. (See Soils and Foundations Reference Manual 5.5.6.3 for shear strength of cohesion less soils.)

____

____

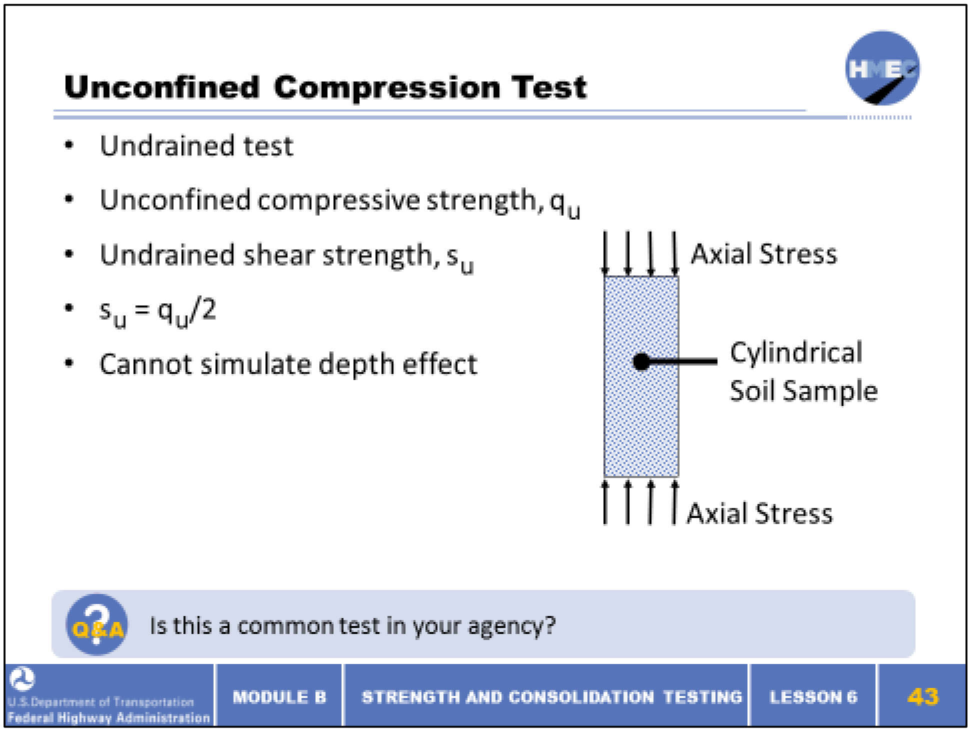
____

____

____

____

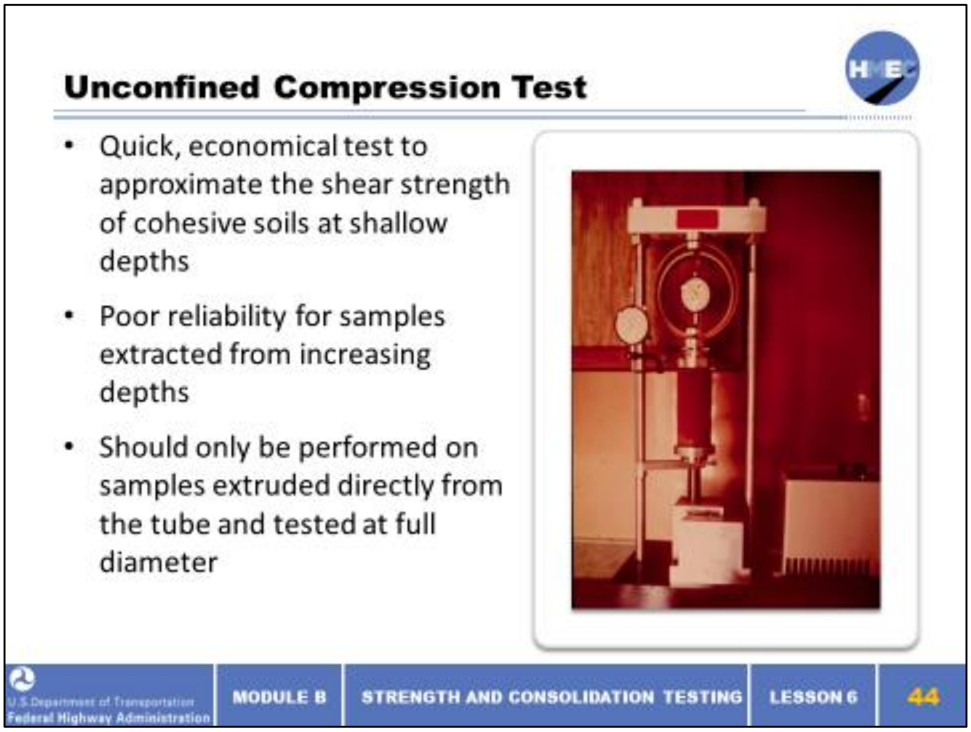
Slide 43



The unconfined compression test is a quick, relatively inexpensive means to obtain an approximate estimation of undrained shear strength of cohesive specimens. In most cases, undrained strength results from an unconfined compression test are conservative.

The limitations of the UC test listed below, need to be considered to ensure it is use only when appropriate. The UC test should not routinely be used to determine design strengths. However, when correlated with triaxial tests, the UC test is an economical way to test multiple soil samples for design.

Slide 44



This is a photograph of the unconfined compression test.

In this test, a cylindrical specimen of the soil is loaded axially, without any lateral confinement to the specimen, at a sufficiently high rate to prevent drainage. Since there is no confinement, residual negative pore pressures that may exist in the sample following sample preparation control the state of effective stress. This test cannot be performed on granular soils, dry or crumbly soils, silts, peat, or fissured or varved materials.

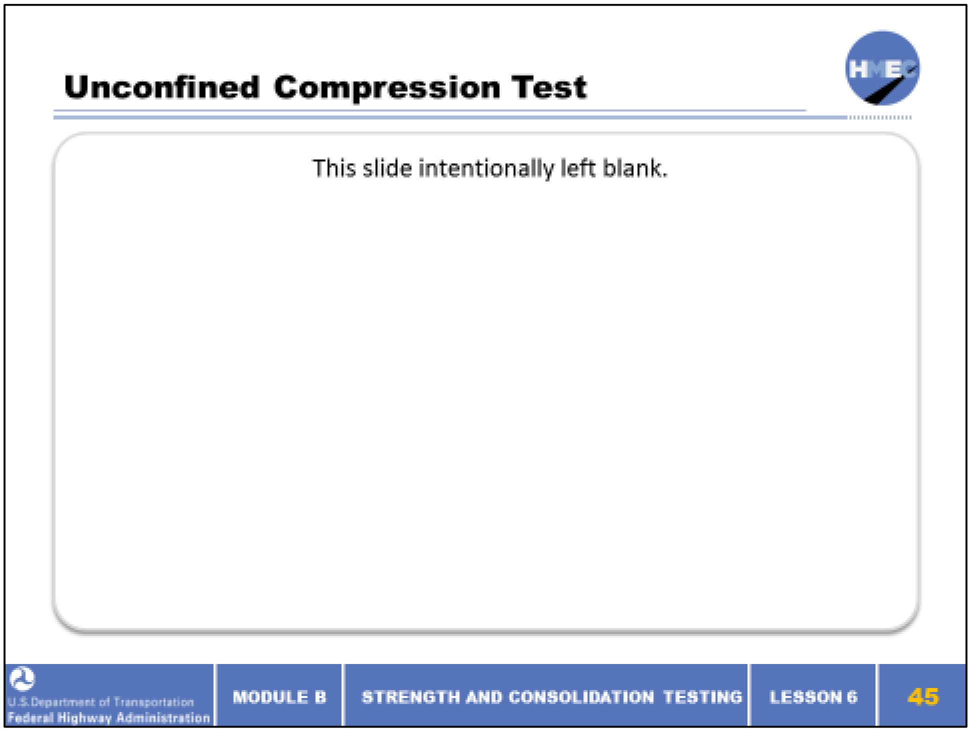
Because the sample is not consolidated or confined to re-establish the in-situ state of stress, the test results become less reliable for samples from increasing depths. Testing the full diameter of the extruded specimen as soon as possible after removal from the tube can minimize swelling. This reduces disturbance and preserves natural moisture content.

____

____

____

Slide 45


Unconfined Compression Test
HMEC
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
45

____

____

____

____

____

____

____

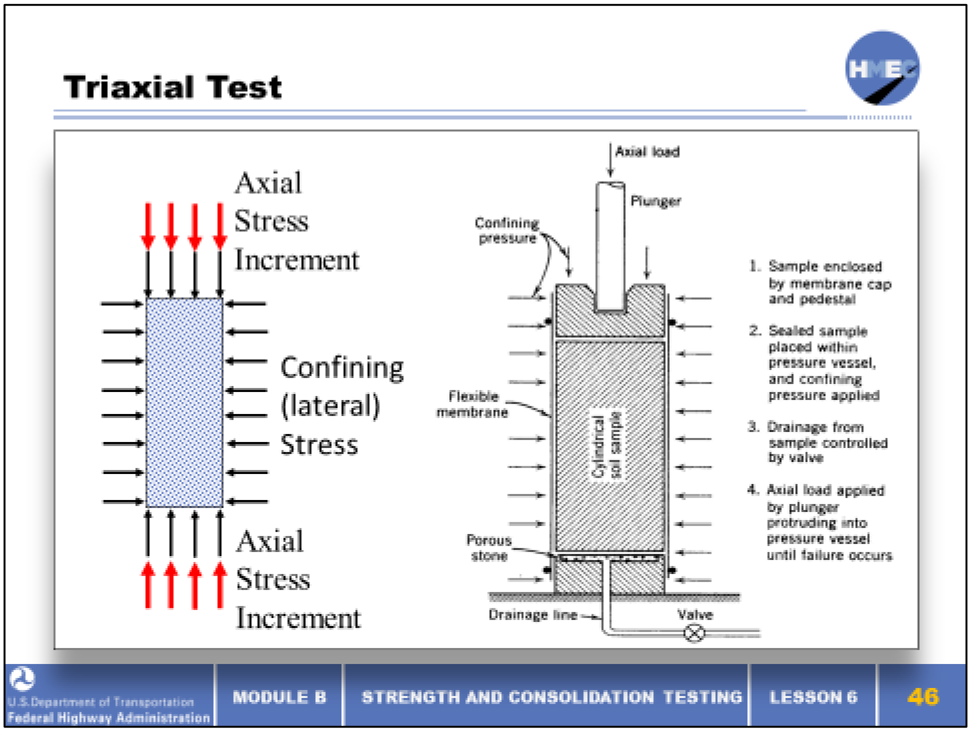
____

____

____

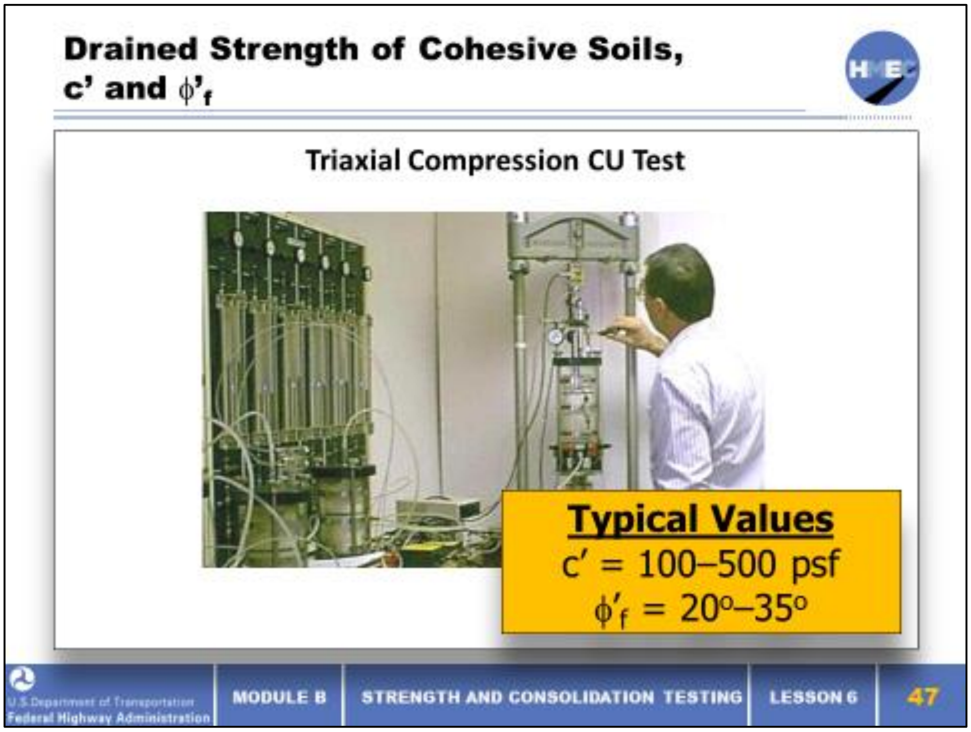
____

Slide 46



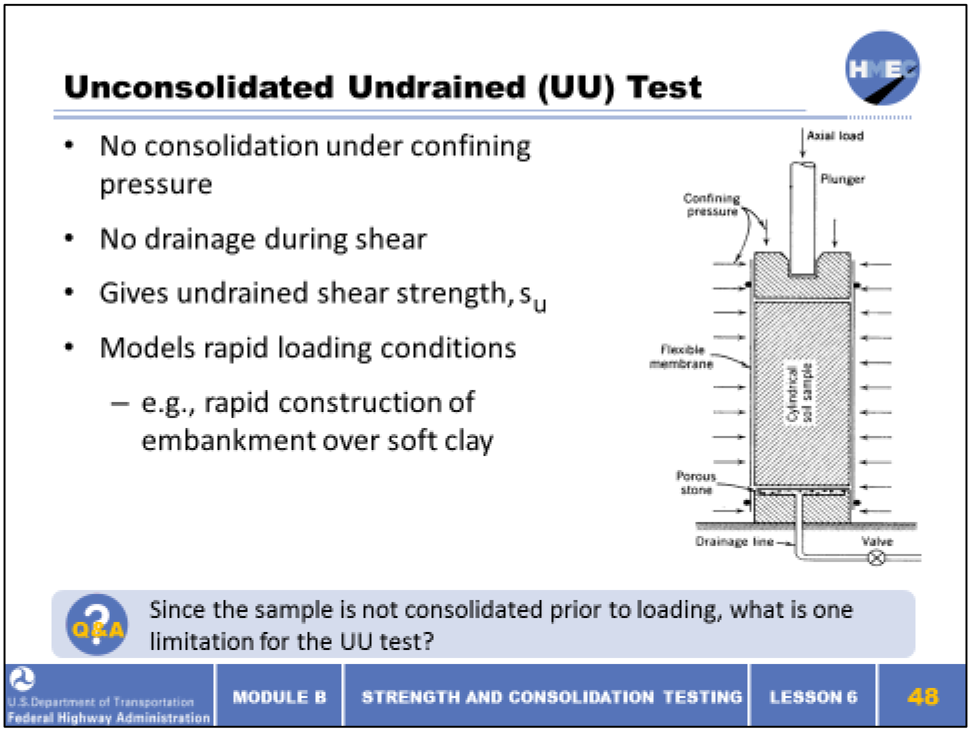
The triaxial test is also a common laboratory strength test. The triaxial test is very versatile in the sense that the shear strength can be evaluated under compression as well as extension loading conditions. A schematic of triaxial compression test is shown in here where the axial stress is greater than the confining stress. Lateral pressures at various depths below the ground surface can be simulated by confining pressures. Note that the confining pressures acts on the entire sample and is equal to the axial stress before the application of an axial stress increment. Typically, failure of the sample is caused by increasing the axial stress (compression) until a shear failure takes place. In an extension test, the confining pressures are increased while keeping the axial stress constant. Pour water pressures during the test can be measured.

Slide 47



This CU test was used to determine the effective stress values of cohesion and friction angle for the soil. Typical values are shown. The values shown here for c' are minimal values used for design since cohesion is an uncertain parameter.

Slide 48



The UU test is conducted in accordance with AASHTO T 296. There is no drainage allowed during shear, therefore this is an undrained, total stress test. The strength parameter from the test is the undrained shear strength. The strength test performed should be chosen to represent the conditions in the field. This test models the response of a soil that has been subject to a rapid application of confining pressure and shearing load. Therefore, it is appropriate for rapid construction of an embankment over soft clay. The low permeability of the clay will not allow drainage as the embankment load is placed, therefore, an undrained test is appropriate. This allows the designer to determine if the undrained shear strength of the soft clay is adequate to support the full height embankment or if stage construction is required.

____

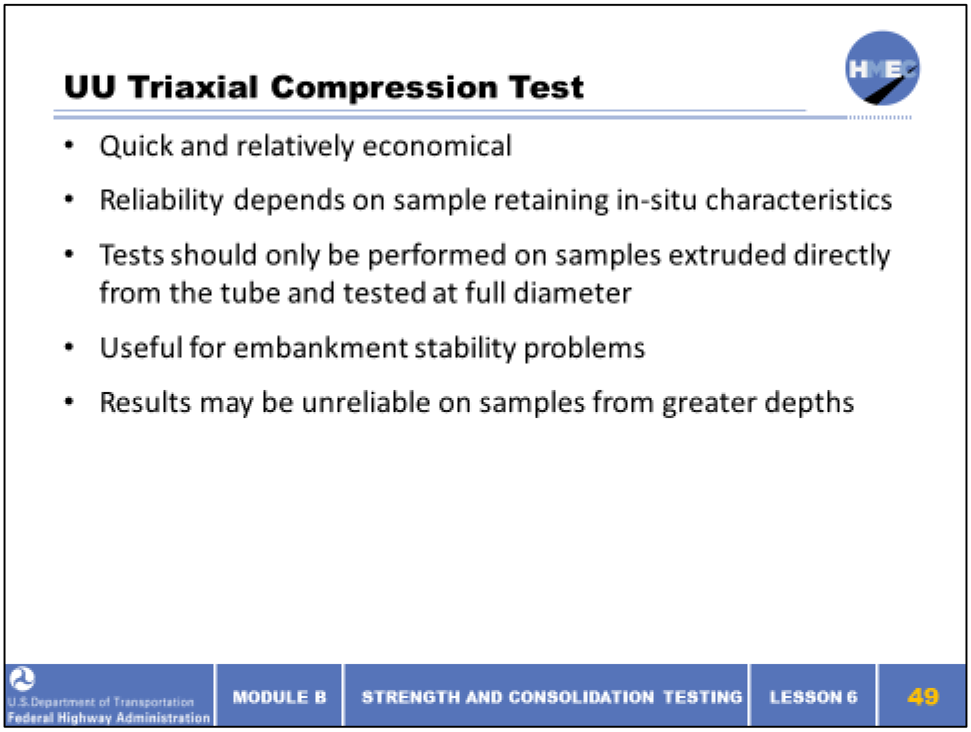
____

____

____

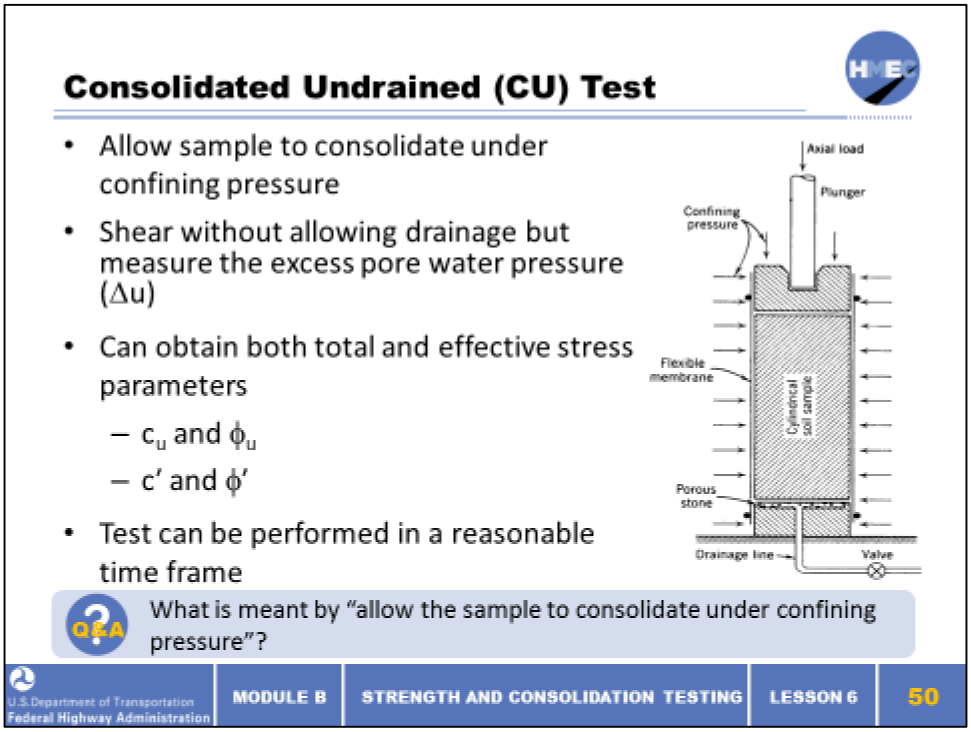
____

Slide 49



The UU test is quick and fairly economical, and therefore widely used. It is important that the sample retain its in-situ characteristics; however, that is difficult, because of disturbance, negative pore pressure in the sample, and loss of moisture. If the test is performed on full diameter samples immediately after they are extruded from the tube, some of these issues can be minimized. The test is useful for embankment stability problems but is considered unreliable for samples taken on normally consolidated clays from a depth greater than about 20 ft. because of this reduction in effective stress.

Slide 50



In the initial part of the CU test the specimen is allowed to consolidate under the confining pressure. The axial load is applied with the drainage lines closed in the CU test, so drainage does not occur. Thus, during shearing there is continual development (+ or -) of excess pore water pressure. Pore pressures are measured during the CU test so that both total stress and effective stress strength parameters can be obtained. (ASTM D4767)

____

____

____

____

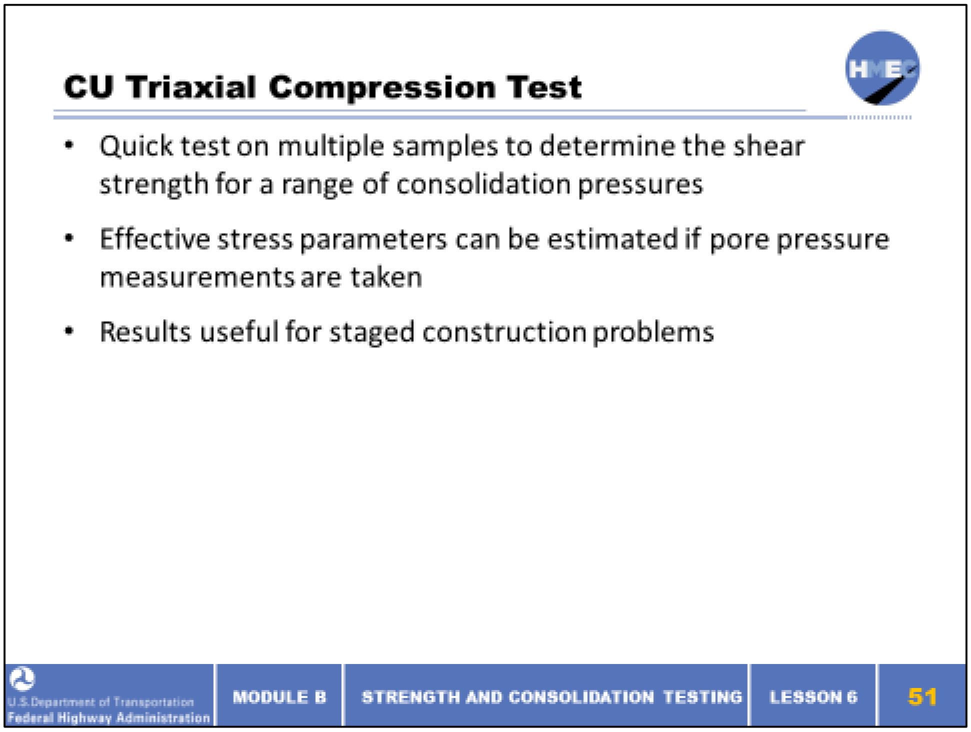
____

____

____

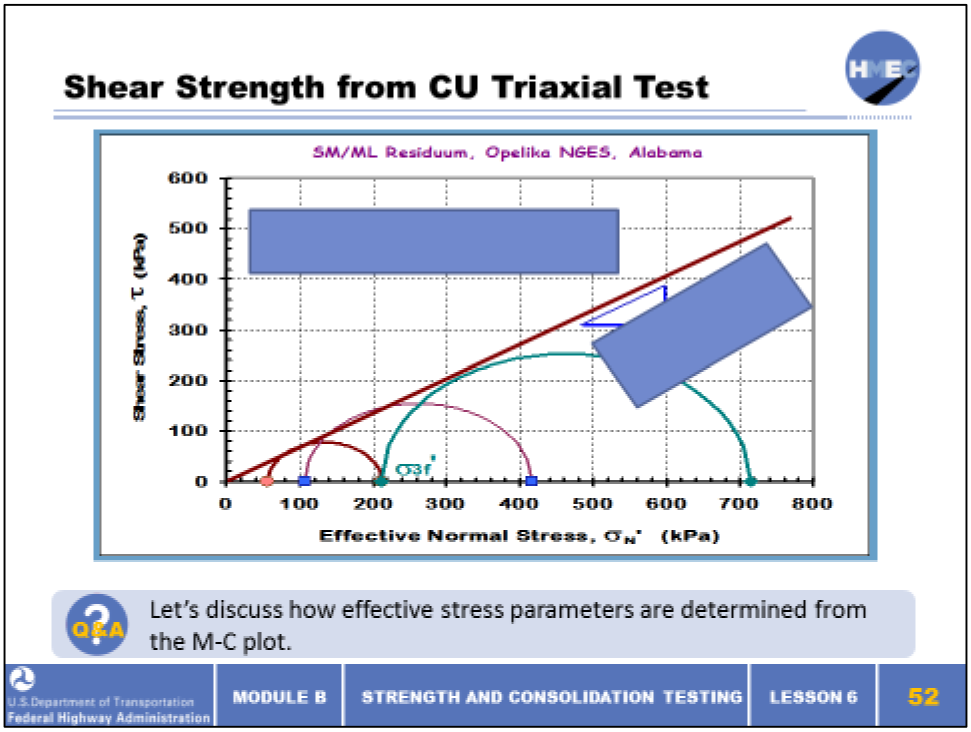
____

Slide 51



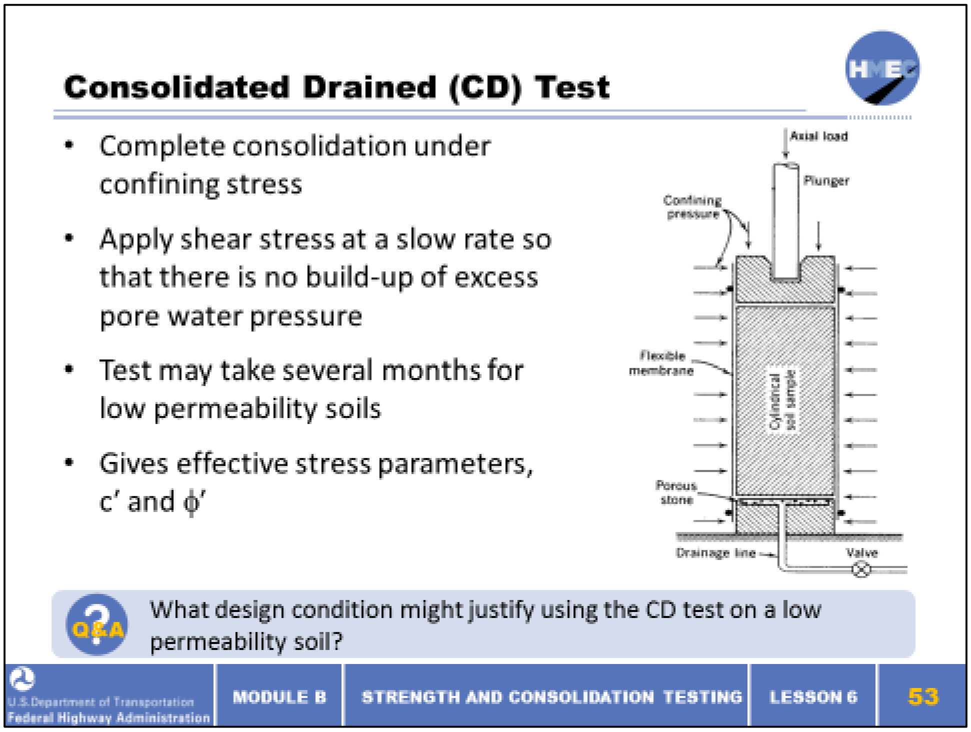
Consolidated undrained triaxial testing with pore pressure measurements provides data for both total and effective stress strength interpretation. Unlike the UU test, the sample in a CU test is reconsolidated in the laboratory to a predetermined consolidation pressure. The CU test can be consolidated to higher than existing in-situ stress to determine fine-grained soil strength gain due to consolidation under staged fill heights.

Slide 52



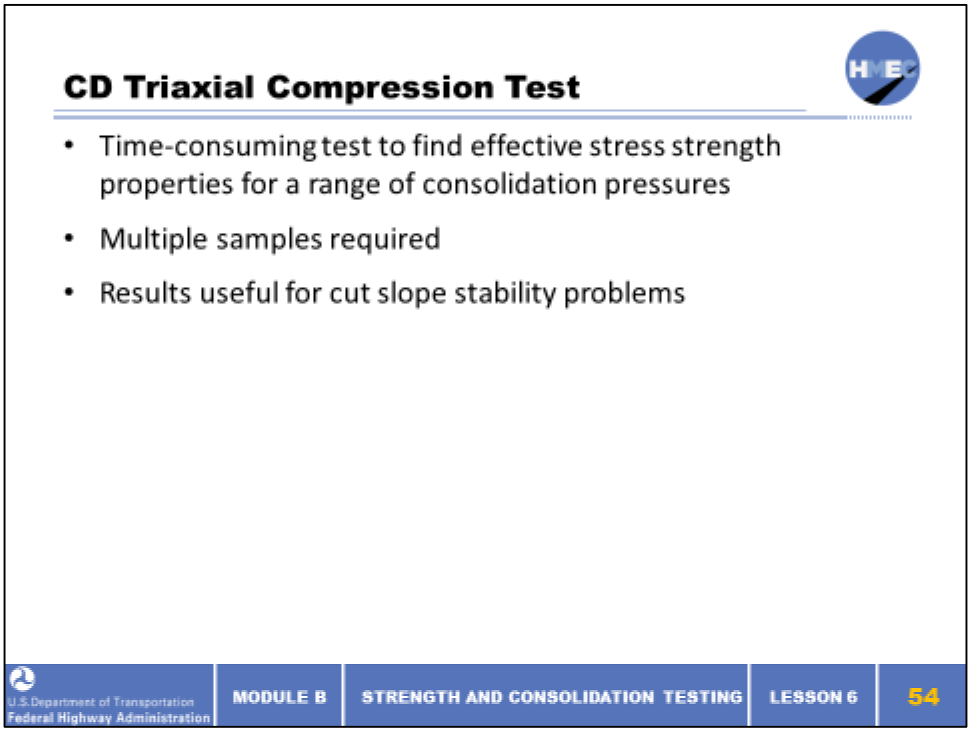
Here is another example of a CU triaxial test on a sandy silt. Although it is in metric, it still provides a good example of an actual test result on a residual soil.

Slide 53



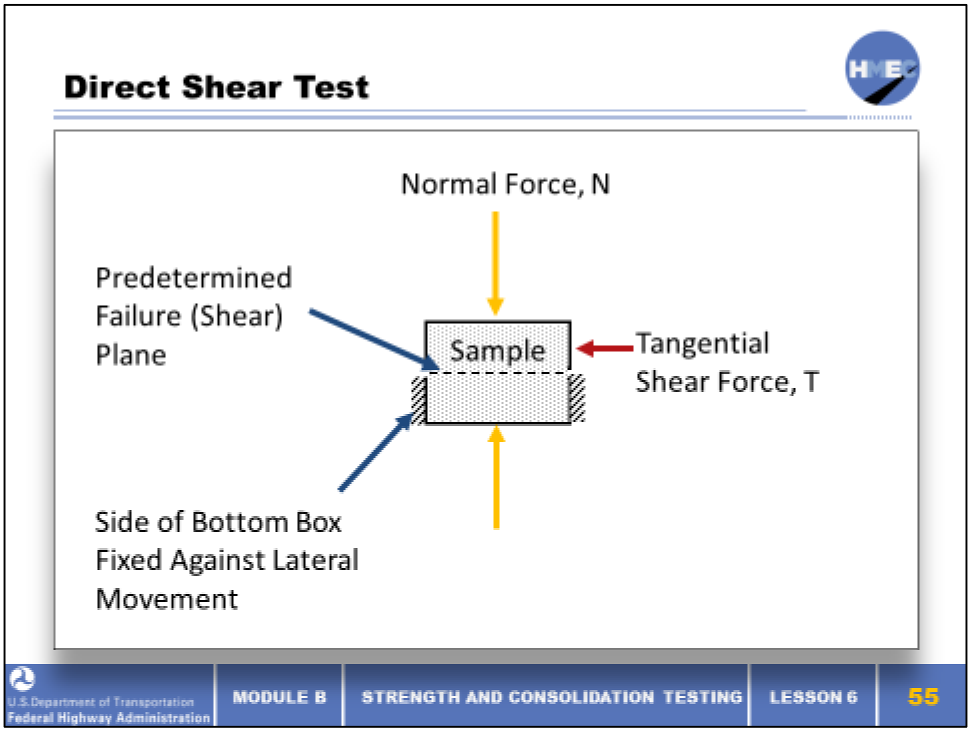
In the CD test, the specimen is allowed to consolidate completely under the confining pressure prior to the application of axial load, i.e., the confining pressure acts as an effective stress throughout the soil specimen. The axial load is applied at a rate slow enough to allow drainage of pore water so that there is no buildup of excess pore water pressures, i.e., the stresses imposed by the axial load are effective stresses. The shear stresses induced in the specimen by the axial load result in failure. The time required to conduct this test in low permeability soil may be as long as several months. Therefore, it is not common to conduct a CD test on low permeability soils. The CD test models the long-term (drained) condition in soil. Effective stress strength parameters (i.e., ϕ' and c′) are evaluated from the results of the CD test. (ASTM D7181-11)

Slide 54



The primary issue with the CD test is the time it takes to run when testing low permeability soils. The CU test with pore water measurements has proven to produce similar effective stress parameters, making CD tests unnecessary for typical applications. As with other triaxial tests, at least three tests must be run on three samples, however, most laboratories would run them simultaneously.

Slide 55



The oldest form of shear test upon soil is the direct shear test, first used by Coulomb in 1776. A schematic of the essential elements of the direct shear apparatus is shown. The soil is held in a box that is split across its middle; the bottom portion of the box is usually fixed against lateral movement. A confining normal force, N, is applied, and then a tangential shear force, T, is applied so as to cause relative displacement between the two parts of the box. The magnitude of the shear force is recorded as a function of the shear displacement, and usually, the change in thickness of the soil sample is also recorded. Although it is widely used in practice, the direct shear device lacks a number of features that limit is applicability. For example, there is no way to control the confining pressure. Also, since there is no way to measure excess pore water pressures generated during shearing of saturated clay specimens, use of the direct shear test is generally limited to cohesion less soils.

____

____

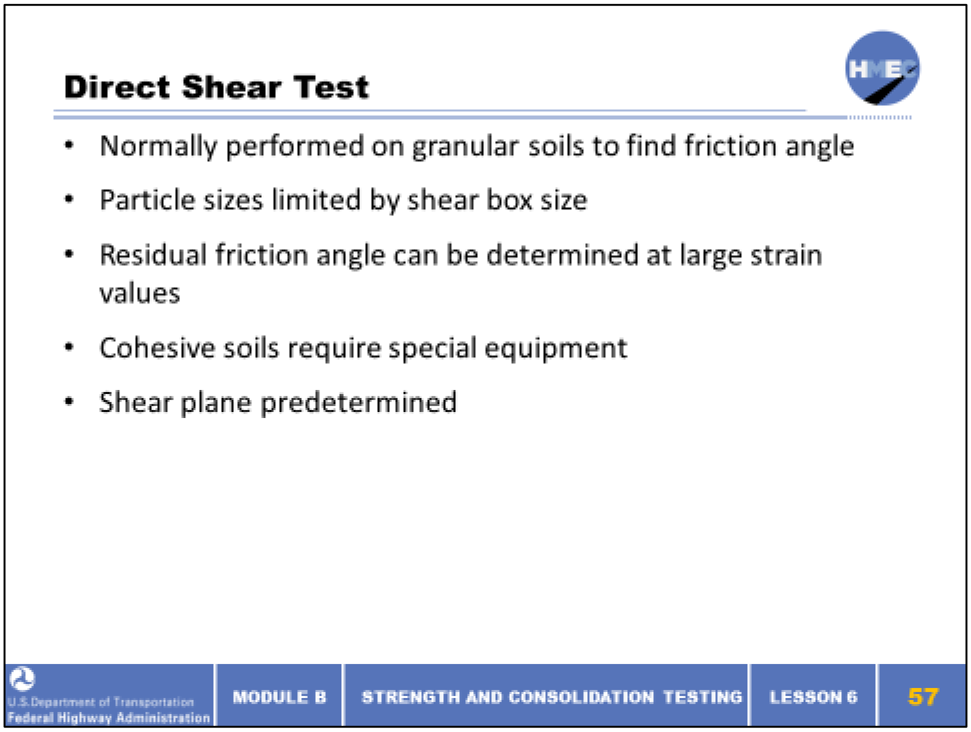
____

____

Slide 56

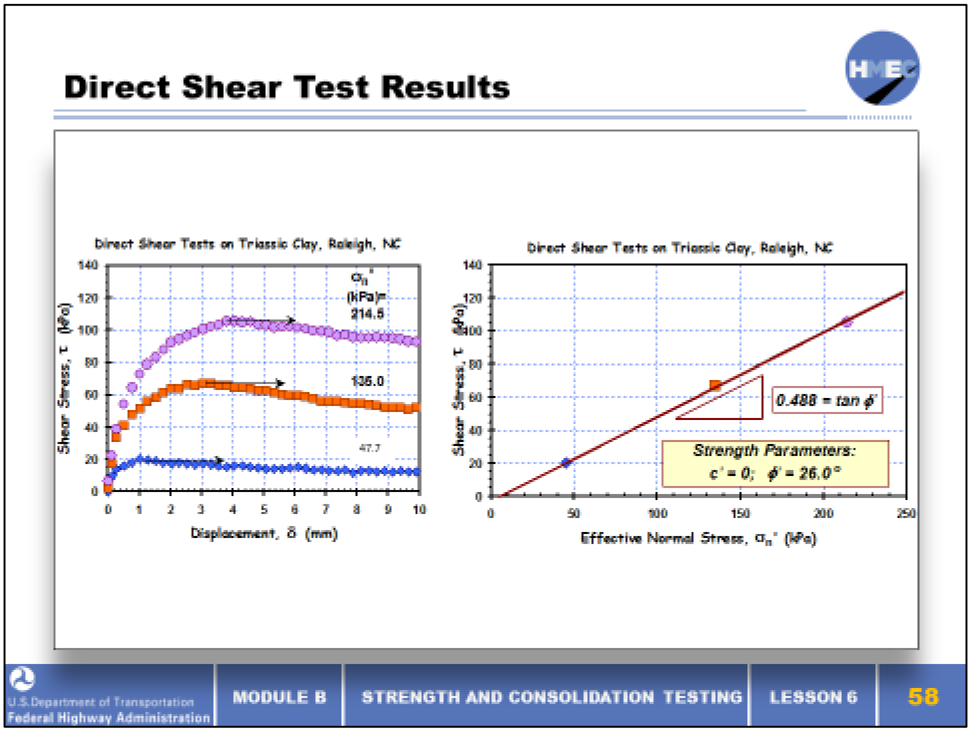
Here is a photo of the direct shear apparatus. The apparatus and procedures for direct shear testing are discussed in ASTM D3080. A specimen is prepared in a split square or circular box, and is sheared as one box is displaced horizontally with respect to the other using upper and lower loading frames. Load cells are used to monitor the shear force and LVDTs are used to monitor both horizontal and vertical deformation. Using this instrumentation, as well as a loading frame that provides a constant rate of horizontal deformation, it is possible to automate the direct shear test.

Slide 57



A version of this test can be performed on jointed rock to determine the shear strength of the discontinuity.

Slide 58



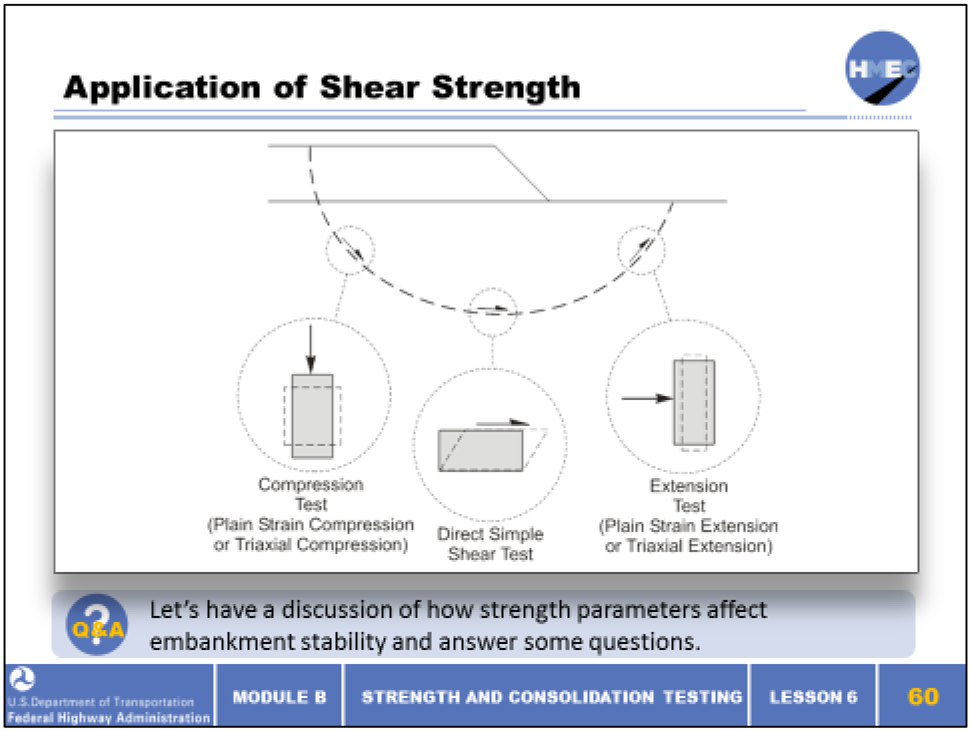
The test is performed on at least three samples at different normal stresses. The displacement and shear stress are measured and the peak shear stress is plotted versus the normal stress to obtain the failure envelope. Since the shear rate is controlled to allow full drainage, the parameters obtained are effective stress. The slope of the line is the tangent of ϕ'.

Slide 59

The DSS test was developed in an attempt to refine the direct shear test by providing shear strain distortion, rather than horizontal displacement. Studies at the Massachusetts Institute of Technology (MIT), the Norwegian Geotechnical Institute (NGI), the Swedish Geotechnical Institute, and Politecnico di Torino have concluded that the DSS provides the most representative mode for the mobilized undrained strength in stability analyses involving embankments, footings, and excavations in soft ground. (ASTM D6528)

The DSS test has been found to be a good overall representation of shear strength along a roughly horizontal failure plane, which is applicable to many loading conditions in situ (e.g. slope stability, bearing capacity, etc.). In addition, values of undrained shear strength from DSS tests are between values measured using triaxial compression and extension tests (Ladd and Degroot, 2003)

Slide 60


Application of Shear Strength
HMEC
Compression
Test
(Plain Strain Compression
or Triaxial Compression)
Direct Simple
Shear Test
Extension
Test
(Plain Strain Extension
or Triaxial Extension)
Q&A
Let's have a discussion of how strength parameters affect
embankment stability and answer some questions.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRENGTH AND CONSOLIDATION TESTING
LESSON 6
60

Slide 61

Learning Outcomes Review

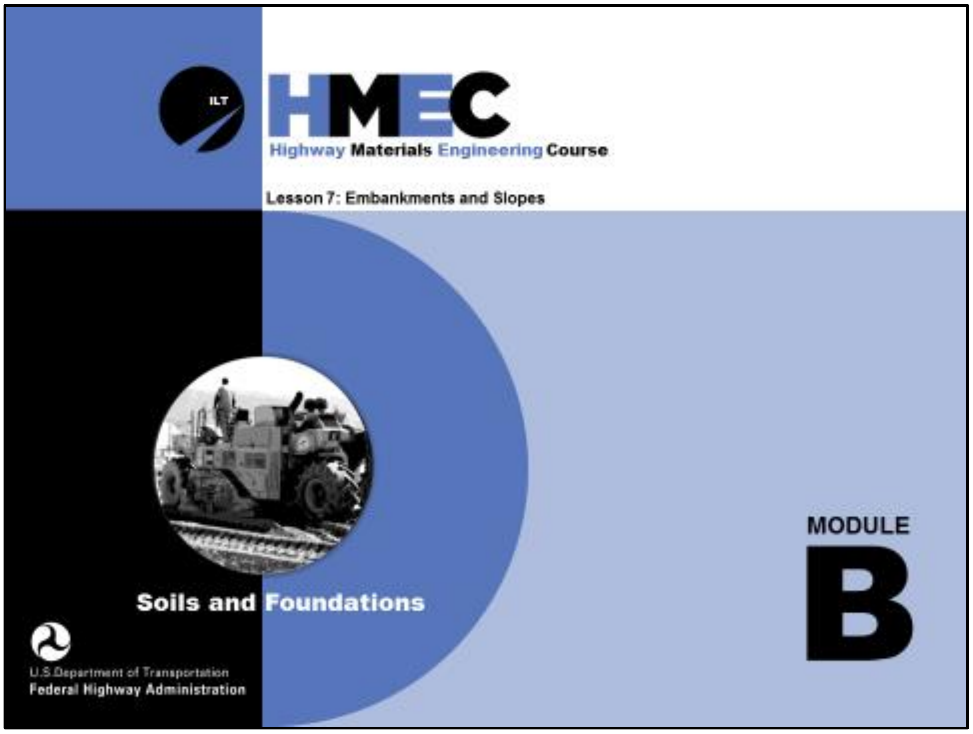
You are now able to:

- Explain the soil phase system
- Explain the importance of sample preparation
- Apply the concept of sampling and testing variation to geotechnical design
- Explain the effect of water on soil strength as determined by computing and plotting total and effective overburden and water pressure
- Describe consolidation and shear strength test methods and applications

U.S. Department of Transportation Federal Highway Administration | MODULE B | STRENGTH AND CONSOLIDATION TESTING | LESSON 6 | 61

This page is intentionally blank.

Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 7: Embankments and Slopes
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2

Learning Outcomes

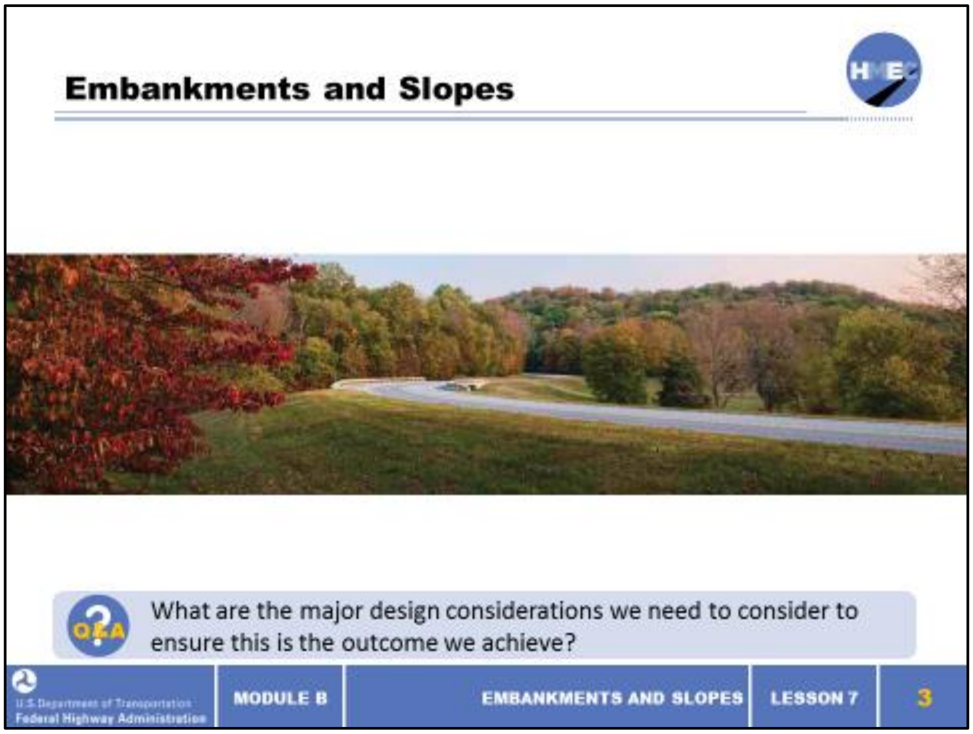
By the end of this lesson, you will be able to:

- Describe the process for determining the factor of safety
- Describe potential failure modes in soil and rock
- Using given criteria, discuss possible solutions to slope stability problems
- Using given criteria, discuss possible solutions to embankment settlement problems
- List two rock slope stabilization techniques
- Describe best practices in construction monitoring methods for stability and settlement

This lesson will take approximately 3 hours to complete.

U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 2

Slide 3



Note the slopes and embankment in this photograph. What are the major design considerations we need to consider to ensure this is the outcome we achieve?

____

____

____

____

____

____

____

____

____

____

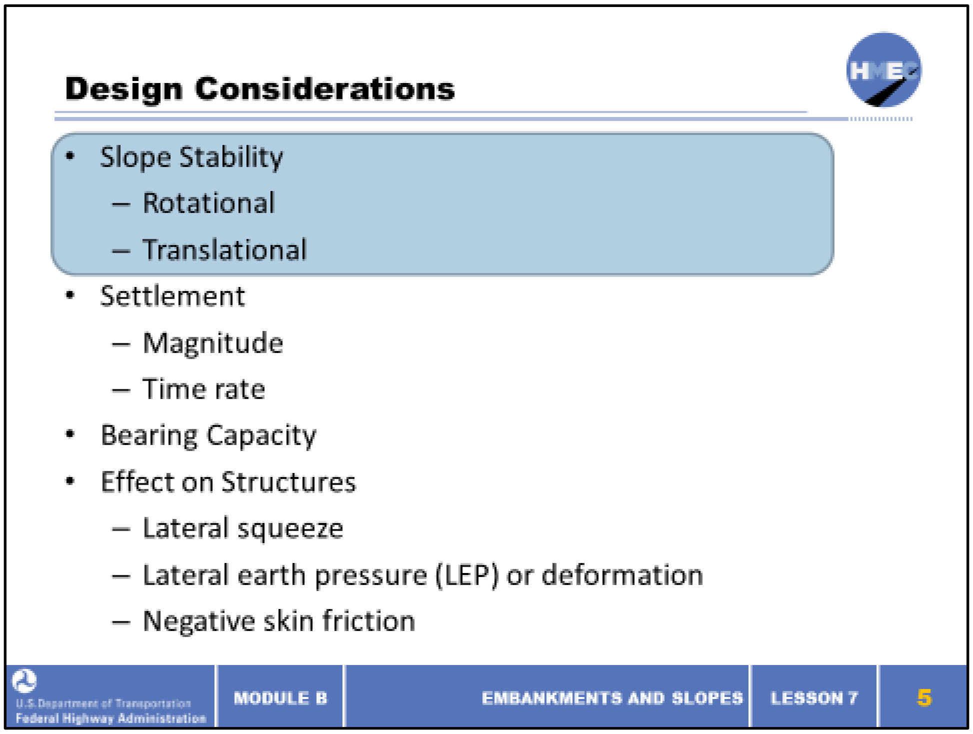
Slide 4

Design Considerations

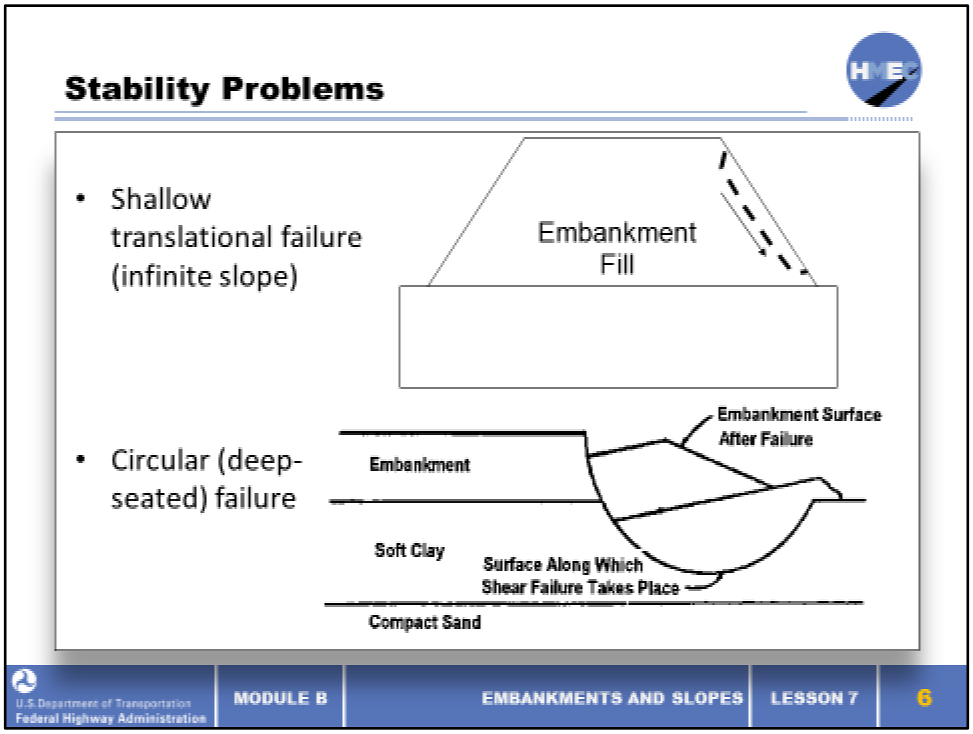
- Slope Stability
  - Rotational
  - Translational
- Settlement
  - Magnitude
  - Time rate
- Bearing Capacity
- Effect on Structures
  - Lateral squeeze
  - Lateral earth pressure (LEP) or deformation
  - Negative skin friction

U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 4

Slide 5


Design Considerations
• Slope Stability
– Rotational
– Translational
• Settlement
– Magnitude
– Time rate
• Bearing Capacity
• Effect on Structures
– Lateral squeeze
– Lateral earth pressure (LEP) or deformation
– Negative skin friction
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
5

Slide 6



"Internal" embankment stability problems generally result from the selection of poor quality embankment materials and/or improper placement of the embankment fills and/or improper placement requirements. The shallow translational failure or slough is probably the most irritating. It seems to be a problem every spring when the rains come. Although they can be deep and extensive, they are mostly shallow and a maintenance headache. They are primarily the result of our inability to get good compaction in the last 3 to 4 ft. near the embankment slope. Geosynthetic reinforcement placed near the slope has proven effective in preventing these failures.

The deep-seated circular failure is more catastrophic and causes major damage to the facilities and can result in fatalities. There are many causes for this type of failure, including inadequate design and poor construction practices.

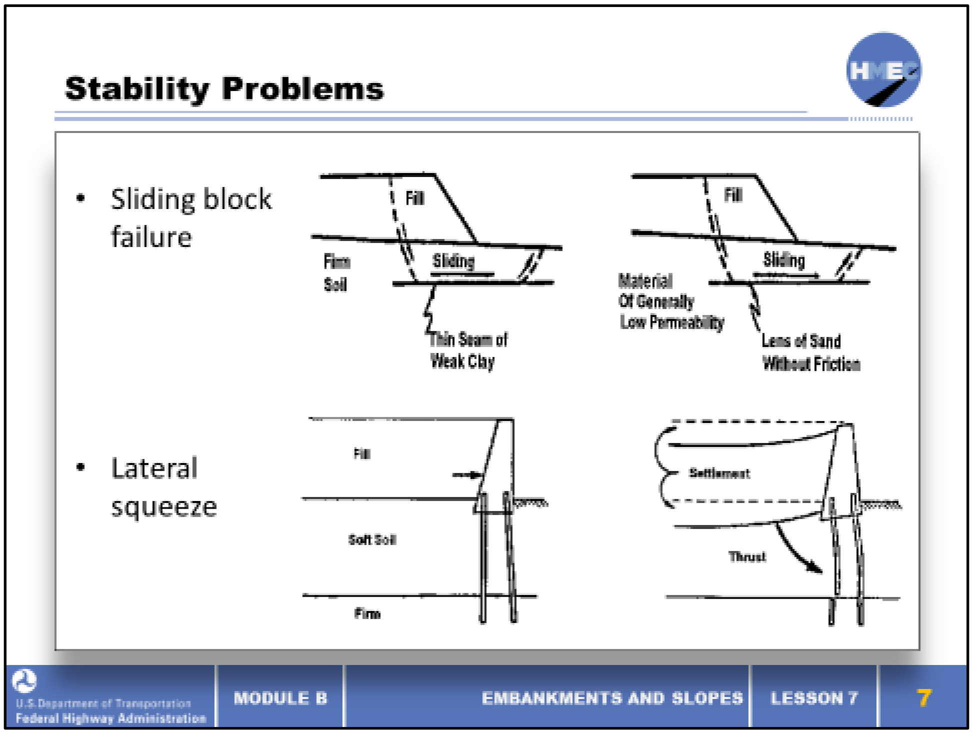
____

____

____

____

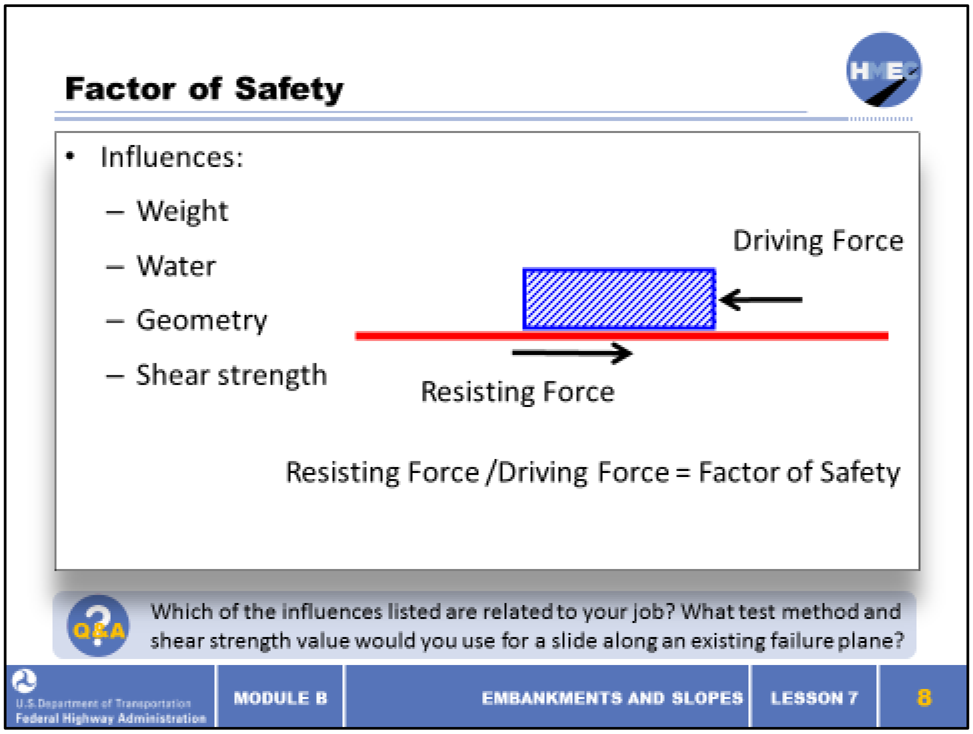
Slide 7



Most slides are translational in nature. This linear movement is especially true when the slide is shallow. The sliding surface is usually shallow and along a plane of weakness (a thin weak layer or stratum boundary). Therefore, translational slides are most common in thinly bedded soil slopes with soft clays, fine sands, or loose non-plastic silts.

Lateral squeeze of the foundation soils can occur if the soils are soft and if their thickness is less than the width of the end slope of the embankment. Consolidation settlement and lateral squeeze are not an issue within embankment fills since coarse-grained soils placed under controlled compaction conditions are generally used.

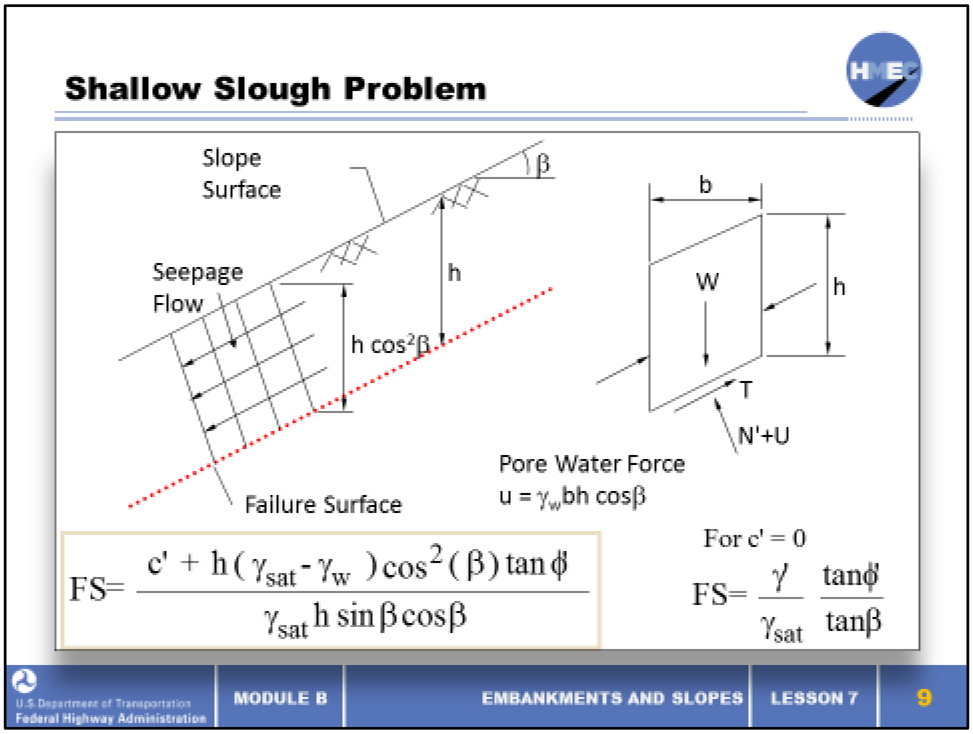
Slide 8



This is a simplified depiction, but all four of the stability problems involve driving forces that act to create failure and resisting forces that, if large enough, prevent failure.

____
____
____
____
____
____
____
____
____
____

Slide 9



How do we determine the FS? Let's take a look at the shallow slough problem first.

Concentrate on the formula to the right. Participants don't need to calculate FS, just recognize what influences the FS. Discuss the influence of water on stability. Note that for dry soil, the FS is dependent only on the friction angle and the slope angle. If the slope is saturated with flow down the slope, the FS is then reduced by about half of the ratio of effective unit weight/saturated unit weight (say 62/125), which decreases the mobilized soil shear strength that resists failure. If the slope material has some cohesion, that will improve the FS; however, we have to consider that cohesion may not be a reliable soil parameter for long-term analysis of stability.

____

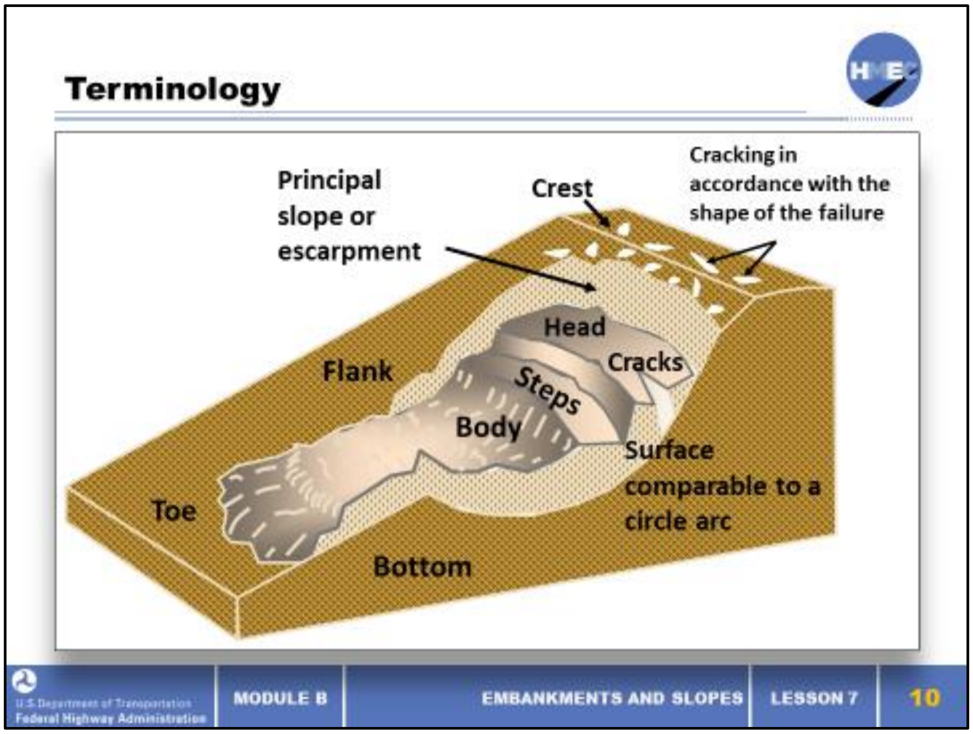
____

____

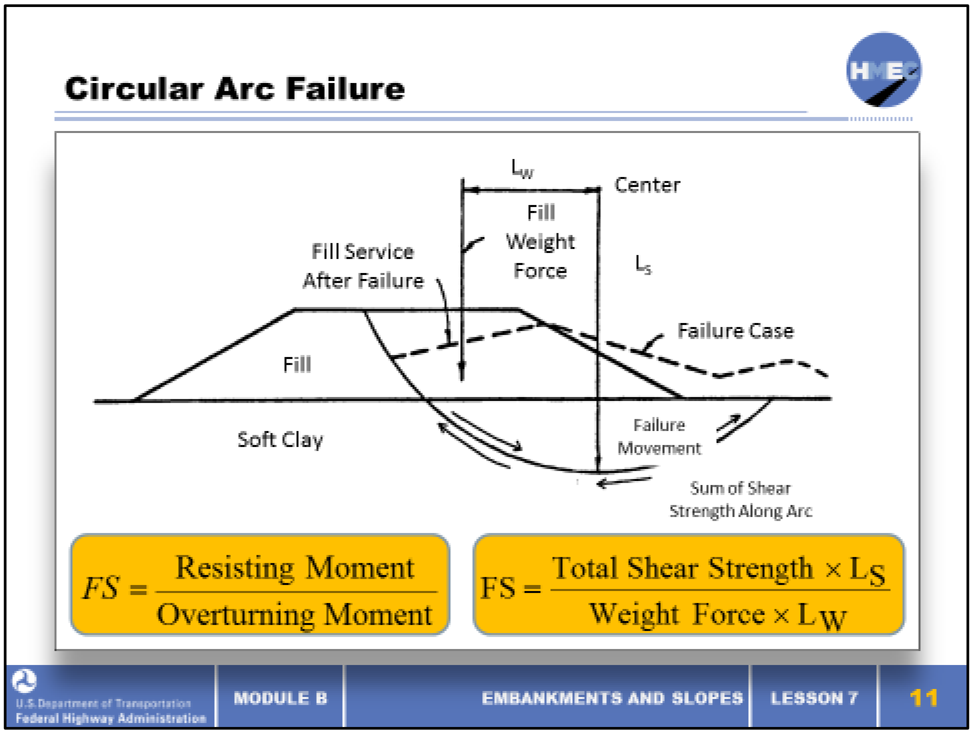
____

____

Slide 10


Terminology
Principal slope or escarpment
Crest
Cracking in accordance with the shape of the failure
Head
Cracks
Flank
Steps
Body
Surface comparable to a circle arc
Toe
Bottom
U.S. Department of Transportation Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
10

Slide 11



How do we determine the FS we want to design for to minimize the potential for a circular arc failure?

This FS is a bit more difficult to determine than the one for a shallow slough. However, all of the same influences still apply. What are those influences? Weight, water, geometry, and shear strength.

____

____

____

____

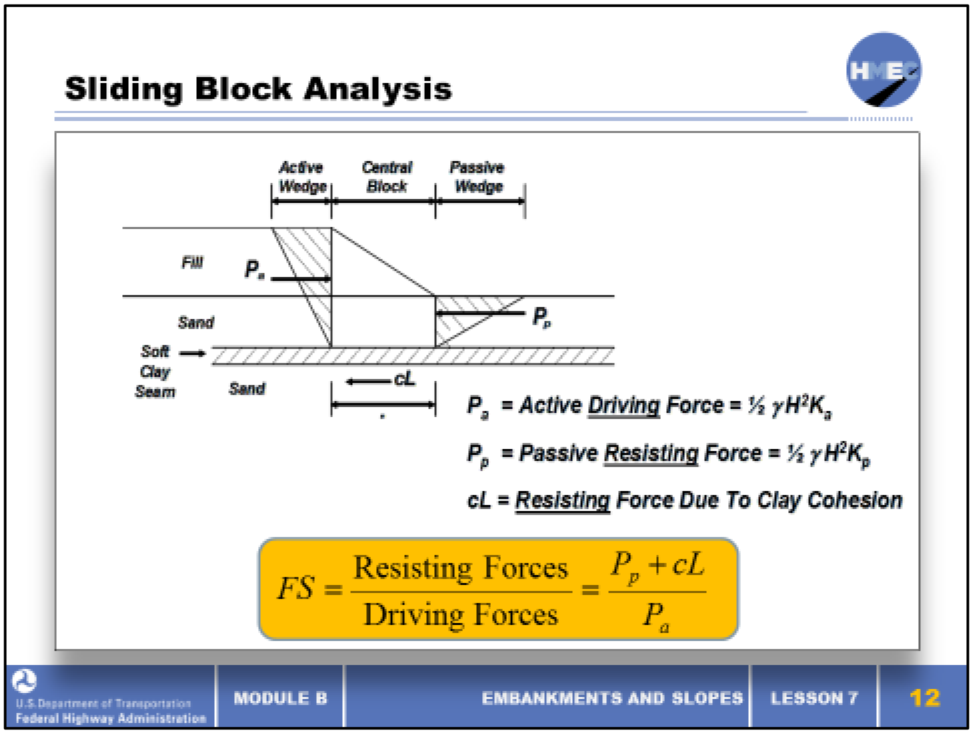
____

____

____

____

Slide 12



It's a bit easier to determine the FS in a translational or sliding block failure, since we don't need moments-only forces. However, other aspects of the FS are similar.

____

____

____

____

____

____

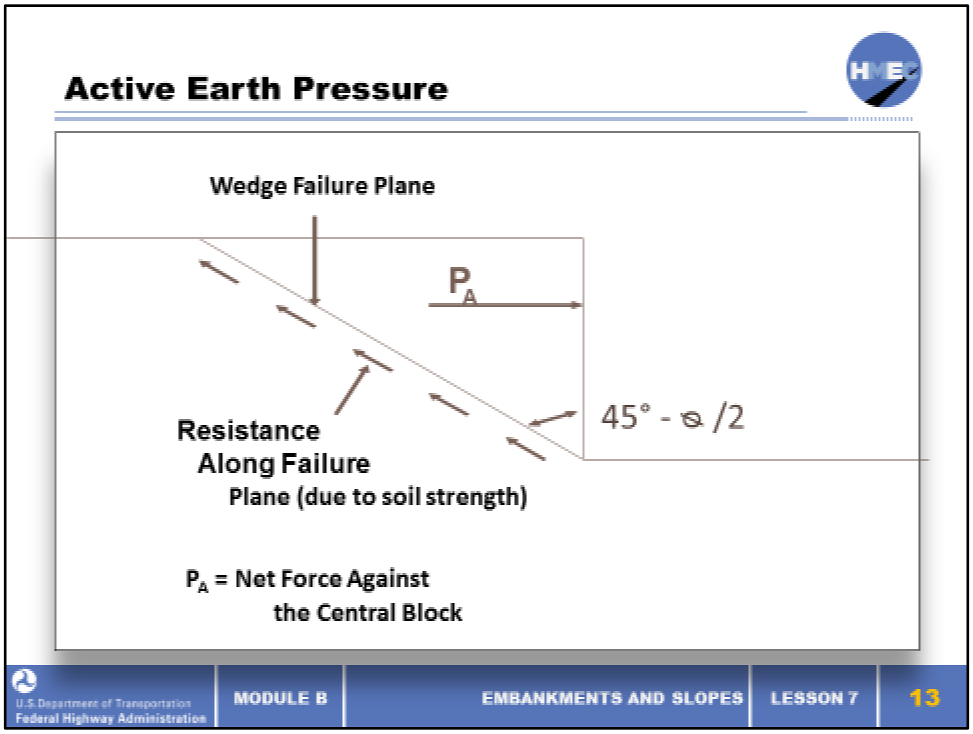
____

____

____

____

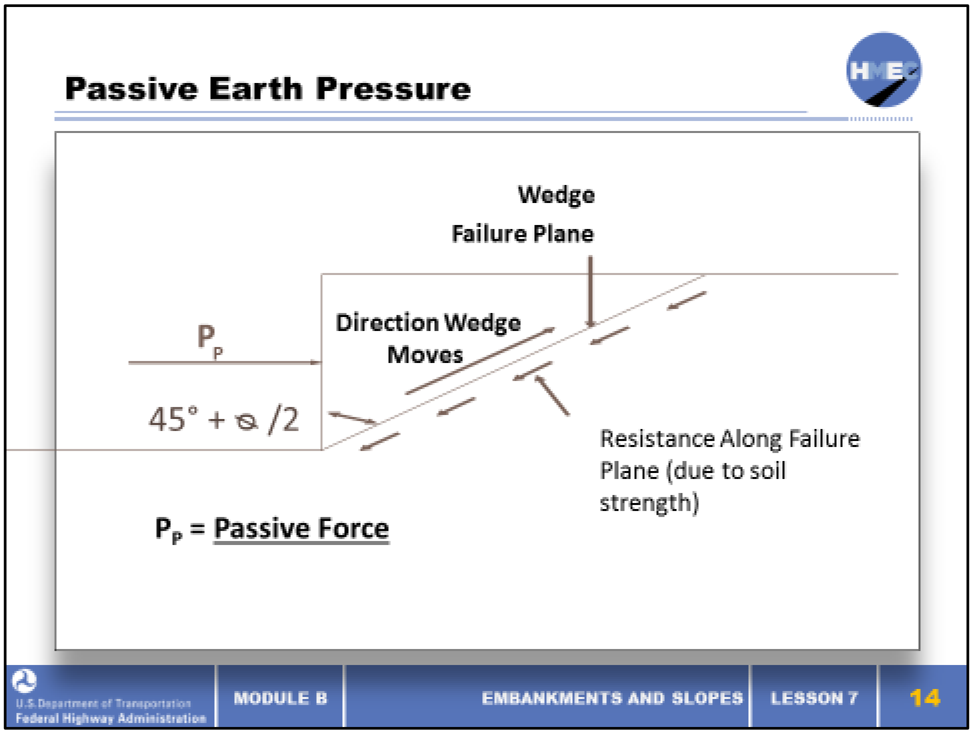
Slide 13



The basis of the active earth pressure concept is that the soil behind the central block shown on the previous slide (sliding block failure) or a retaining wall will try to fail in a wedge shape. The wedge creates the triangular pressure diagram against the block of soil or a wall depending on what we are analyzing. In the case of the sliding block analysis, the pressure diagram is applied to the central block. Note that the force against the central block is calculated the same way as for a wall analysis.

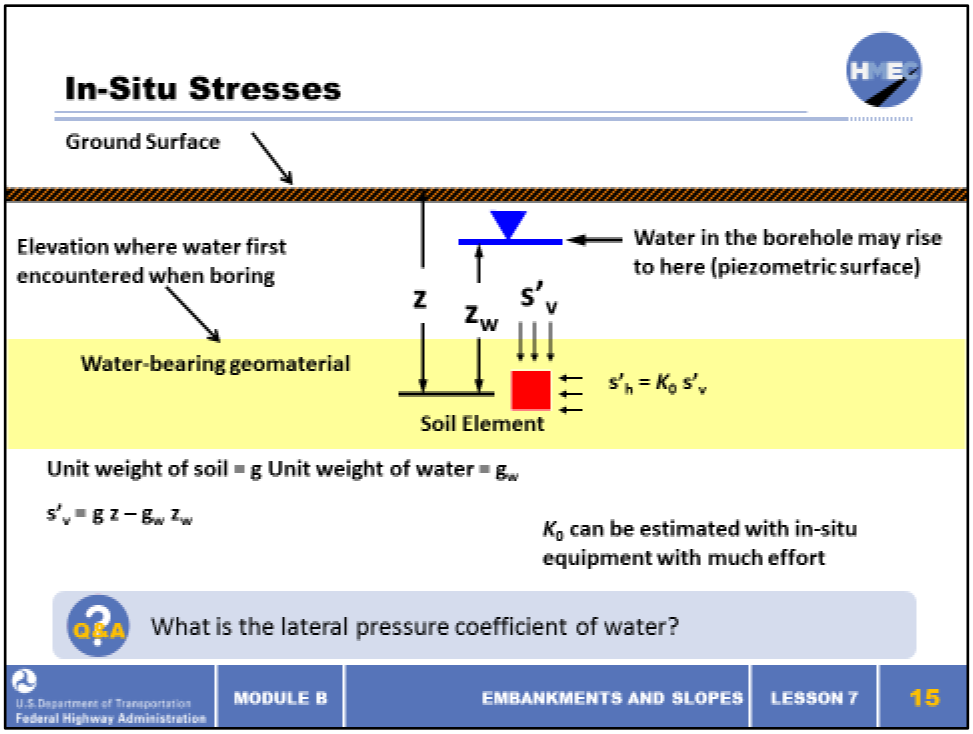
Note that the angle of the failure surface is directly related to the friction angle. The failure wedge is a function of the strength of the soil determined by our lab and/or field tests.

Slide 14



A similar explanation can be made for the passive pressure. In this case, the wall or central block must move the passive wedge up the failure plane before failure can occur.

Slide 15



This slide shows an element of soil below the ground. Its in-situ stress is determined by its depth, the weight of soil above it (overburden), the depth and weight of water above it, and the buoyancy of water. Remember we said that K_o is important in the design of many geotechnical features.

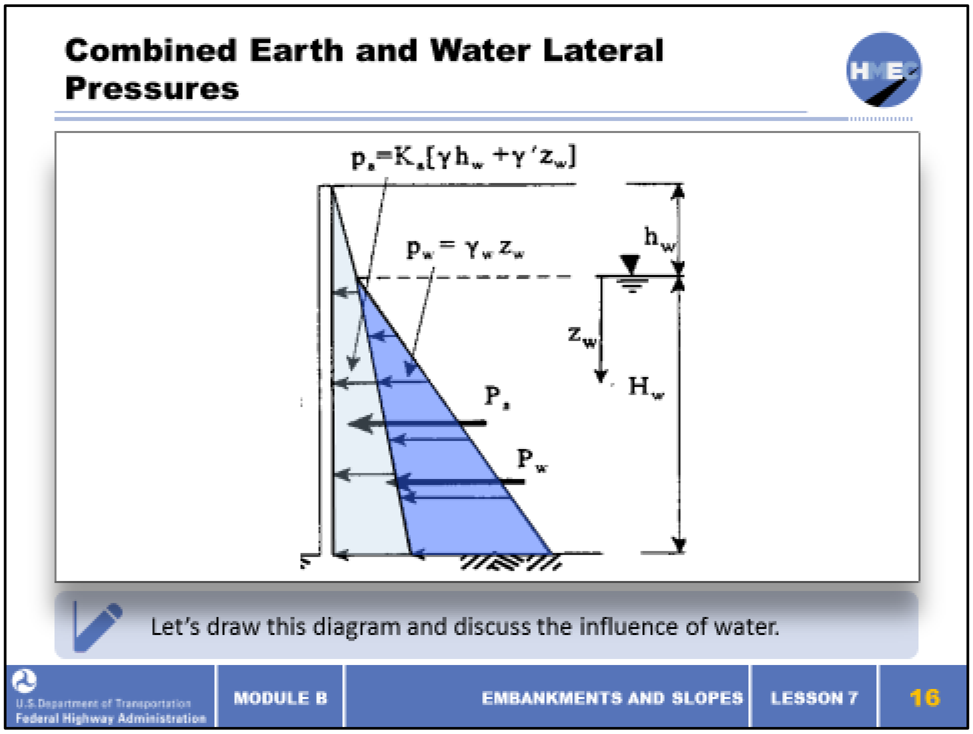
What we didn't mention before is that the value of K will vary depending on whether or not the soil is allowed to move. If the movement is away from the stable soil mass as depicted in the active wedge from the previous slides, K is called an active pressure coefficient and designated K_a. A common value would be about ⅓ for loose sand. If the force is pushing into the soil mass as depicted for the passive wedge on the previous slides, K is called a passive pressure coefficient and is designated K_p, which is typically > 1 and will be about 10 times the active pressure. Have students think back to the sliding block failure and how it is influenced by K.

K_o is call the at rest (no movement) coefficient. $K_a < K_o < K_p$.

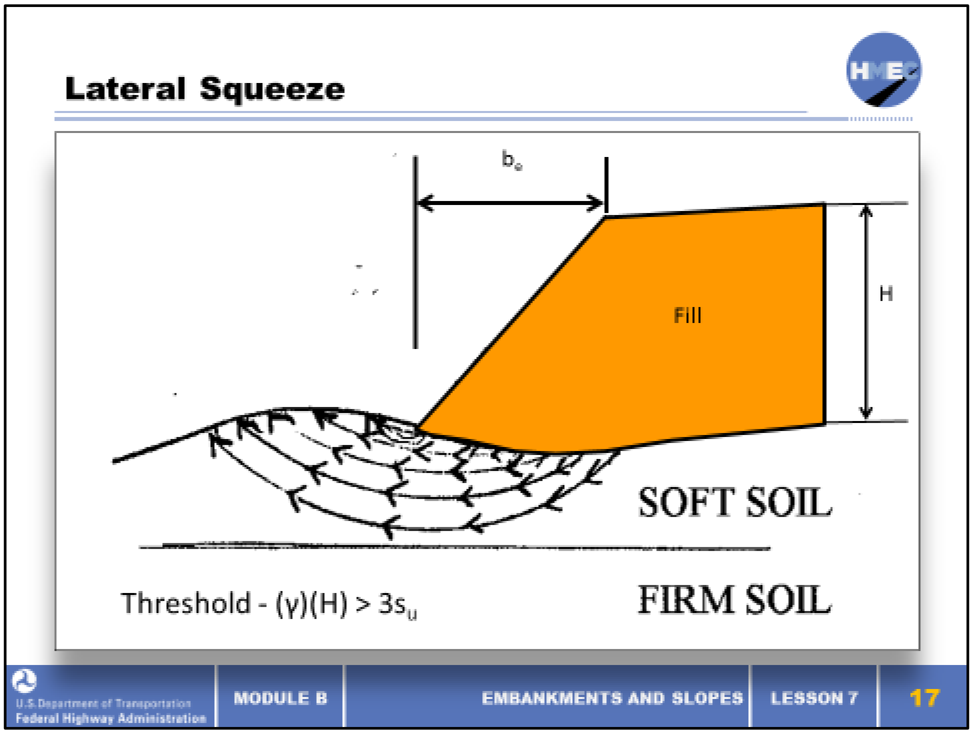
____

____

Slide 16


Combined Earth and Water Lateral Pressures
$p_a=K_a[\gamma h_w+\gamma' z_w]$
$p_w=\gamma_w z_w$
h_w
z_w
H_w
P_a
P_w
Let's draw this diagram and discuss the influence of water.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
16

Slide 17



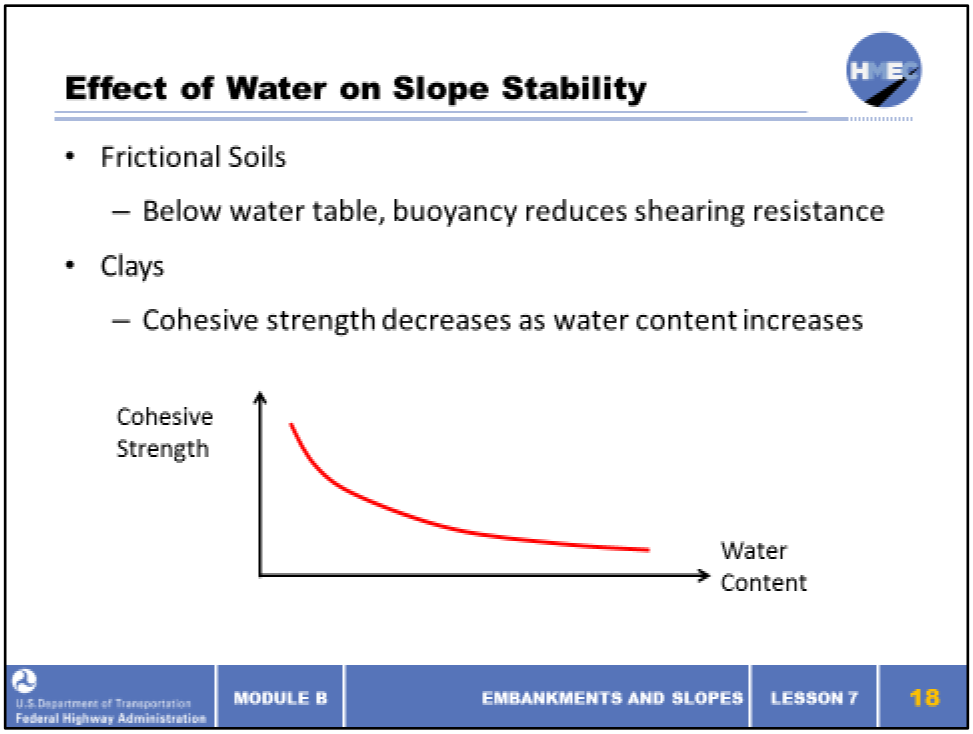
When the geometry of the applied load is larger than the thickness of the compressible layer, or when there is a finite soft layer within the depth of significant influence (DOSI) below the loaded area, significant lateral stresses and associated lateral deformations can occur. For example, if the thickness of a soft soil layer beneath an embankment fill is such that it is less than the width b_e of an end or side slope, then the soft soil may squeeze out.

Another example is: What if your hand (an embankment) is placed on a tube of toothpaste (soft soil). If enough pressure is applied, that soft soil (toothpaste) will flow outward in a lateral squeeze.

The rule of thumb to determine if lateral squeeze is a possibility is: If (γ) H > 3S_u then further analysis is required. S_u = the undrained shear strength of the soft layer.

Note that the lateral squeeze phenomenon is not related to consolidation but is more related to undrained shear failure or local bearing capacity failure.

Slide 18



We can mostly agree that water is a good thing when we want a cold drink or a cool dip in the pool, but hopefully you have begun to recognize that it is not a good thing when dealing with many geotechnical problems. Besides the increase in load caused by water, let's discus some other effects water has on slope stability.

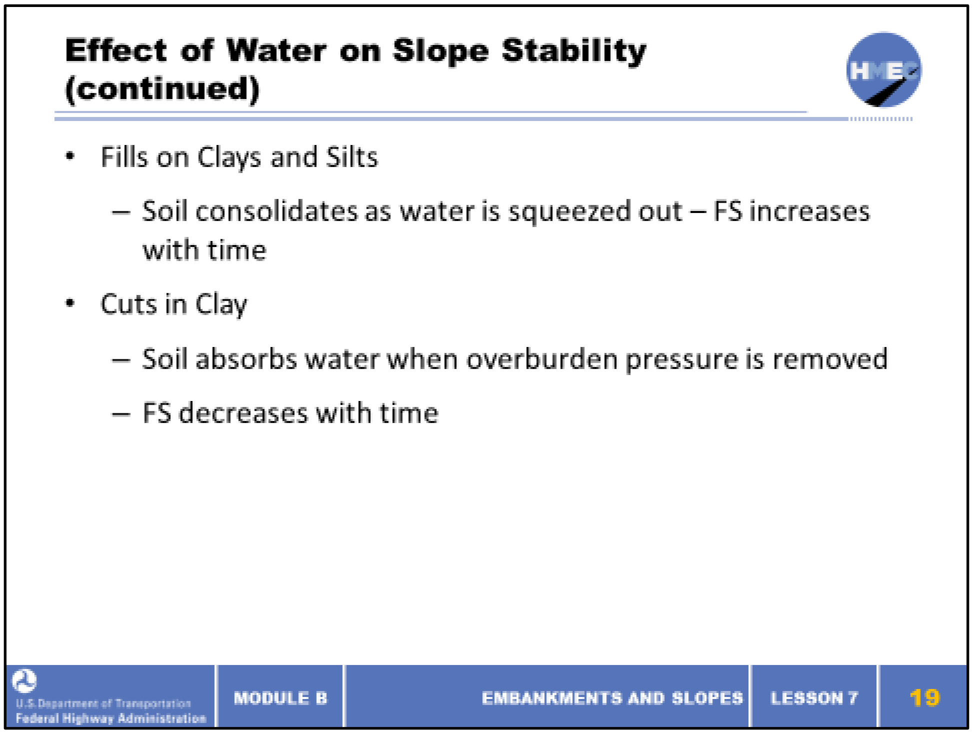
Next to gravity, water is the most important factor in slope stability. The effect of gravity is known, therefore, water is the key factor in assessing slope stability.

Very soft, saturated foundation soils or groundwater generally play a prominent role in geotechnical failures in general. They are certainly major factors in cut slope stability and in the stability of fill slopes involving both “internal” and “external” slope failures.

Because shear strength in frictional soils (sands and gravels) depends on normal stress and soils below the water table have an effective unit weight (reduced by γ_w), their shear strength is less.

Water reduces the long-term strength of cohesive soil. In addition, if the cohesive soil has cracked due to drying and those cracks later fill with water, increased loads and loss of strength may occur.

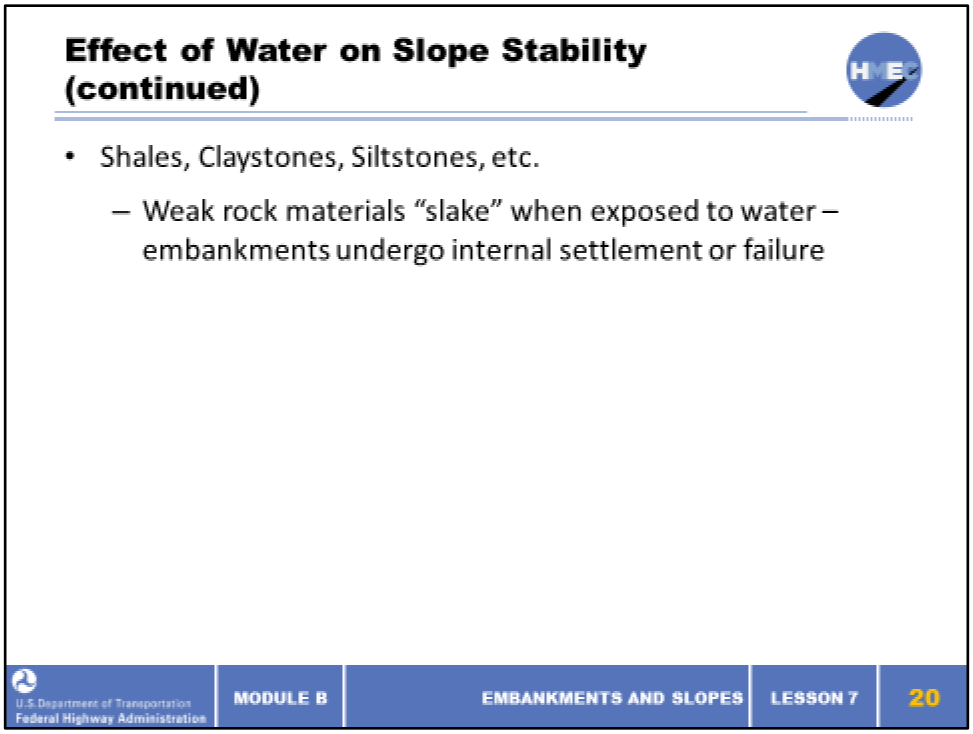
Slide 19



The excess pressure causes the pore water to drain and the intergranular stress causes consolidation and an increase in shear strength. That increase in shear strength during consolidation is what allows stage construction over unstable ground to be a viable construction technique. We will discuss stage construction later.

When a roadway cut is made, it removes the overburden, thereby reducing the in-situ stress and the clay in the cut absorbs water. The increase in moisture content reduces the shear strength and increases the driving force (weight), resulting in a reduced FS and possibly failure.

Slide 20



The selection of fill material is important to the long-term stability of the embankment, especially when shales, claystones, or siltstones are considered.

Slide 21

Aspects of Stability for Cut Slopes

IMPORTANT ASPECTS OF THE STABILITY OF EXCAVATION SLOPES (Duncan et al., 1987)		
	Soil Type	
	Cohesionless	Cohesive
Factors that control stability	ϕ' of fill Slope angle Pore pressures External water	Strength of soil γ of soil Slope angle Pore pressures External water
Failure mechanism	Surface raveling	Deep sliding, possibly extending below toe of slope
Special problems	Surface erosion Liquefaction during earthquake	Strength loss in stiff-fissured clays surface erosion
Critical stages for stability	Long-term or earthquake	End-of-construction, long-term, or rapid drawdown
Analysis procedures	Effective stress or dynamic	Total stress, effective stress, or combination

U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 21

Slide 22

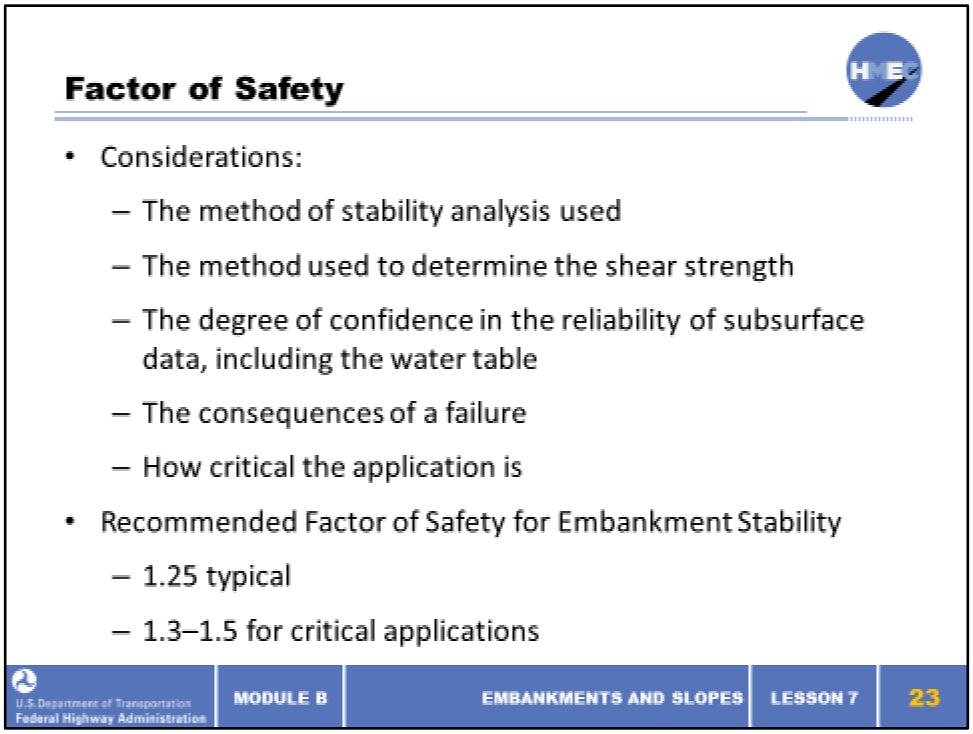
Aspects of Stability Embankments

IMPORTANT ASPECTS OF THE STABILITY OF COMPACTED FILLS (Duncan et al., 1987)			
	Type of Fill and Foundation		
	Cohesionless fill on firm foundation	Cohesive fill on firm foundation	Any type of fill on weak foundation
Factors that control stability	ϕ' of fill Slope angle Pore pressures External water	Strength of soil γ of soil Slope angle Pore pressures External water	Strength of foundation; Depth of weak foundation layer; Strength of fill γ of fill; Height of fill; Slope angle; Pore Pressures; External water
Failure mechanism	Surface raveling	Sliding tangent to top of foundation	Deep sliding extending into foundation
Special problems	Surface erosion Liquefaction during earthquake	Surface erosion weathering and weakening of compacted shales	Embankment cracking Progressive failure Surface erosion
Critical stages for stability	Long-term or earthquake	End-of-construction, long-term, or rapid drawdown	End of construction, long-term, or rapid drawdown
Analysis procedures	Effective stress or dynamic	Total stress, effective stress, or combination	Total stress, effective stress, or combination

U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 22

Embankment fills over soft clay foundations are frequently stronger and stiffer than their foundations. This leads to the possibility that the embankment will crack as the foundation deforms and settles under its own weight, and the possibility of progressive failure because of stress-strain incompatibility between the embankment and its foundation. Settlement of the embankment material must also be considered during the design and is a function of the material used to construct the embankment, the density that the material is placed at, the moisture content of the fill, and the loads imposed on the embankment.

Slide 23



The selection of the FS is based on the level of uncertainty and the how critical the geotechnical feature is. Discuss each of the considerations and how they influence the selection of the FS. For example, some methods of analysis provide more reliable results, certain strength tests provide more consistent and dependable results, a limited subsurface investigation may warrant a higher FS, a bridge berm or a retaining wall slope may warrant a higher FS, etc.

Slide 24

As a cut is made in clay, the effective stress is reduced. This reduction will allow the clay to expand and absorb water, which will lead to a decrease in the clay strength with time. For this reason, the factor of safety of a cut slope in clay may decrease with time. Cut slopes in clay should be designed by using effective strength parameters and the effective stresses that will exist in the soil after the cut is made.

Slide 25

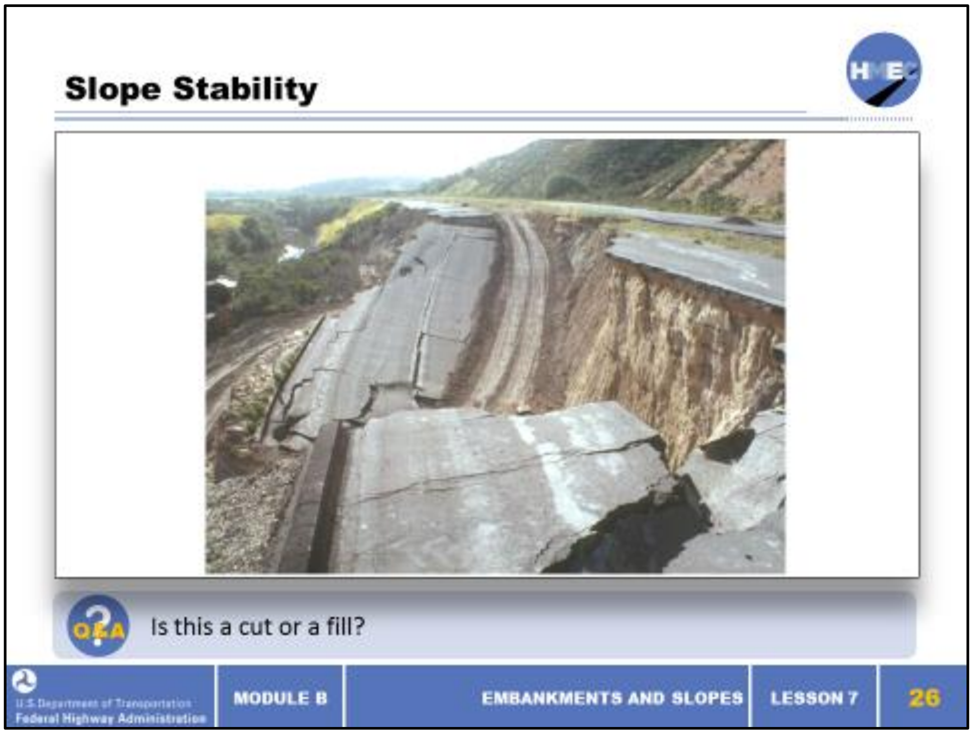
Case Study Notes: NHI Course-132035, Rock Slopes RM: This image shows rock fall of very strong, massive granite that closed the highway. The size of these blocks depends on the strength of the rock, such that blocks are not broken up as they fall down the slope from a height of about 150 m, and the blocks are large because the joints are widely spaced. As shown by the date stamp, the rock fall occurred on November 11. In the northwest, heavy winter rainfall starts in late October and early November.

A concrete barrier, fragments of which are visible in the picture, was destroyed by the fall. Rigid barriers are not suitable for containing rock falls, a topic which is discussed later.

Location: Trans-Canada Highway about 24 km north of Hope, British Columbia.

Rock type: Massive, very strong granite.

Slide 26



This is a rotation or circular arc failure. Notice the head scarp we mentioned on the terminology slide.

____

____

____

____

____

____

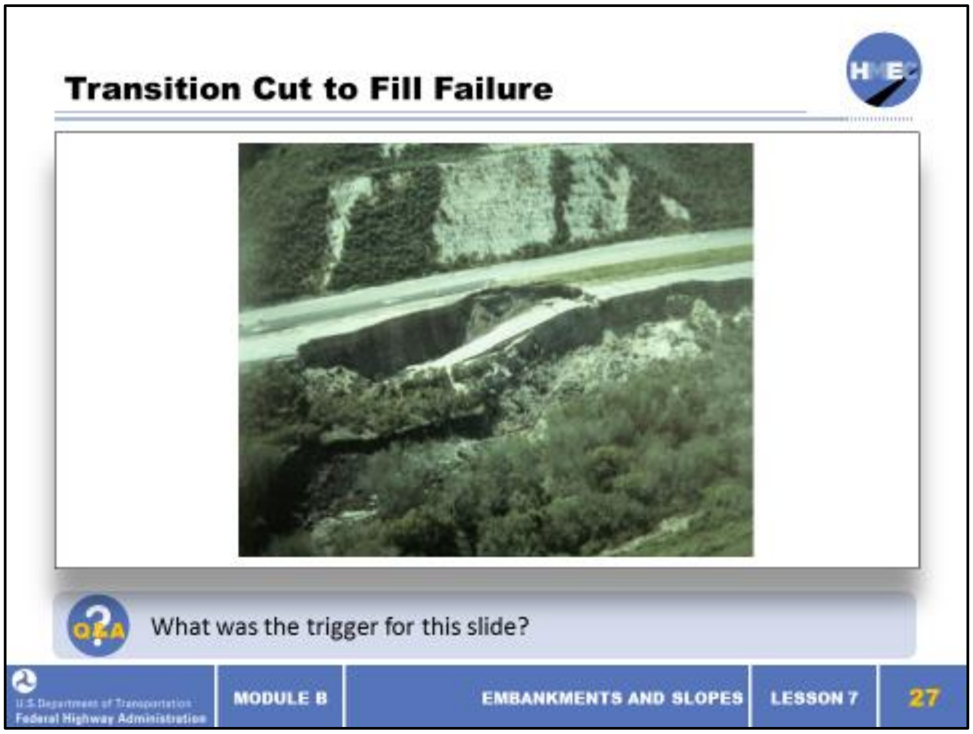
____

____

____

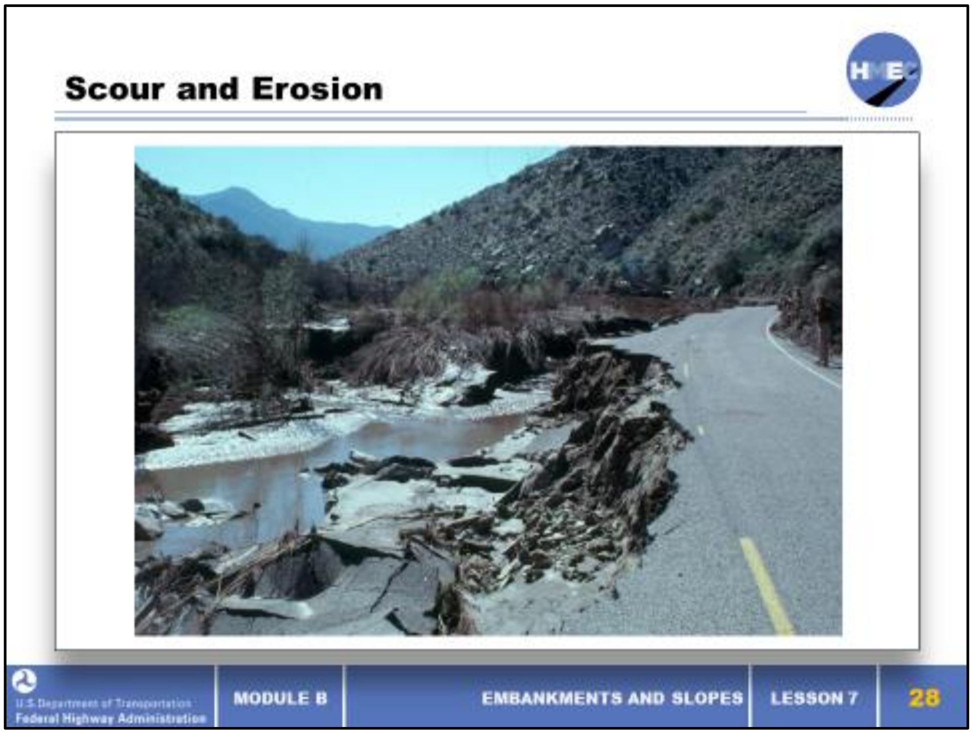
____

Slide 27



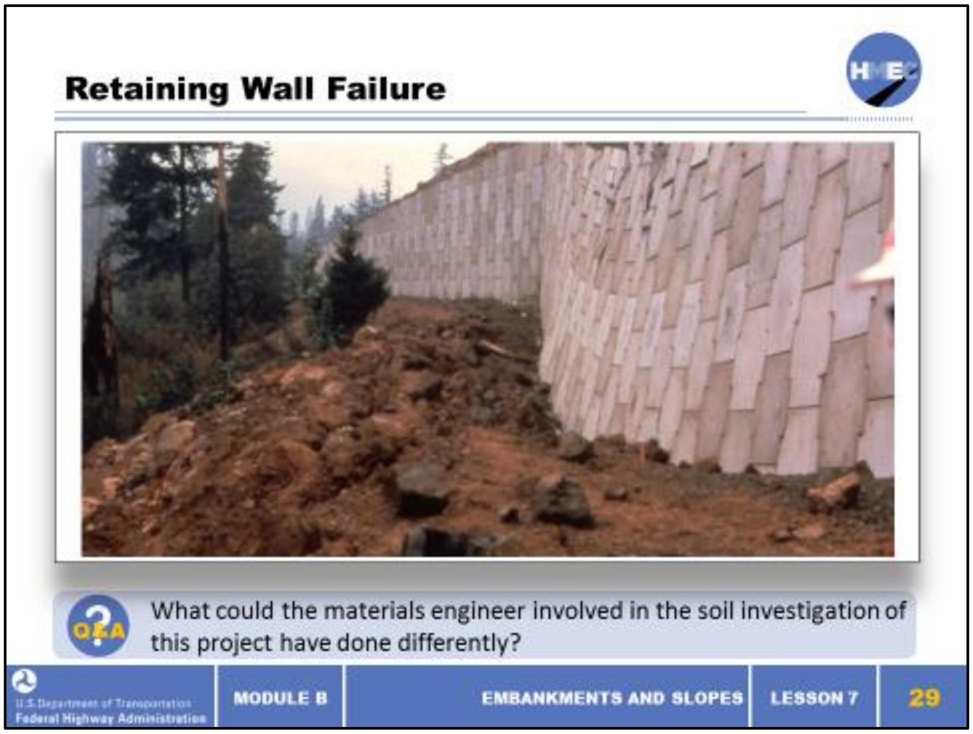
This embankment did not have a chance! It's at the transition from a cut to a fill, at its toe is a stream that floods periodically (just upstream, to the left, a previous rock slope protection repair was completed, but stopped short of protecting this area) and there is a permeable layer in the cut slope, which carries water to the cut/fill transition area. Note the vegetation on the cut slope across the roadway from the slide.

Slide 28



Case Study Notes: Reference FHWA/NHI-5-123, Soil Slope and Embankment Design (NHI course 132033): Scouring of an embankment due to flooding of adjacent river has resulted in over-steepening of the side hill fill and the subsequent instability, due to a loss of resistance at the slope toe. Slope protection needs to be designed to prevent reoccurrence of this embankment failure.

Slide 29



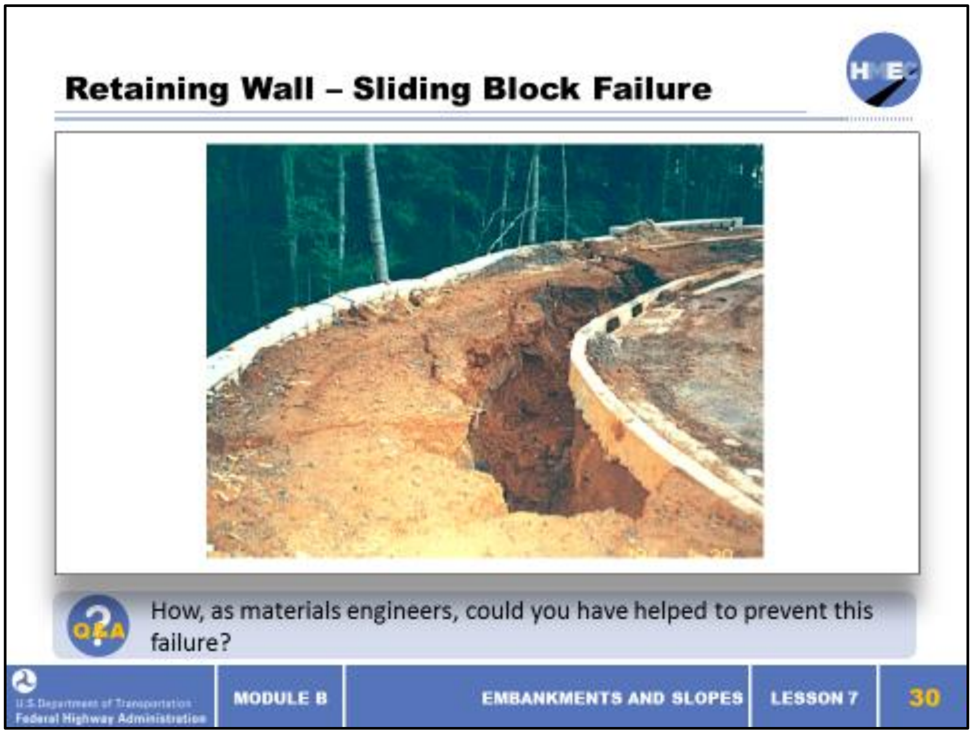
Retaining walls are structures that are very steep (vertical) embankments**.** We design them to last for 75–100 years. Slope instabilities can cause problems for retaining walls.

The case history for failure of reinforced earth wall in Coos Bay, Oregon. This failure was due to a sliding block failure. This wall was built to prevent a sliver fill section for a road widening from spilling into a river. The failure occurred during placement of a fill slope above the top of the wall. Also note how well the reinforced system withstood the failure movement. The wall actually prevented the failed mass from sliding into the river.

The cause of the failure was a thin seam of silty clay that was not found during the initial subsurface investigation.

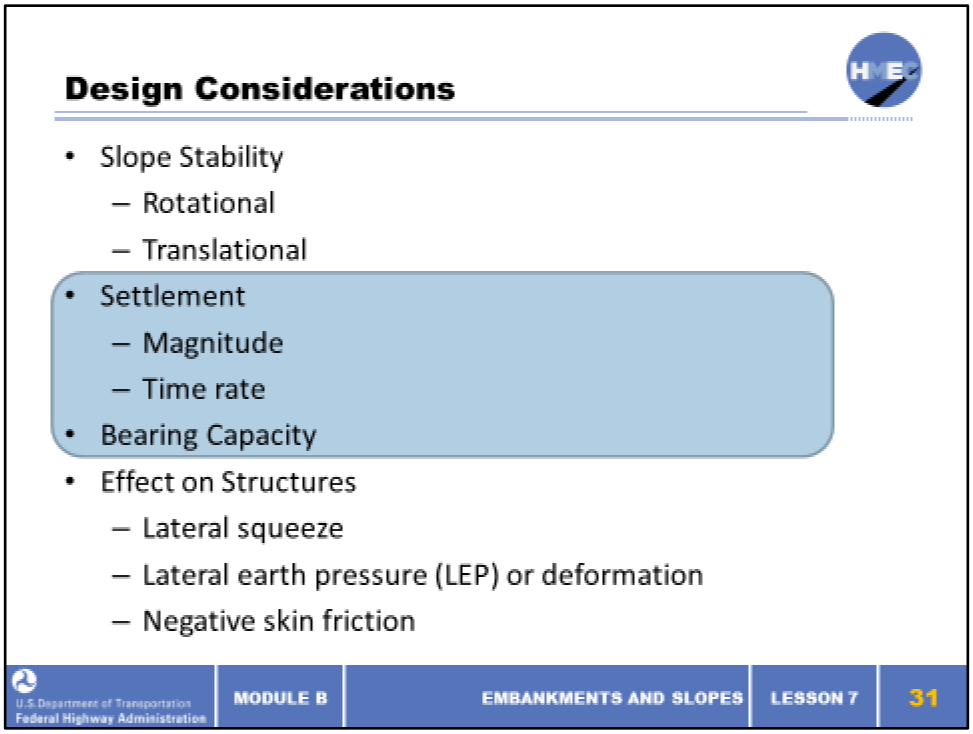
Case Study Notes: The Coos Bay, Oregon reinforced earth wall failure led to the common DOT policy with mechanically stabilized earth (MSE) walls that agencies are responsible for global stability of MSE walls, and vendors are only responsible for internal. Why? Because agency, not the MSE wall vendor, performs the subsurface investigation and interprets the results.

Slide 30

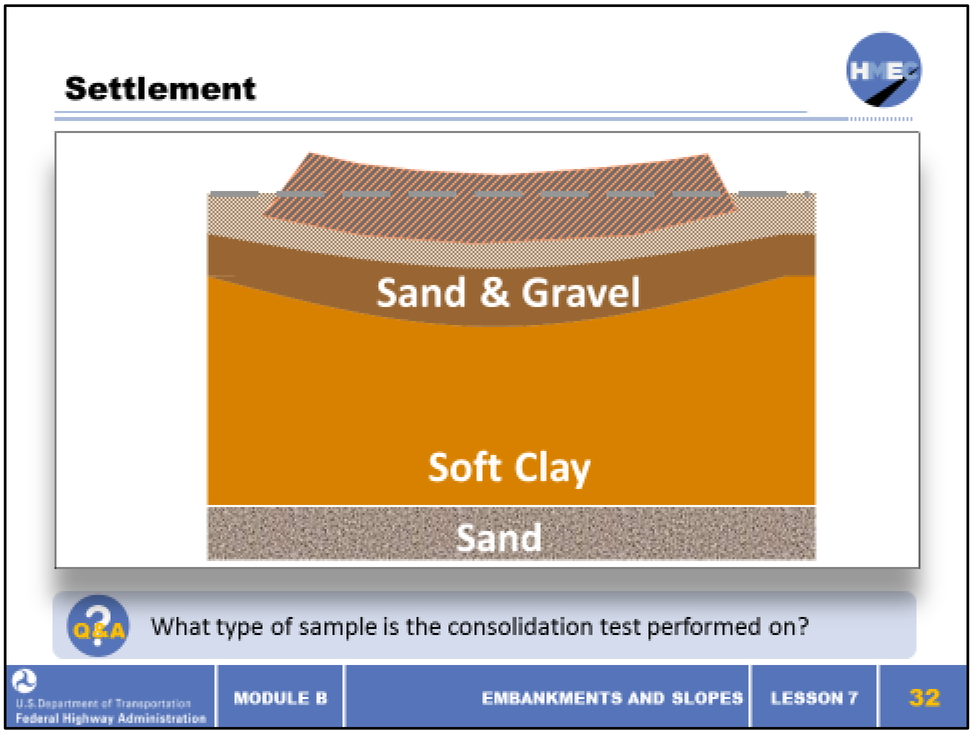


This is a translational or sliding block failure of a segmental retaining wall (SRW).

Slide 31

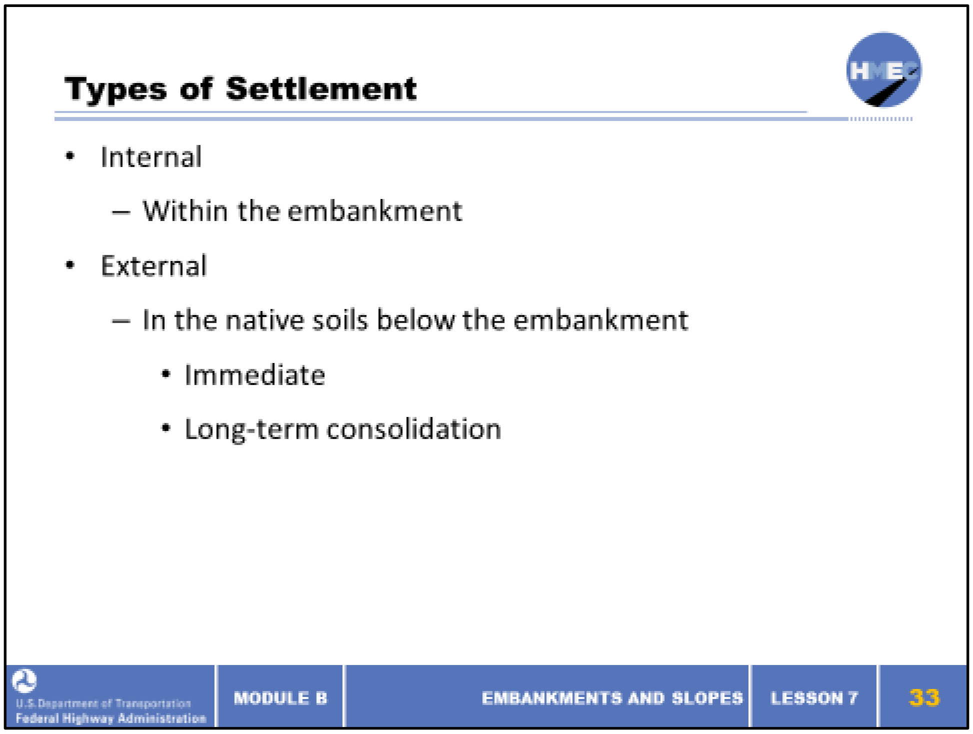

Design Considerations
HMEC
• Slope Stability
– Rotational
– Translational
• Settlement
– Magnitude
– Time rate
• Bearing Capacity
• Effect on Structures
– Lateral squeeze
– Lateral earth pressure (LEP) or deformation
– Negative skin friction
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
31

Slide 32



Settlement becomes an issue when it is necessary to build the roadway fill over a soft compressible subsurface layer.

Slide 33

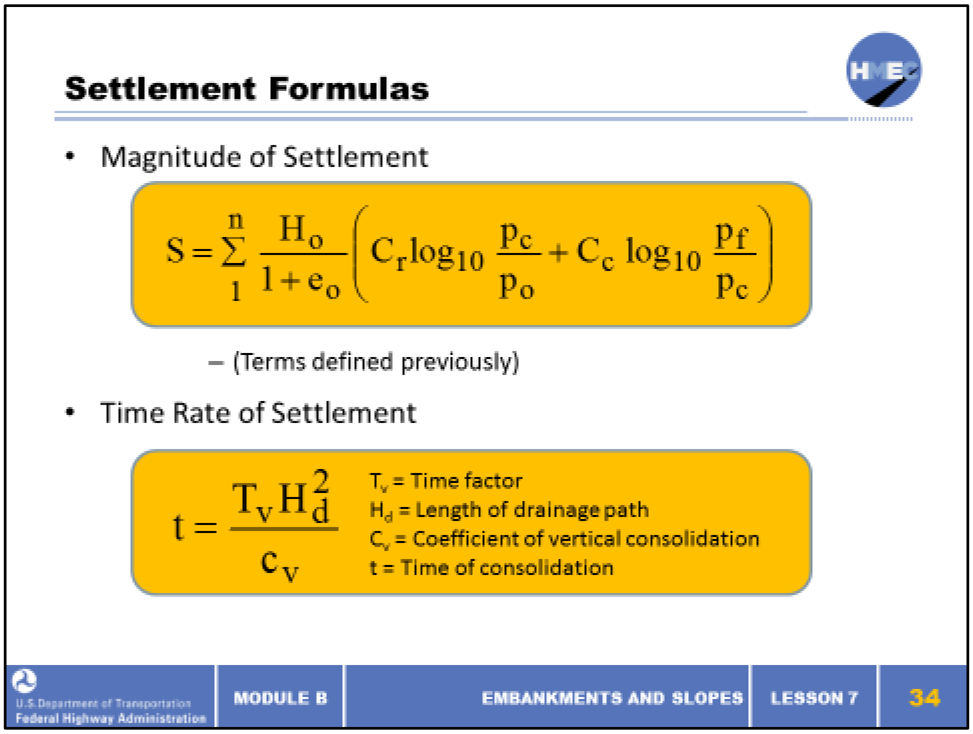


Internal deformation within embankments can be easily controlled by using fill materials that have the ability to resist the anticipated loads imposed on them. A well-constructed soil embankment will not excessively deform internally if quality control is exercised with regard to material and compaction. As a materials engineer, you will have a role in specifying the embankment material and compaction requirements.

As the name implies, immediate settlement takes place quickly, usually during the construction of the embankment, therefore it is not typically a problem. However, one instance where it can be a significant problem is when the embankment is being placed adjacent to an existing bridge. We will discuss that issue later. If the loading from the new embankment causes settlement in the soils supporting the existing bridge, settlement of the bridge can occur or additional load can be placed on the bridge foundation supports. The load is added through negative skin friction, which is a phenomena whereas the soil settles it pulls down on piling.

Compressible layers in the native soils will result in long-term settlement or consolidation, which is a concern both in design and construction. It is influenced by several factors, can cause stability issues, and can impact both new and existing structures.

Slide 34



The formulas used to calculate both the magnitude and the time rate of settlement, so we can discuss what influences those settlement issues.

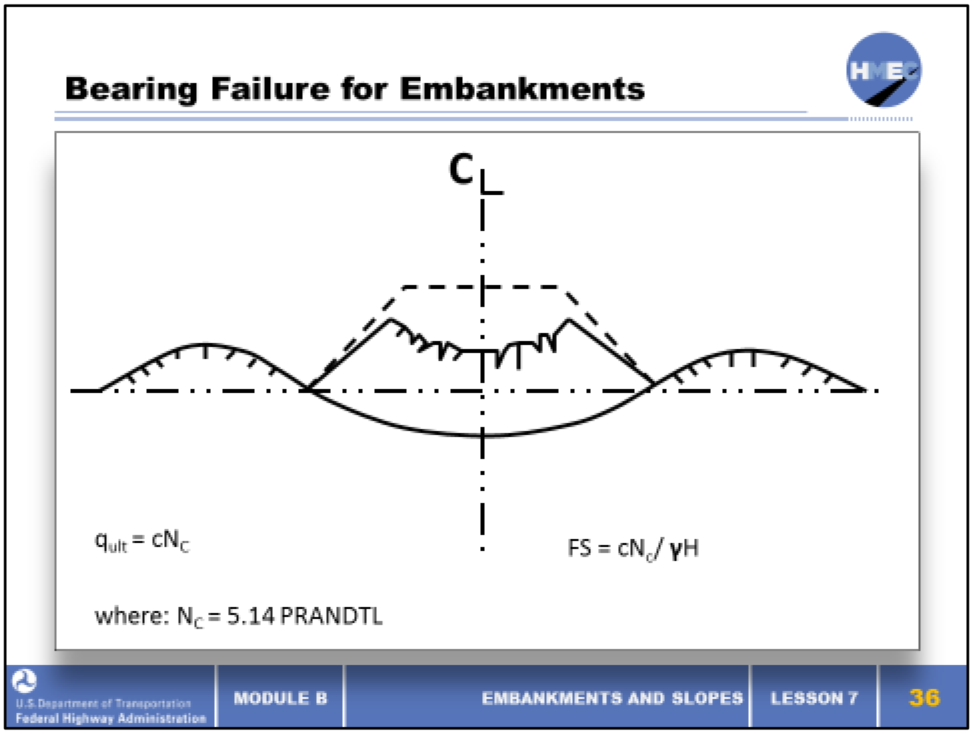
Slide 35

Embankment settlement can be a huge issue that can continue for years.

This project was put on hold and the 33-ft. embankment settled over 4 ft. in about seven years.

In the 1960s, a new highway was proposed across wetlands around a town in the Boston Metropolitan area of Massachusetts. Construction was started, but halted before completion, for environmental reasons. A portion of the 33-ft. (10 m) embankment was instrumented and left in place. The embankment was underlain by a thick layer of clay, about 90 ft. (27 m) thick. After 2,500 days (nearly seven years) the embankment had settled 4 ft. (1.2 m).

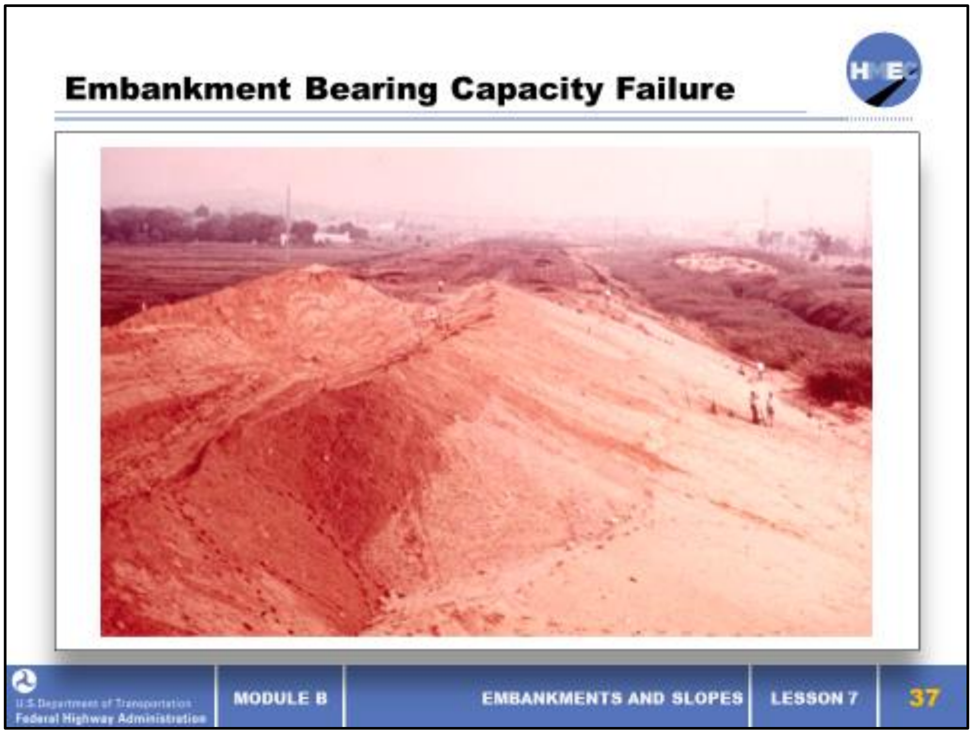
Slide 36



The bearing capacity for an embankment is a function of the cohesion or undrained shear strength of the soft layer in question and a bearing capacity factor N_C.

Bearing capacity may control the design of embankments over soft soils and is typically the first thing to check during the design.

Slide 37

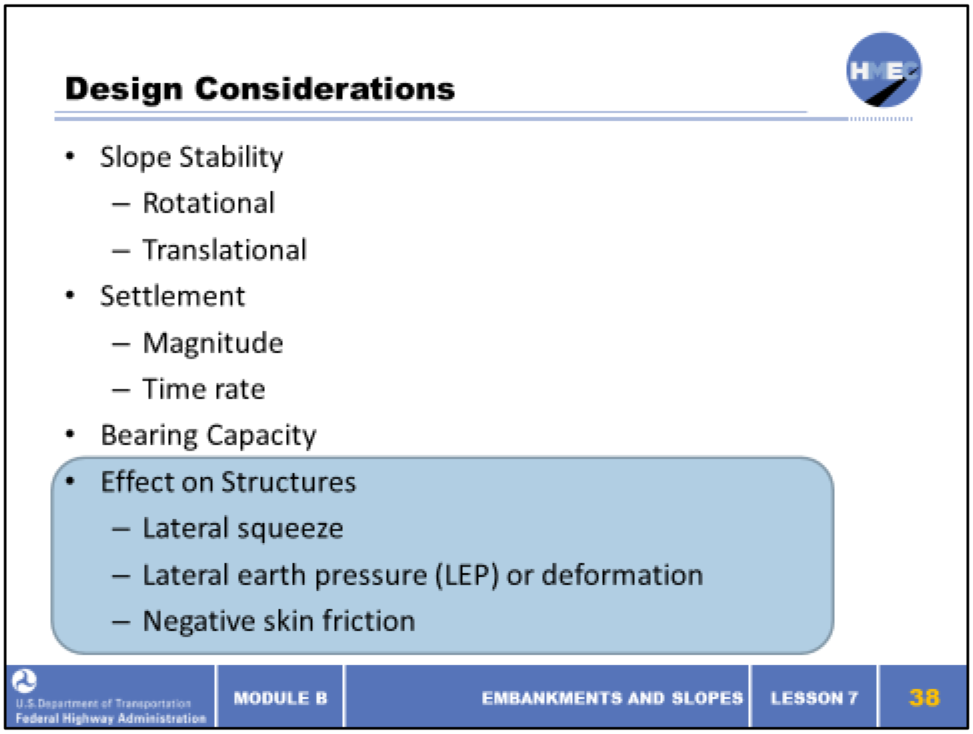


Bearing capacity failure in a soft clay layer underlying this embankment caused a settlement of over 16 ft.

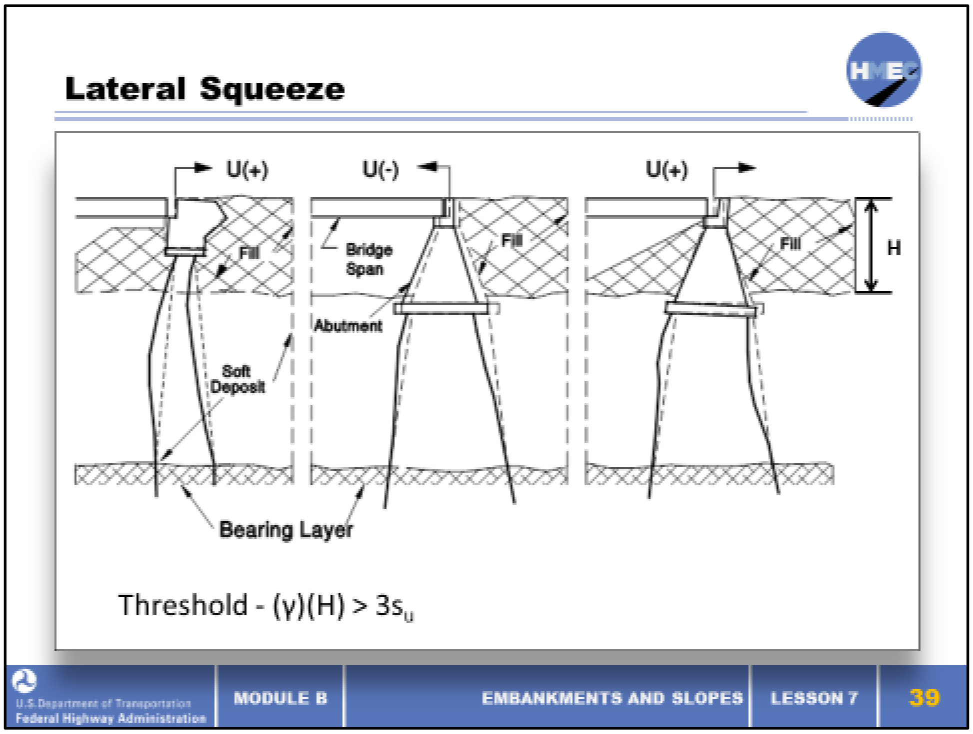
NHI-132033: This embankment fill failed one night after the amount of additional fill reached 18.7 ft. (5.7 m). Predictions varied from 8 to 27 ft. (2.4 to 8.2 m), with the actual (in ft.) being 12, 9, 23.5, 23, 13, 27, 8, 21, 13.5, 13 for an average of 16.3 ft. (5 m).

Most interestingly, instead of failing to one side as expected, it failed as a bearing capacity failure, with the total load creating a stress level fairly close to six times the shear strength of the foundation clay. Sometimes the simple approaches are just as good as the more advanced ones!

Slide 38


Design Considerations
HMEC
• Slope Stability
– Rotational
– Translational
• Settlement
– Magnitude
– Time rate
• Bearing Capacity
• Effect on Structures
– Lateral squeeze
– Lateral earth pressure (LEP) or deformation
– Negative skin friction
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
38

Slide 39



Field observations and measurements have shown that some bridge abutments supported on piles driven through compressible soils tilted toward the embankment fill and of these cases many have experienced large horizontal movements resulting in damage to the structure. The cause of this problem is attributed to the unbalanced fill load that "squeezes" the soil laterally— also called "lateral squeeze. "

The rule of thumb to determine if lateral squeeze is a possibility: If $(\gamma)\ H > 3S_u$ then further analysis is required.

____

____

____

____

____

____

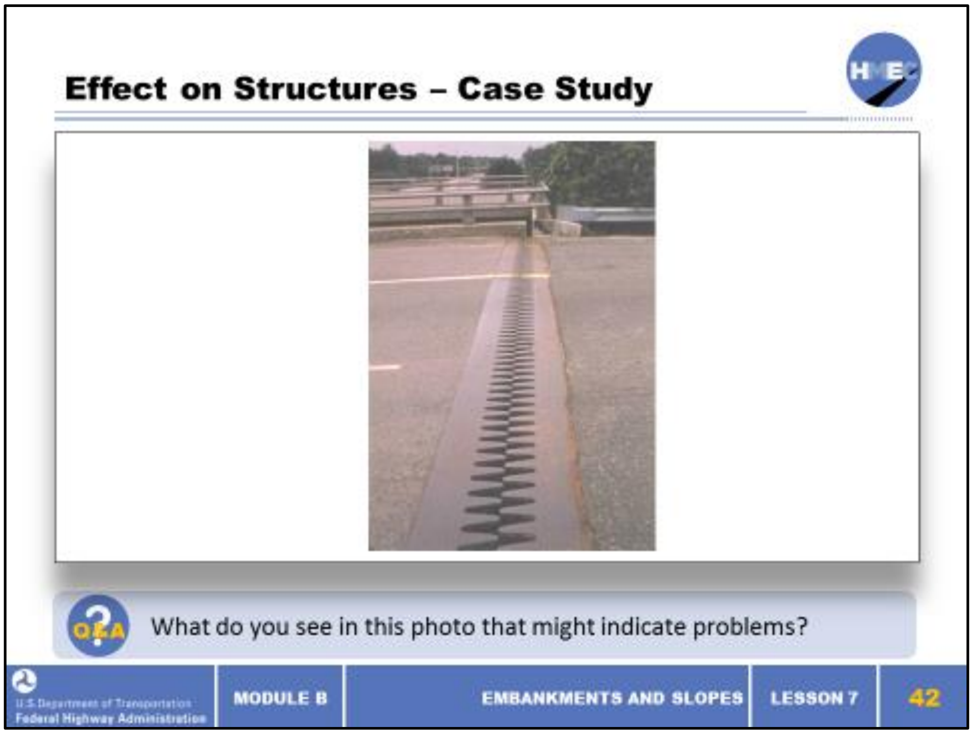
Slide 40

Structures are also affected by unaccounted for lateral earth pressures and settlement of the approach embankment.

Slide 41

This abutment footing has already settled over 1 ft. and more is anticipated.

Slide 42



What do you notice in this photo?

By looking at the gap of this bridge abutment expansion joint, it is either a very cold day or something else is occurring! Notice trees have all their leaves, but joint is at its maximum expansion.

The pavement looks level with the joint (little or no bump), however the bridge railing and the approach guardrail on the embankment do not match up. It is obvious that maintenance crews have done a good job of maintaining the pavement to correct the bump as the embankment has settled. Recall that vertical settlement is also accompanied by horizontal movement (25% is a good number). An inspection of the abutment beam seat will undoubtedly reveal tilting of the abutment into the embankment causing the expansion joint to be locked open.

____

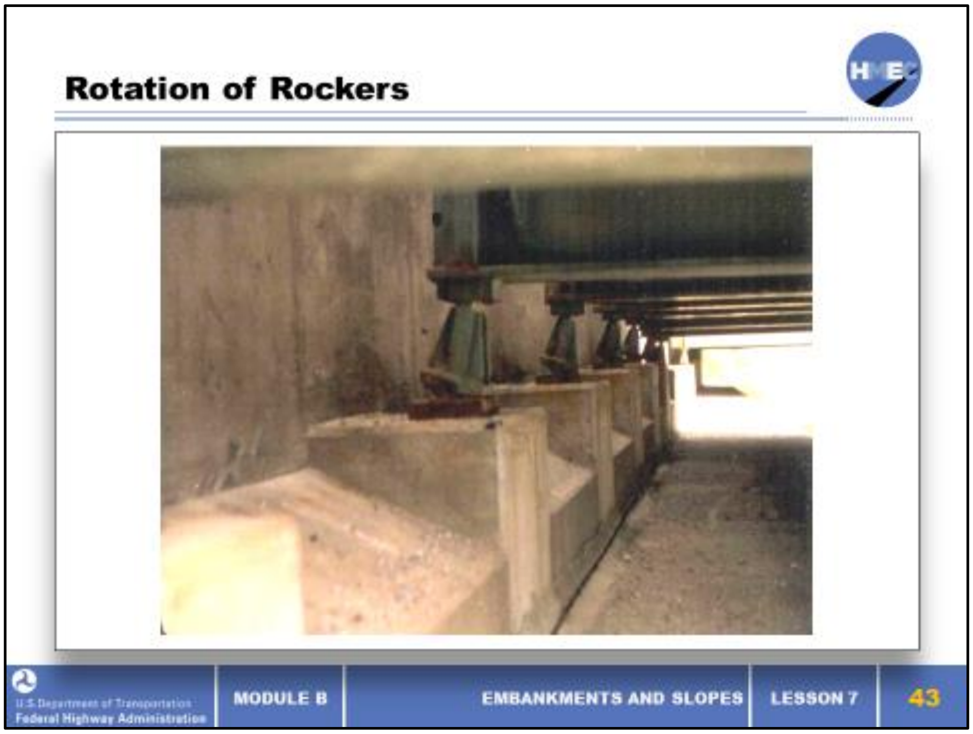
____

____

____

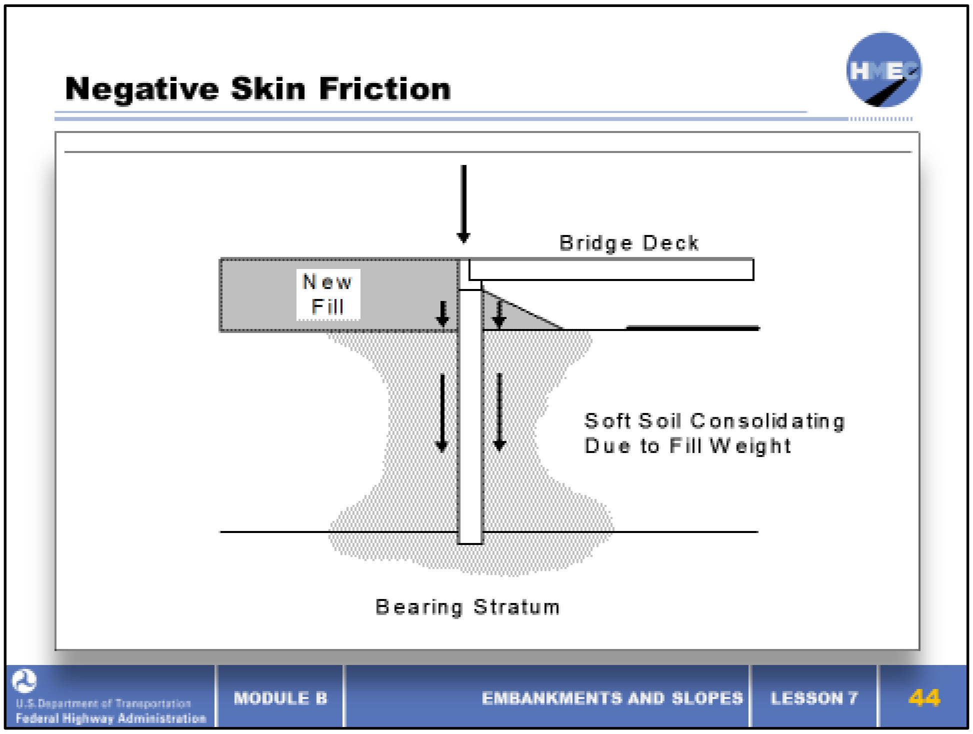
____

Slide 43



Note the backward rotation of the abutment and the position of the rocker bearings.

Slide 44



Normally when a pile is driven into the ground, the soil provides resistance on the pile, which generates the support or capacity of the pile. In other words, when the pile is loaded by the bridge, the soil helps hold the pile up. Think of it as a force arrow pointing up along the pile contrary to the downward pointing force arrow shown on this slide. When the embankment is placed and it causes settlement in the soft layer, the soft layer now wants to drag the pile down with it as it settles. That is down drag and acts as a load on the pile. If excessive, the added load can overstress the pile. Solutions include letting the settlement occur before driving the pile, designing the pile to handle the increased load, pre-boring through the soft layer and isolating the pile, and coating the pile to reduce soil adhesion to the pile.

____

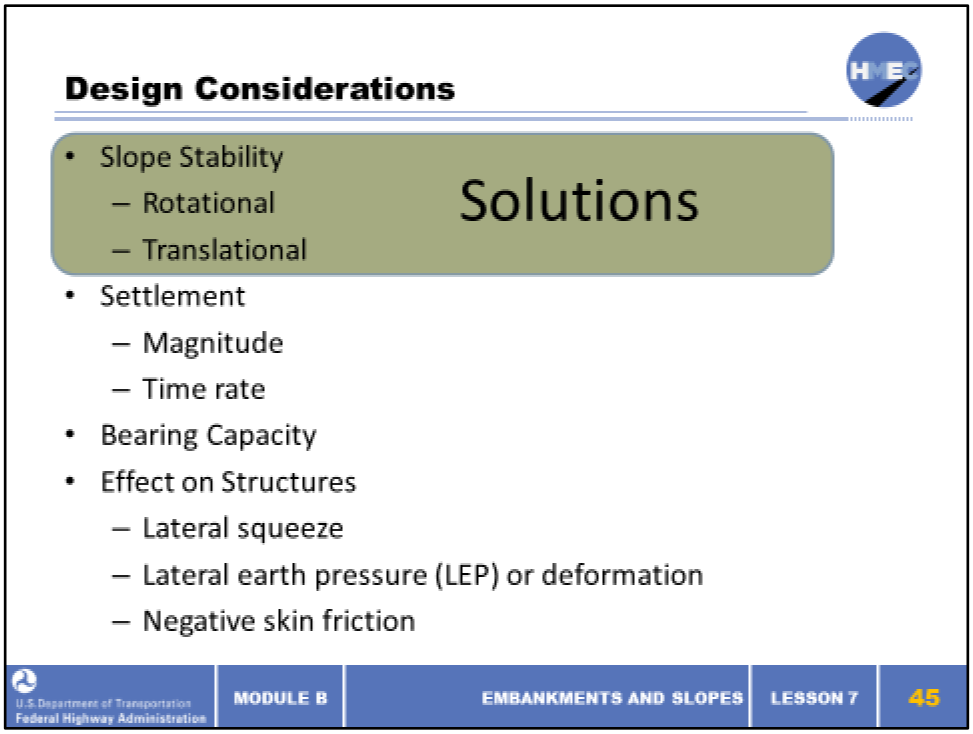
____

____

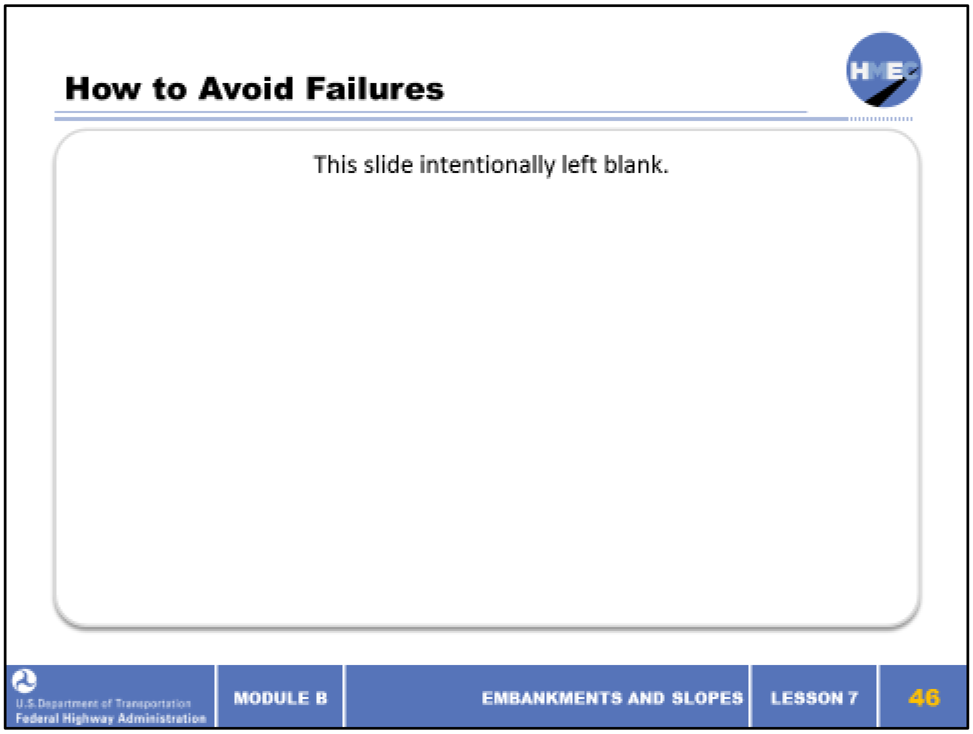
____

____

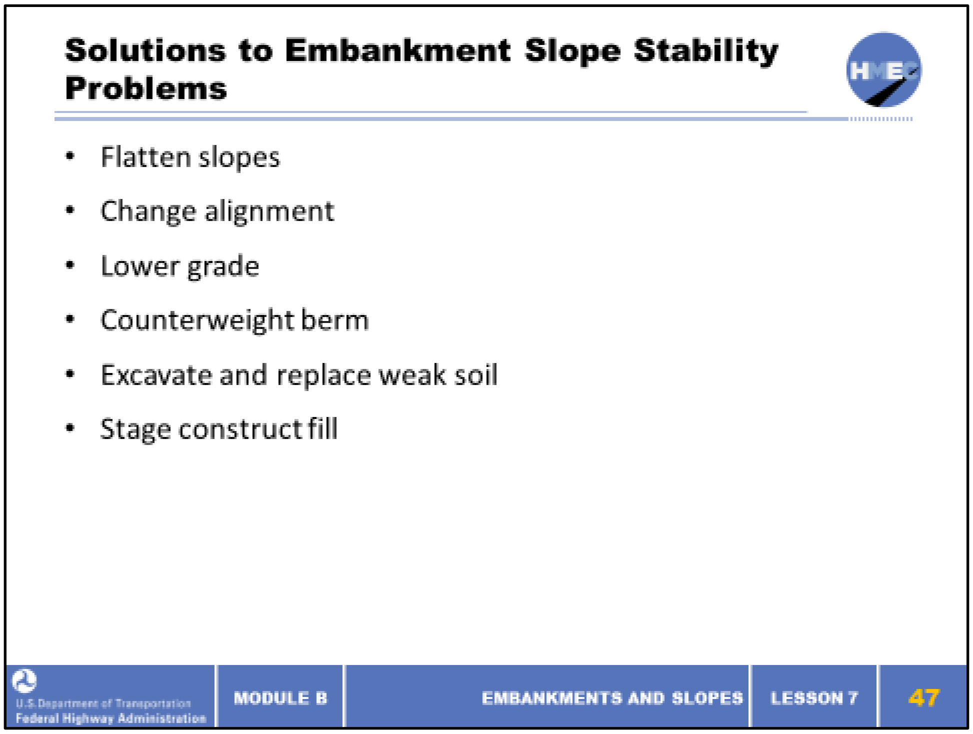
Slide 45


Design Considerations
HMEC
• Slope Stability
– Rotational
– Translational
Solutions
• Settlement
– Magnitude
– Time rate
• Bearing Capacity
• Effect on Structures
– Lateral squeeze
– Lateral earth pressure (LEP) or deformation
– Negative skin friction
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
45

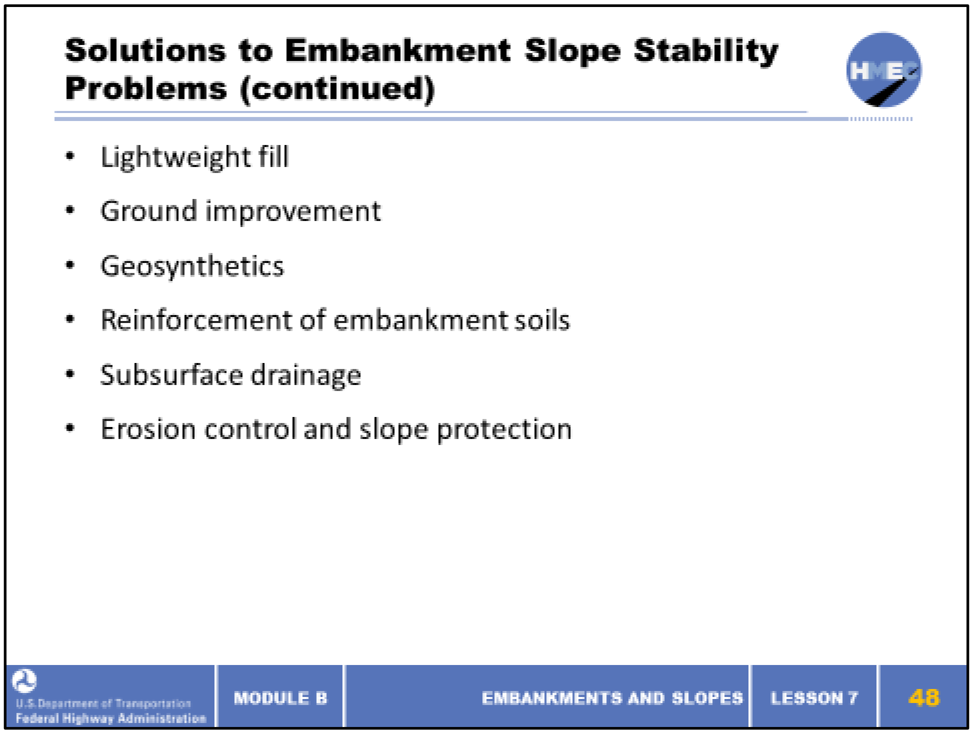
Slide 46


How to Avoid Failures
HMEC
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
46

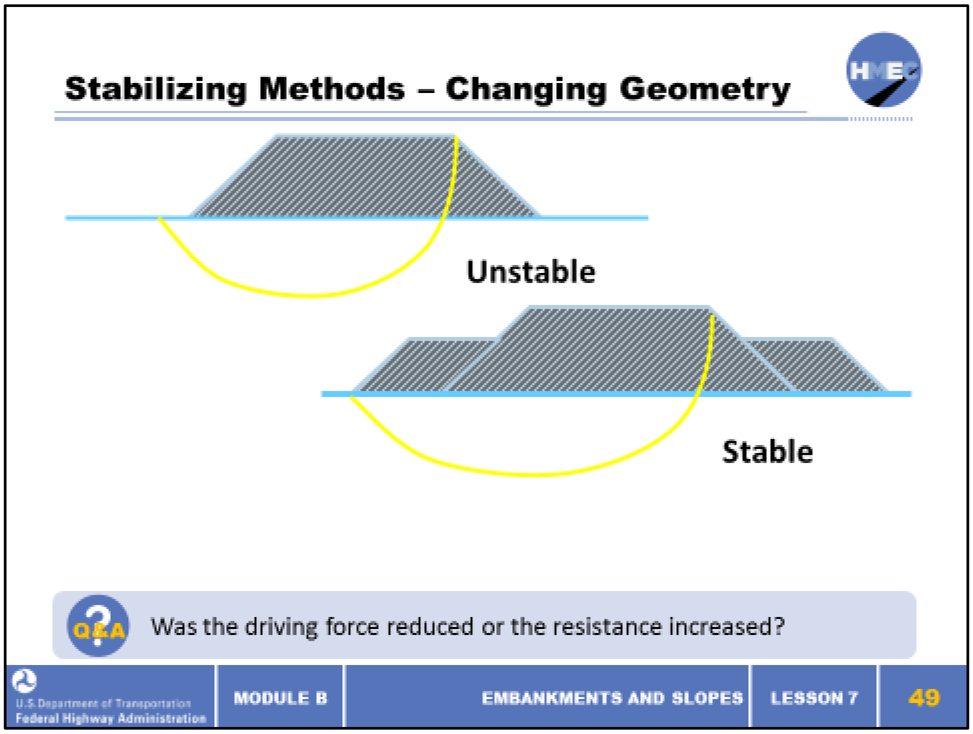
Slide 47


Solutions to Embankment Slope Stability Problems
HMEC
• Flatten slopes
• Change alignment
• Lower grade
• Counterweight berm
• Excavate and replace weak soil
• Stage construct fill
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
47

Slide 48

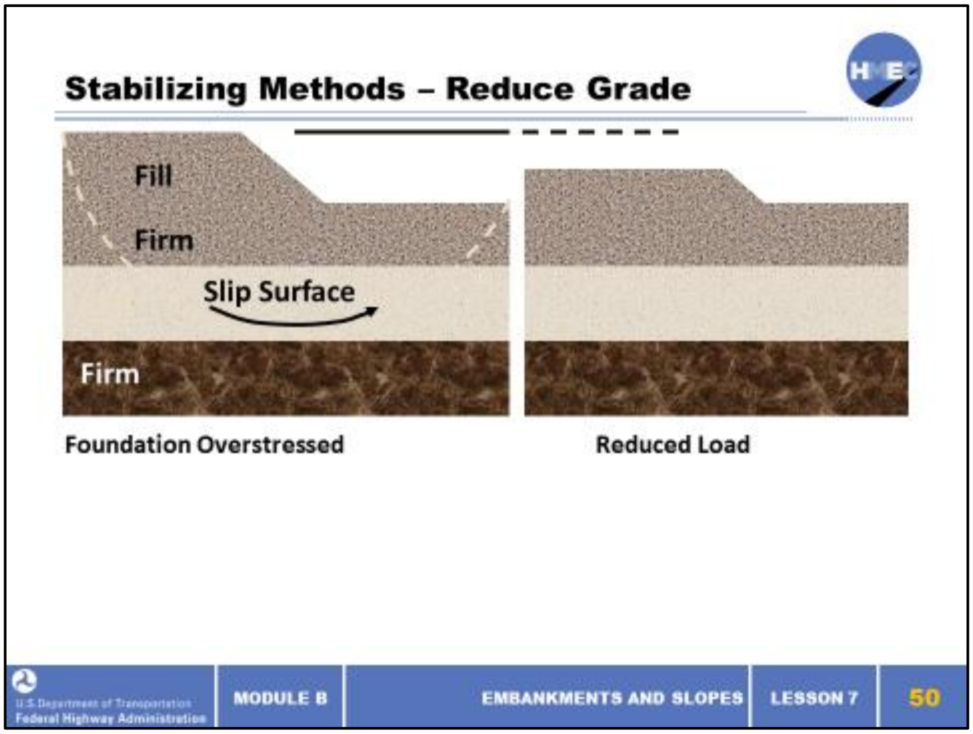

Solutions to Embankment Slope Stability Problems (continued)
HMEC
• Lightweight fill
• Ground improvement
• Geosynthetics
• Reinforcement of embankment soils
• Subsurface drainage
• Erosion control and slope protection
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
48

Slide 49

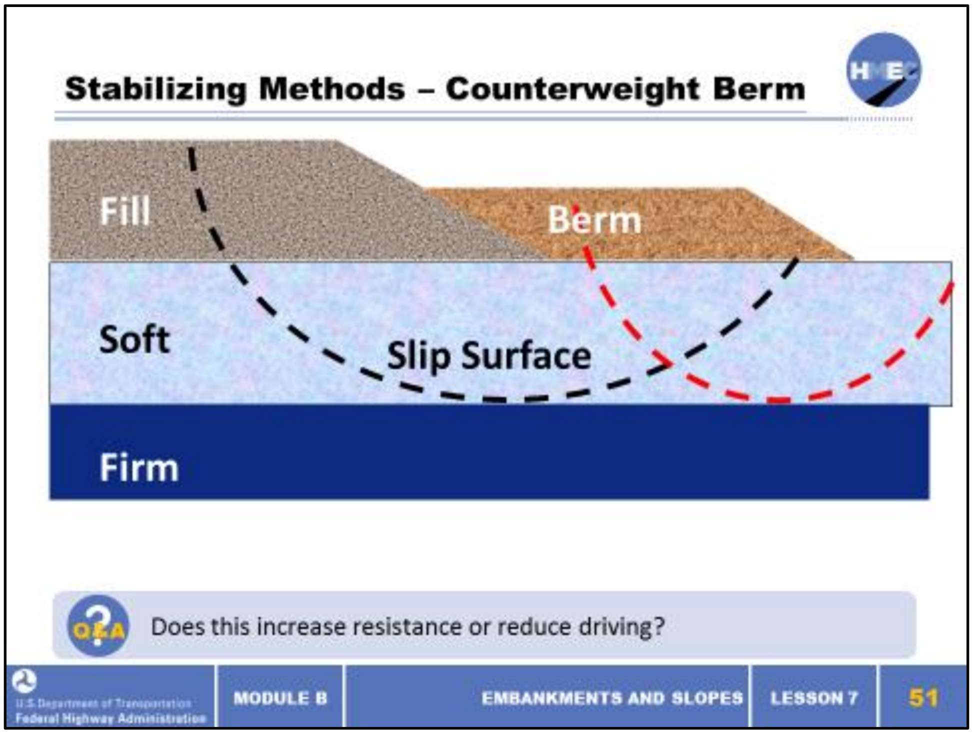


We can change the geometry to increase the FS against failure.

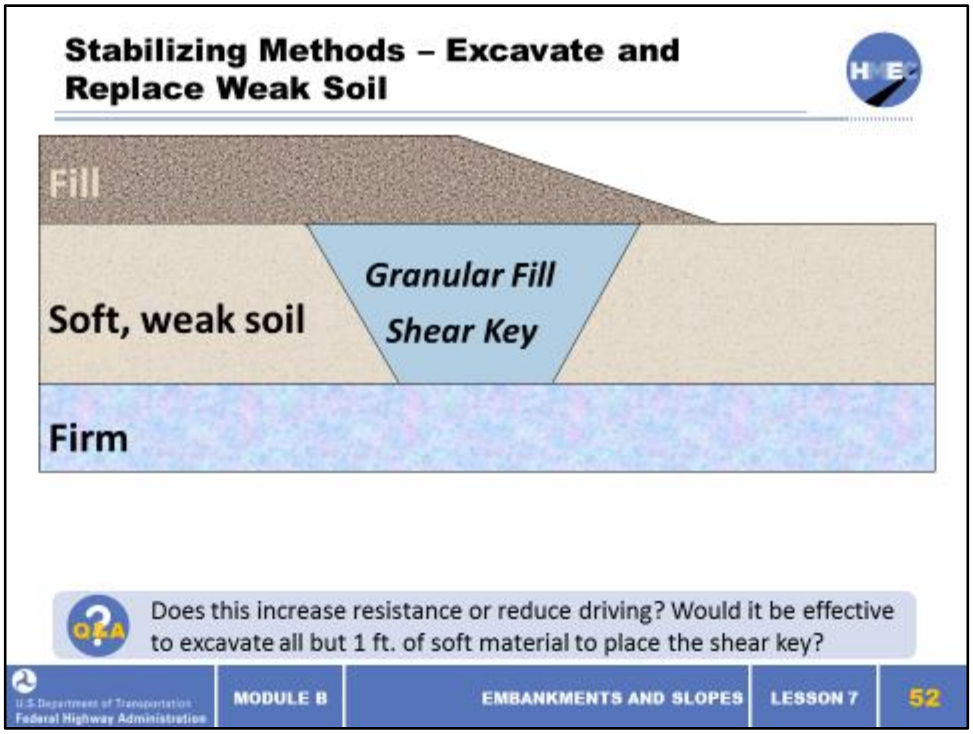
Slide 50


Stabilizing Methods – Reduce Grade
Fill
Firm
Slip Surface
Firm
Foundation Overstressed
Reduced Load
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
50

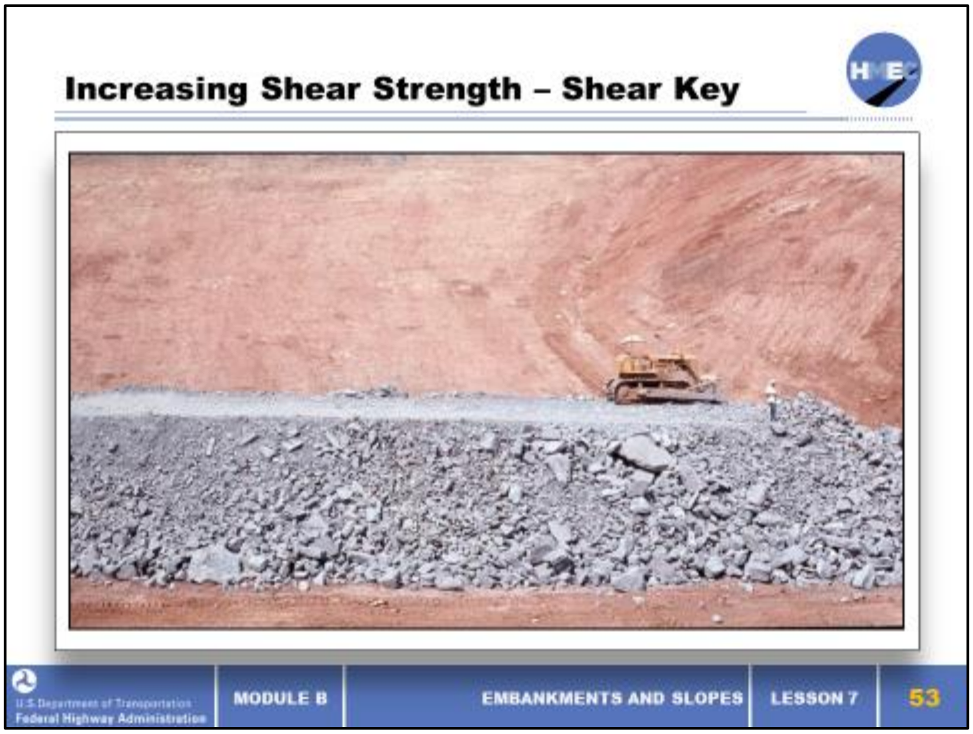
Slide 51


Stabilizing Methods – Counterweight Berm
Fill
Berm
Soft
Slip Surface
Firm
Does this increase resistance or reduce driving?
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
51

Slide 52


Stabilizing Methods – Excavate and Replace Weak Soil
Fill
Soft, weak soil
Granular Fill
Shear Key
Firm
Does this increase resistance or reduce driving? Would it be effective to excavate all but 1 ft. of soft material to place the shear key?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
52

Slide 53



A slide occurred on Interstate 40 in Roane County, Tennessee 1972. The solution was to remove the sliding material and flatten the slope (change geometry). As seen here, a large shear key was also constructed, effectively increasing the shear strength of the earth resisting the sliding forces.

The role of the Materials Engineer is important in ensuring strong, durable material is used in the shear key.

Slide 54

This counter berm is placed at the toe of slope and is composed of rock. Note that the length and height of the berm must be designed. Also, the unit weight assumed for the berm in design must be checked in construction.

____

____

____

____

____

____

____

____

____

____

Slide 55

Recall that cut slope failure we showed earlier. This was the DOT's solution to excavate the weak material and replace it with rock.

____

____

____

____

____

____

____

____

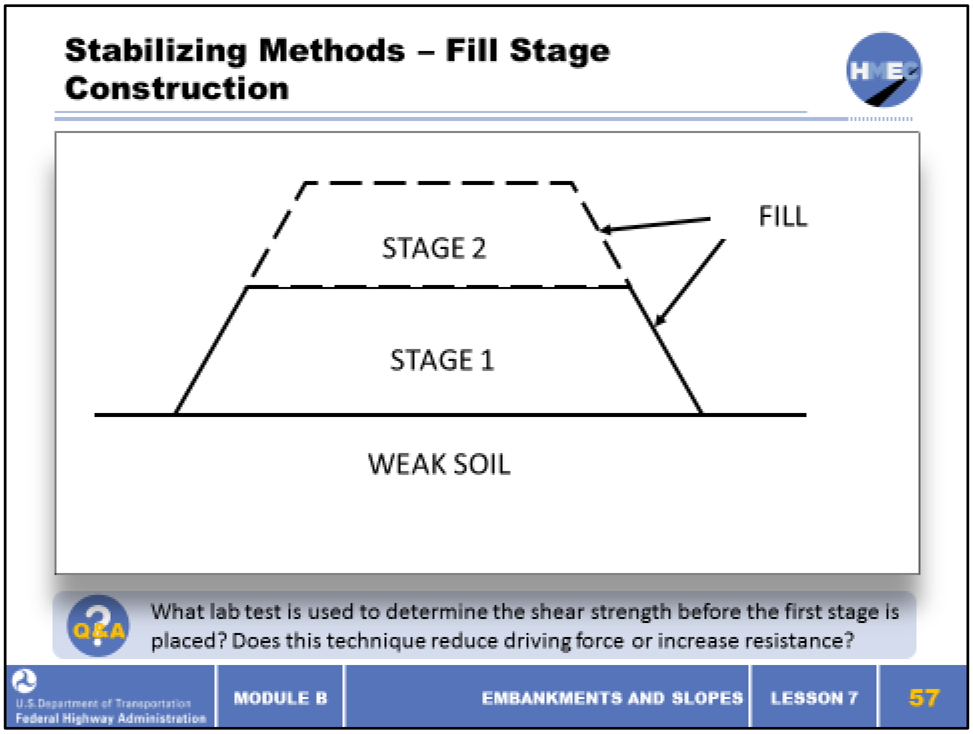
____

____

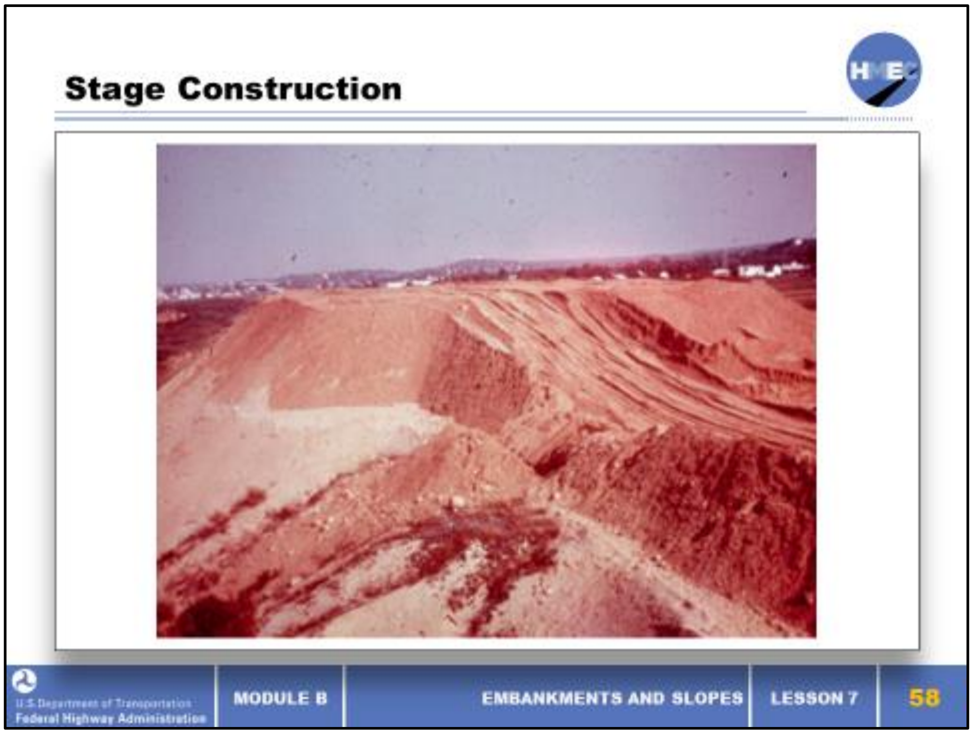
Slide 56

Note that the rock key at the toe is absolutely necessary to provide a base to prevent the rock from slipping back down slope due to erosion at the toe.

Slide 57

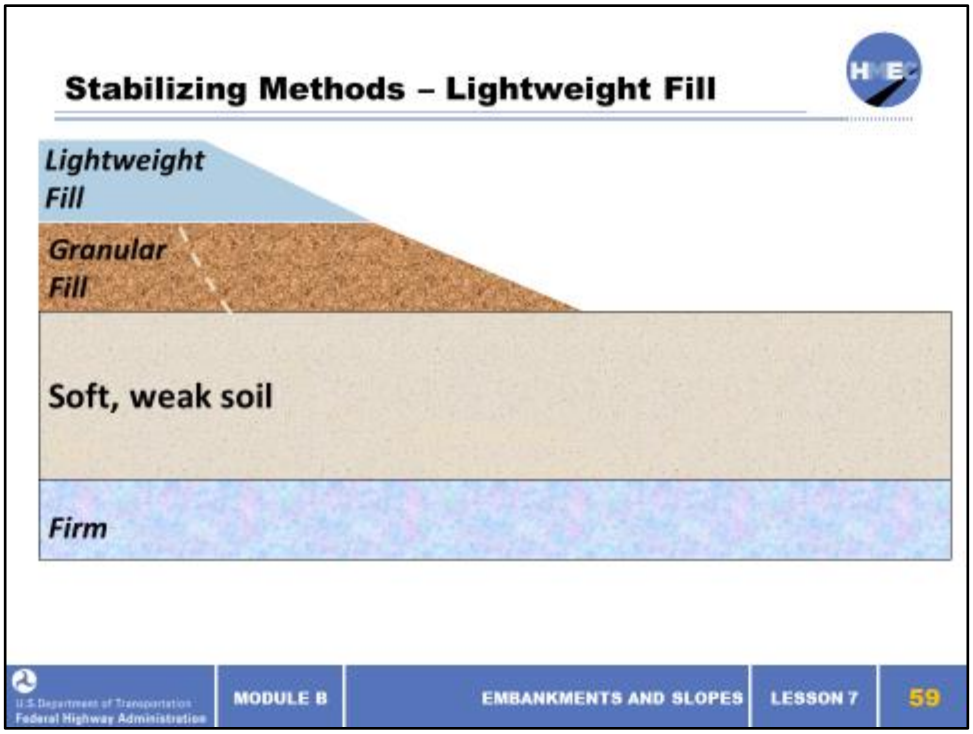

Stabilizing Methods – Fill Stage Construction
HMEC
STAGE 2
FILL
STAGE 1
WEAK SOIL
Q&A
What lab test is used to determine the shear strength before the first stage is placed? Does this technique reduce driving force or increase resistance?
U.S. Department of Transportation Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
57

Slide 58



Stage construction should always be monitored with geotechnical instrumentation to ensure that the consolidation is occurring in the soft soil as planned and that the fill height rate of placement is acceptable.

Slide 59


Stabilizing Methods – Lightweight Fill
Lightweight Fill
Granular Fill
Soft, weak soil
Firm
U.S. Department of Transportation Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
59

Slide 60

The top left photograph shows placement of the geofoam. The bottom right photograph shows the completed project. Describe the project from the details provided below.

Case Study Notes: This slide was in Marion County, Alabama on AL 44. The roadway goes up the hillside and after several years of investigation it was determined that there were actually two failure planes one went very deep the other was approximately 30 to 35 ft. below the roadway. It was determined that to fix the upper slide and not contribute to the lower failure plane that a lightweight fill would be the best approach. After looking at the unit weights of several materials, the Geofoam seemed to be the best even though its cost was more.

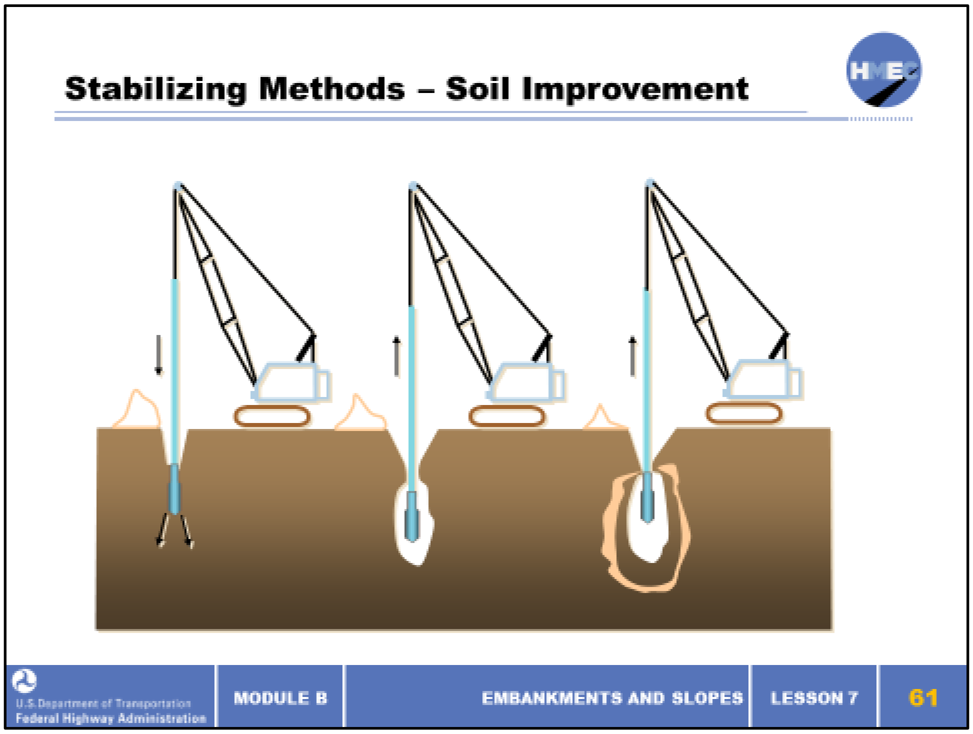
Geofoam was installed roughly 300 ft. in length and about 35 to 40 ft. in depth. The factor of safety for the shallow slide was increased to about 1.5 and the factor of safety for the deep slide was estimated to be improved to approximately 1.

____

____

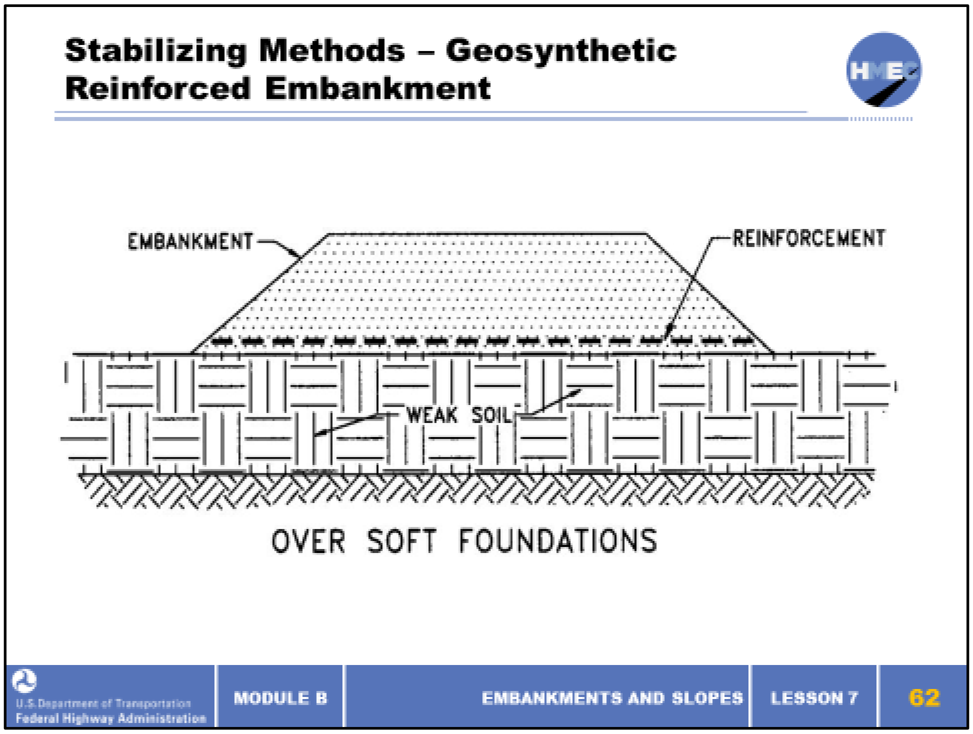
____

Slide 61



There are numerous types of ground improvement techniques, some of which improve shear strength of the soft layers, and therefore increase resistance or reduce settlement.

Slide 62



Geosynthetic mats can be used to construct fills over very soft ground, such as peat deposits and bogs. Geosynthetic materials allow the placement of soil fill and the operation of equipment that would otherwise not be possible. Geosynthetics increase resistance.

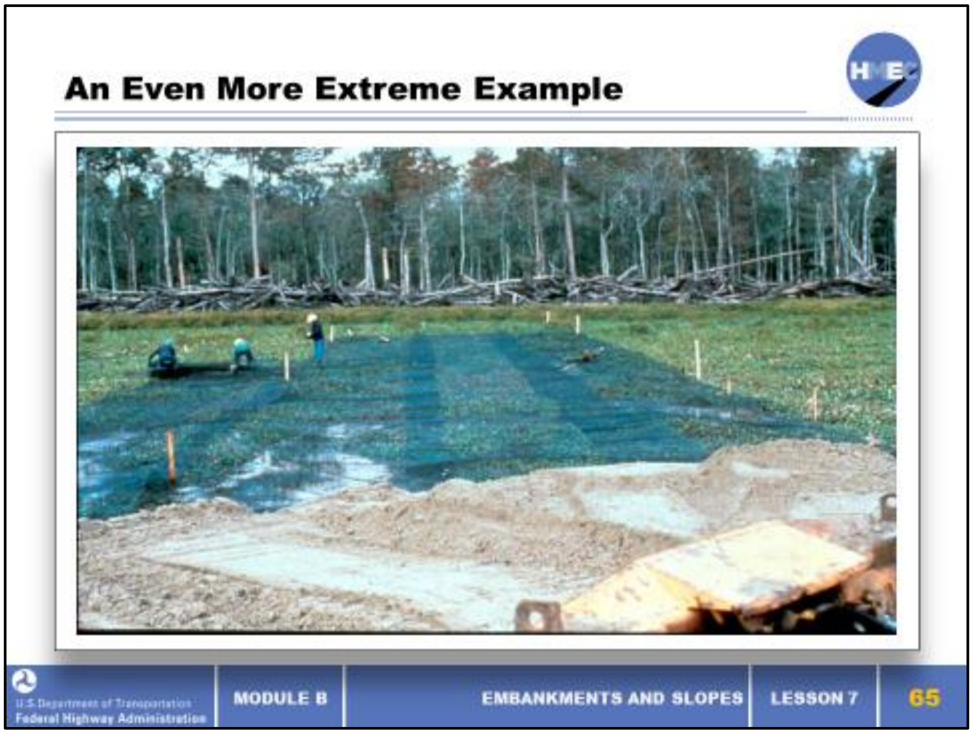
Slide 63

This is a case study of a project to construct an embankment for a roadway widening by the NCDOT over very soft ground.

Slide 64

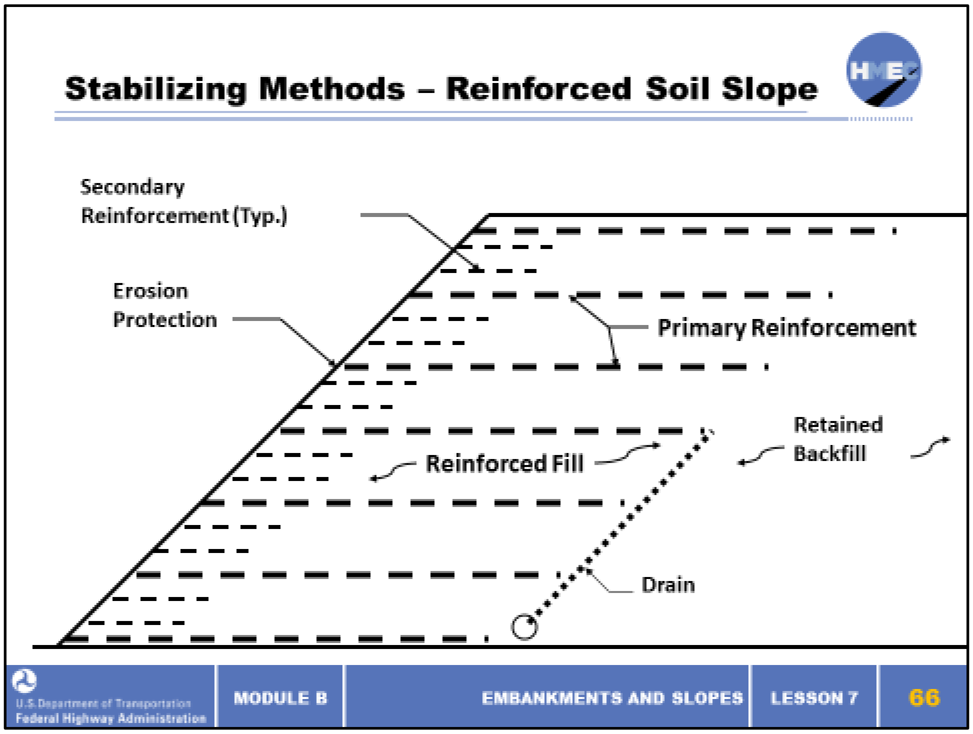
The NCDOT project showing fill placement after the initial lift.

Slide 65



Use of geogrid to reinforce an embankment across a swamp.

Slide 66



When the geosynthetic material is used to reinforce a steepened slope, it must be designed in accordance with accepted procedures. When used as a compaction aid near the embankment slope, a 3-4-ft. piece is placed on about every 18 inches. The only design requirement is that the geosynthetic material must be able to survive construction equipment during placement of the fill.

____

____

____

____

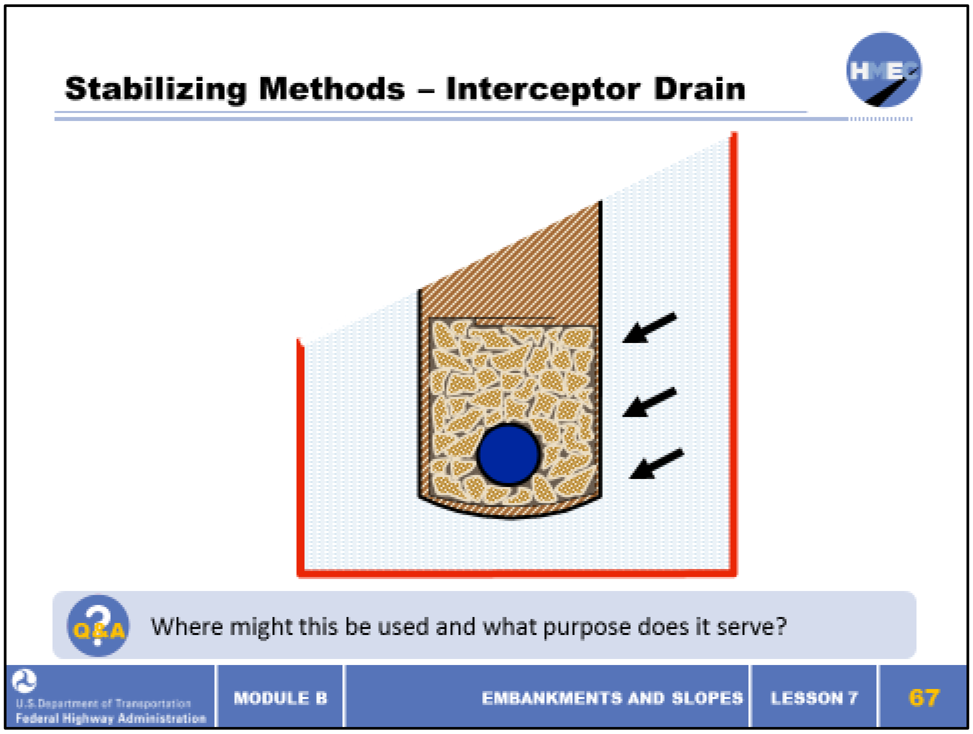
____

____

____

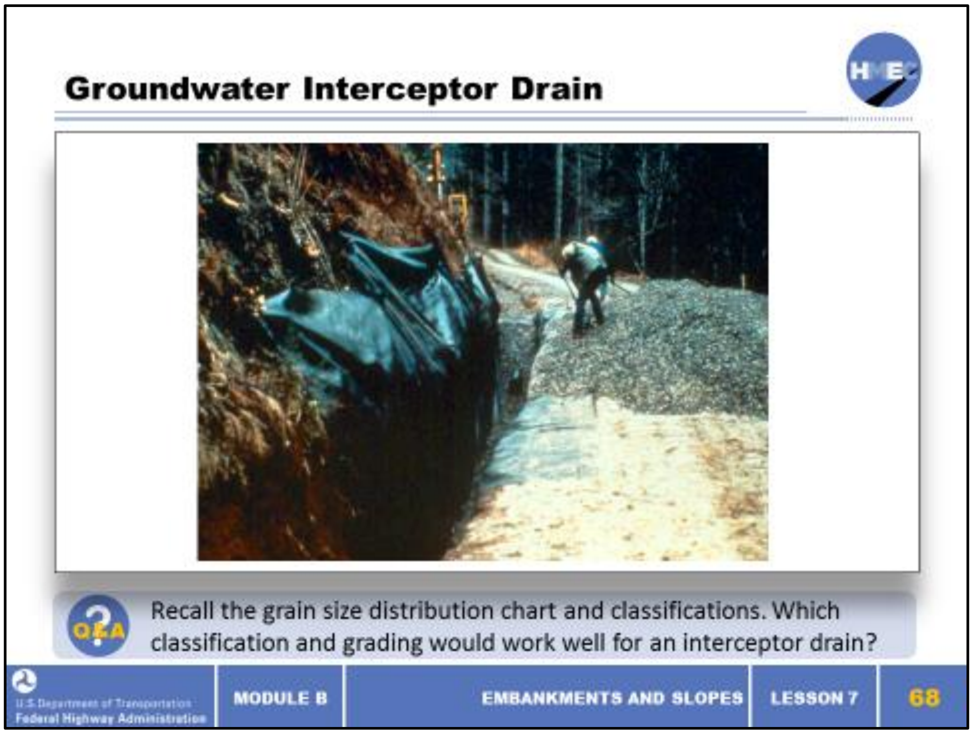
____

Slide 67

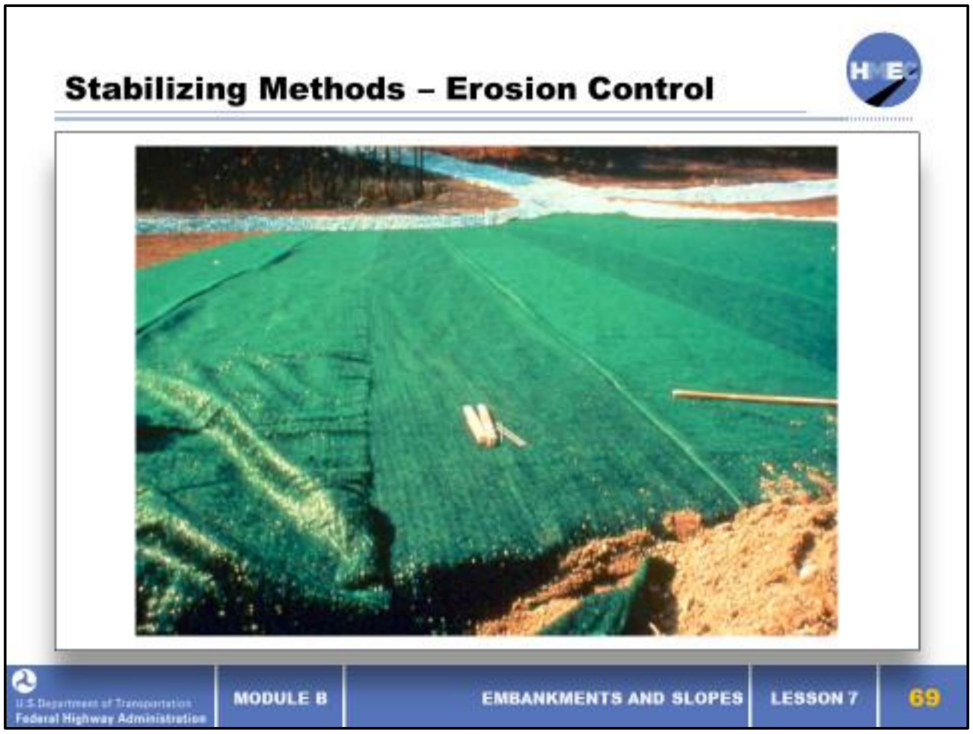


An interceptor drain can be used to control groundwater.

Slide 68

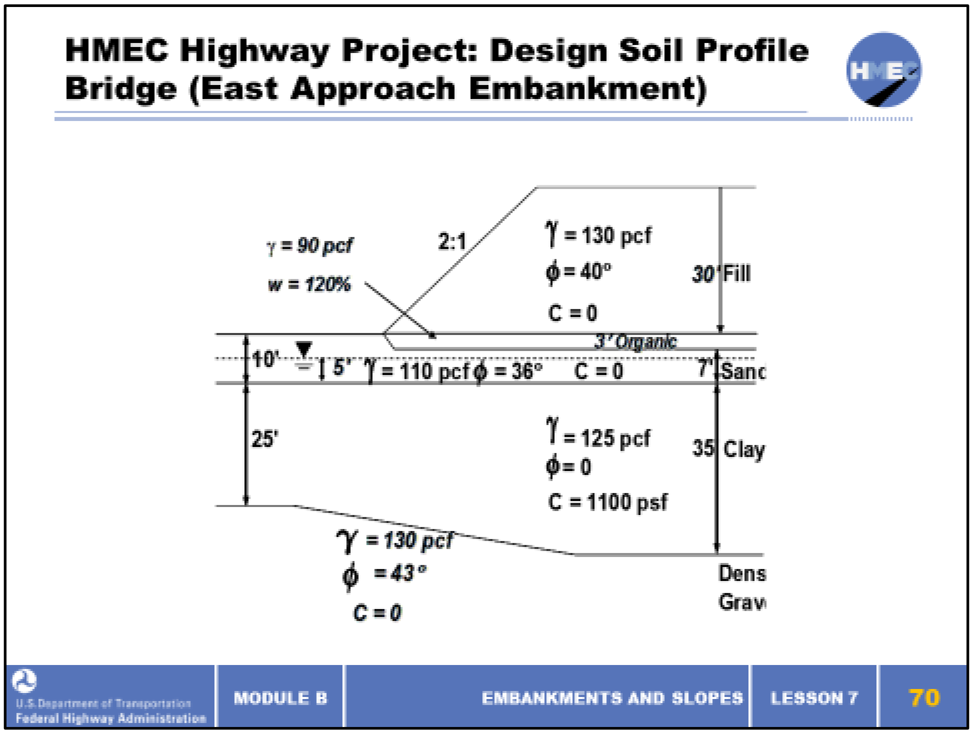

Groundwater Interceptor Drain
Recall the grain size distribution chart and classifications. Which classification and grading would work well for an interceptor drain?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
68

Slide 69

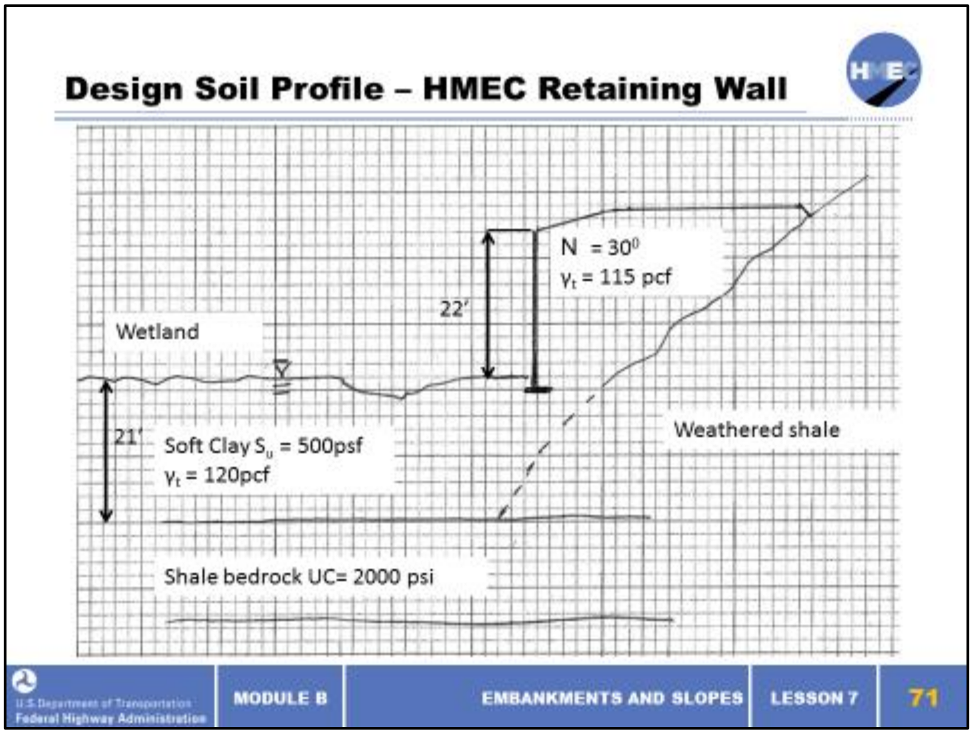


Erosion left unchecked will continue until it becomes a problem for the embankment or cut slope.

Slide 70

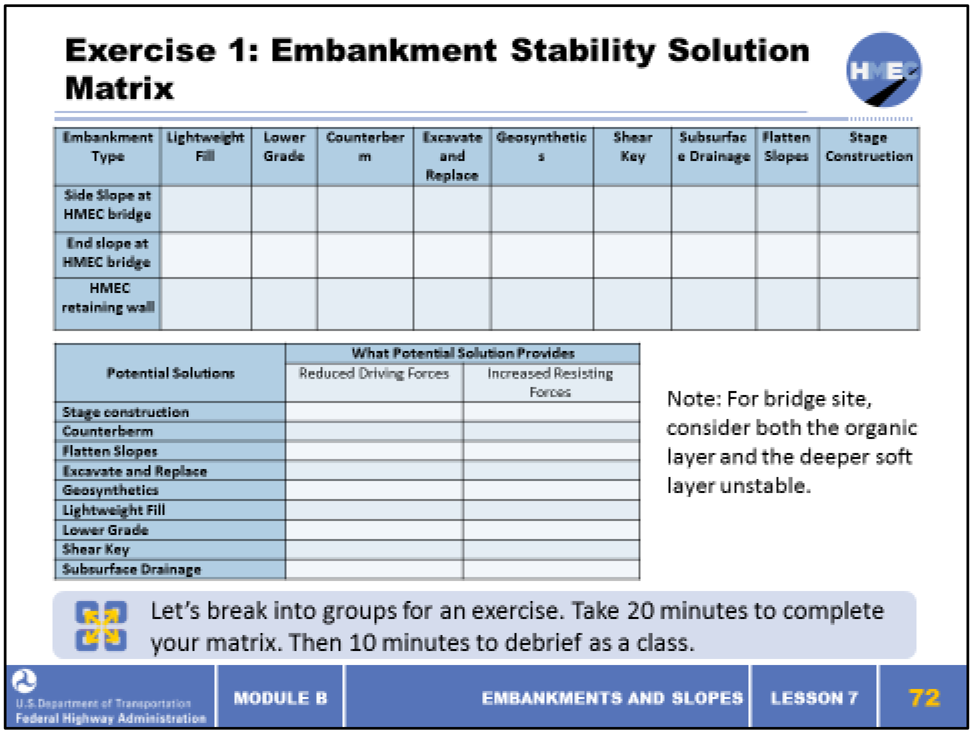

HMEC Highway Project: Design Soil Profile
Bridge (East Approach Embankment)
γ = 90 pcf
w = 120%
2:1
γ = 130 pcf
ϕ = 40°
C = 0
30' Fill
3' Organic
10'
5'
γ = 110 pcf ϕ = 36° C = 0
7' Sanc
25'
γ = 125 pcf
ϕ = 0
C = 1100 psf
35' Clay
γ = 130 pcf
ϕ = 43°
C = 0
Dens
Grav
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
70

Slide 71



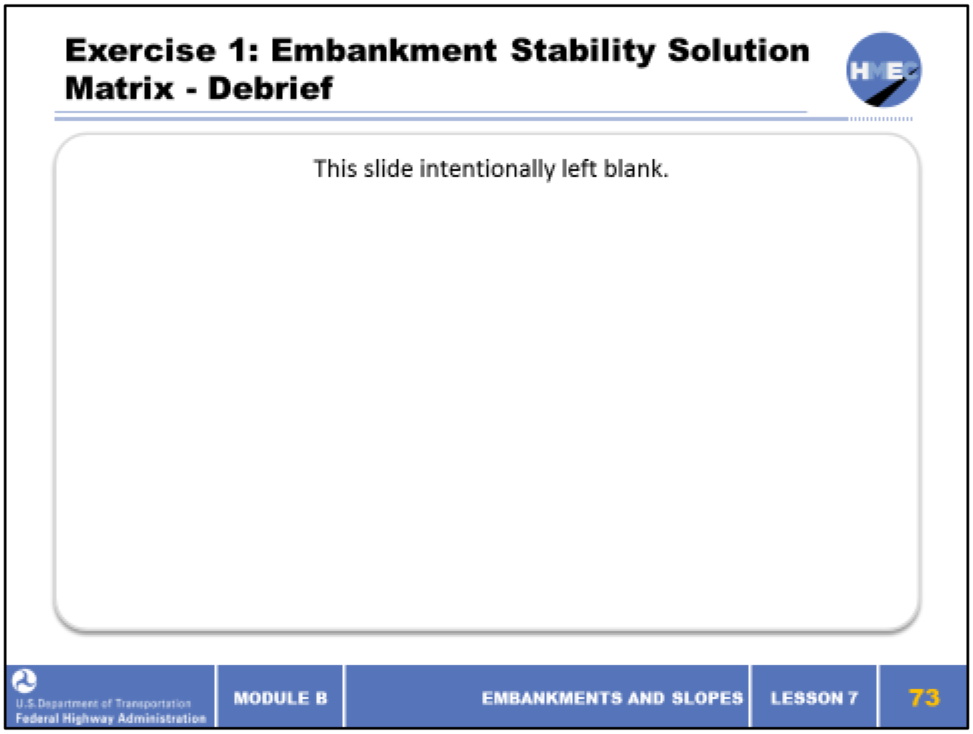
The HMEC project also has a 25-ft. high retaining wall adjacent to a wetland.

Slide 72

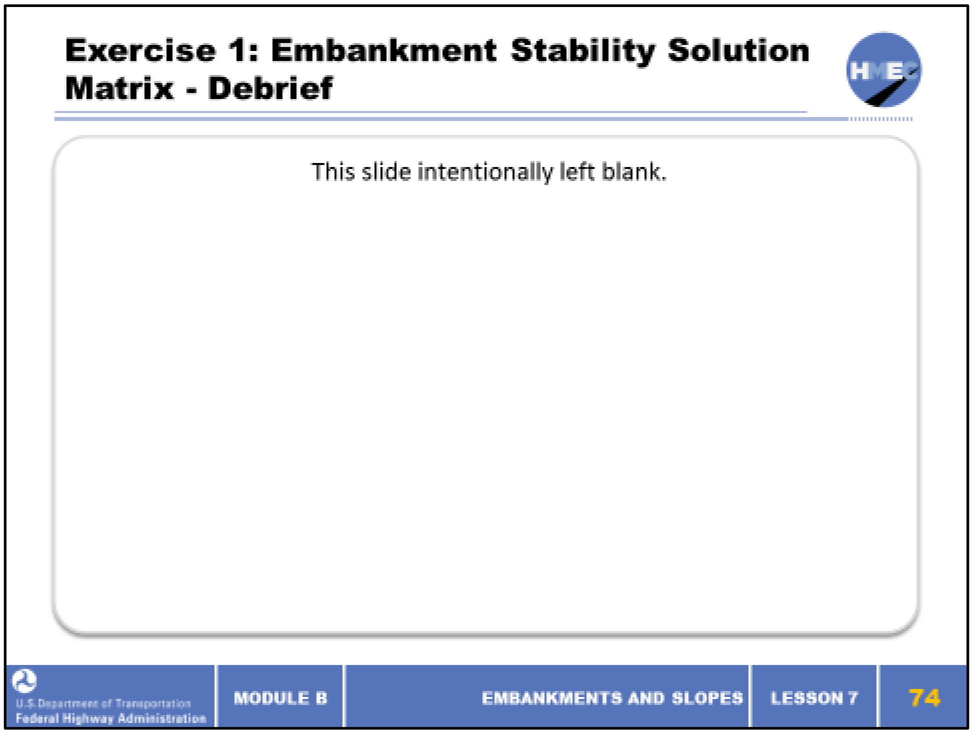


Find the two matrixes shown here in your participant workbook (PW). This exercise should help you choose solutions for potential stability issues. You will work together in groups to evaluate the soil profile at a site on the HMEC project and determine what solutions will be appropriate for the stability issues that may exist.

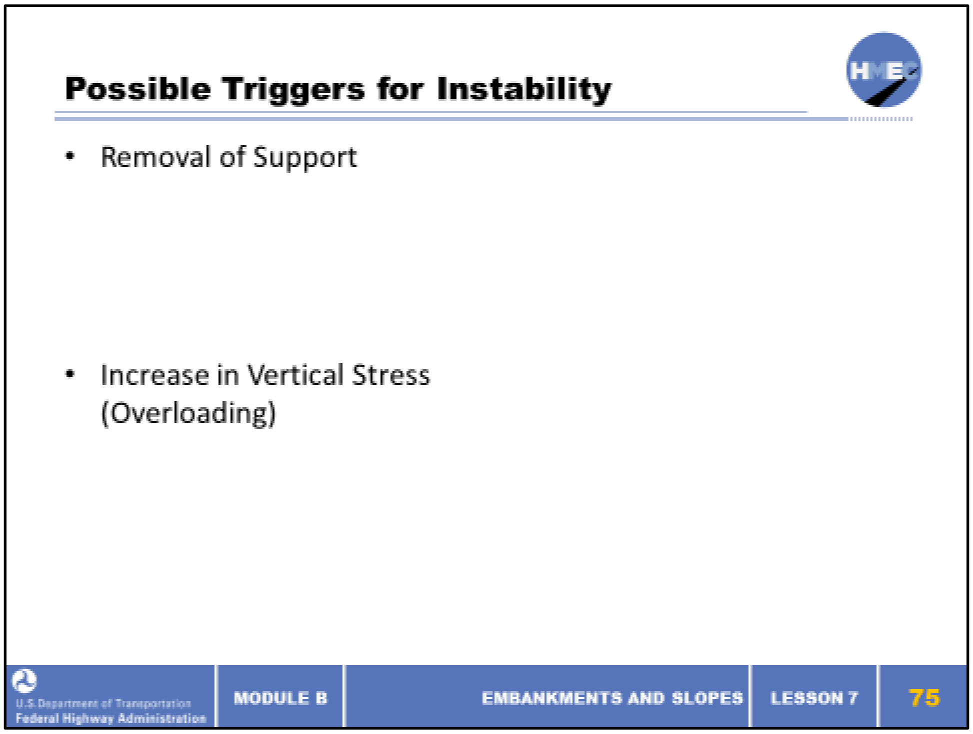
Slide 73


Exercise 1: Embankment Stability Solution Matrix - Debrief
HMEC
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
73

Slide 74

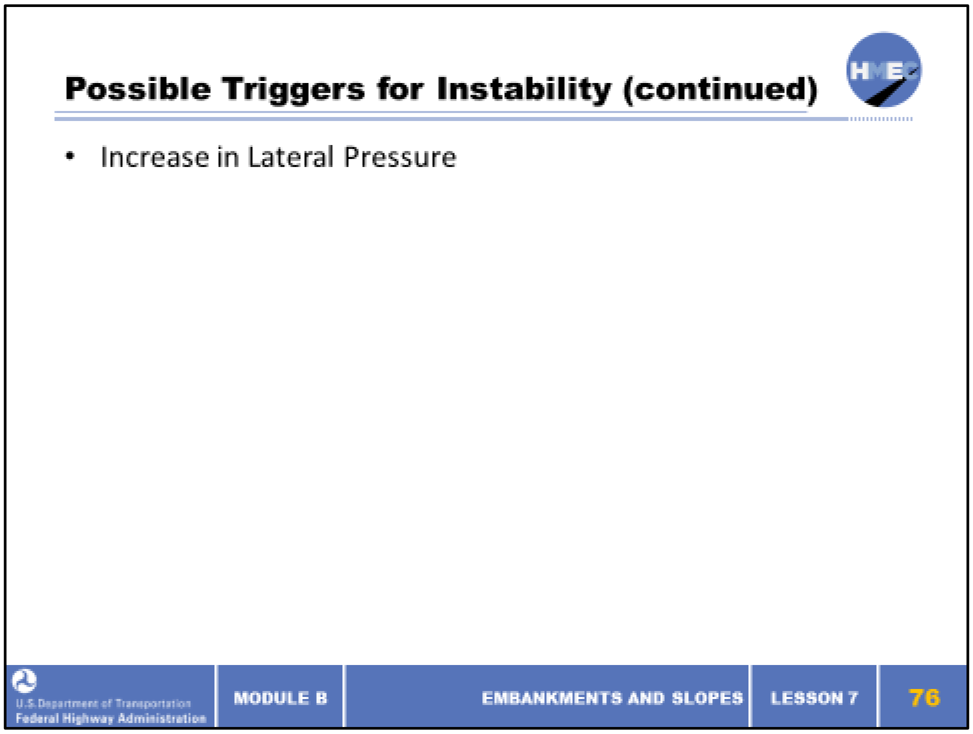

Exercise 1: Embankment Stability Solution Matrix - Debrief
HMEC
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
74

Slide 75



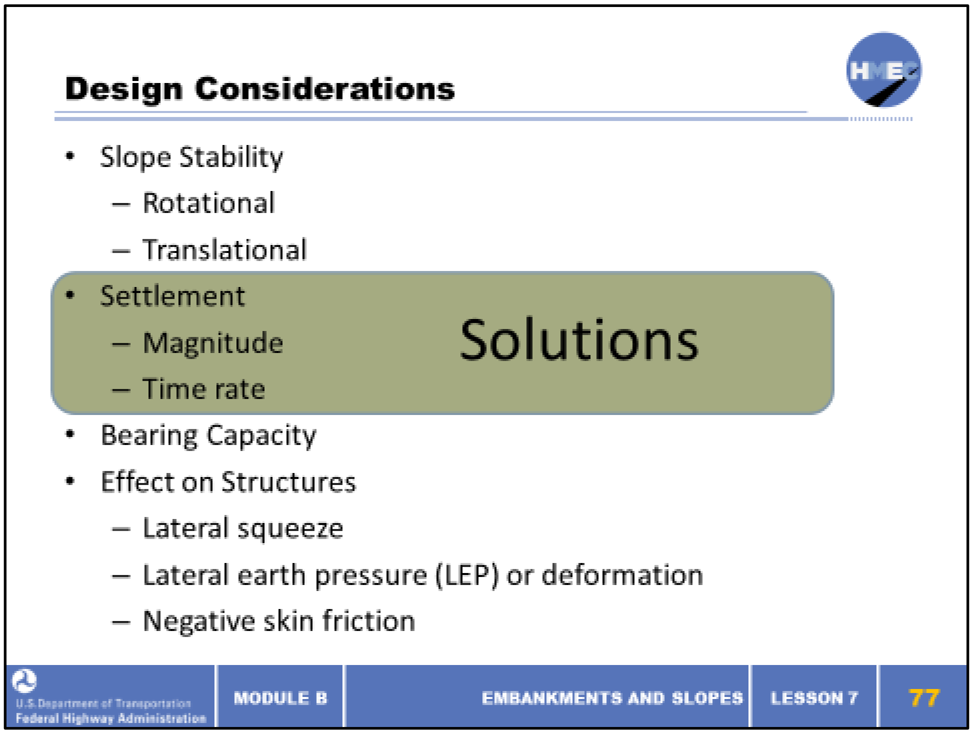
Just a quick overview of some triggers that can initiate stability problems.

Slide 76



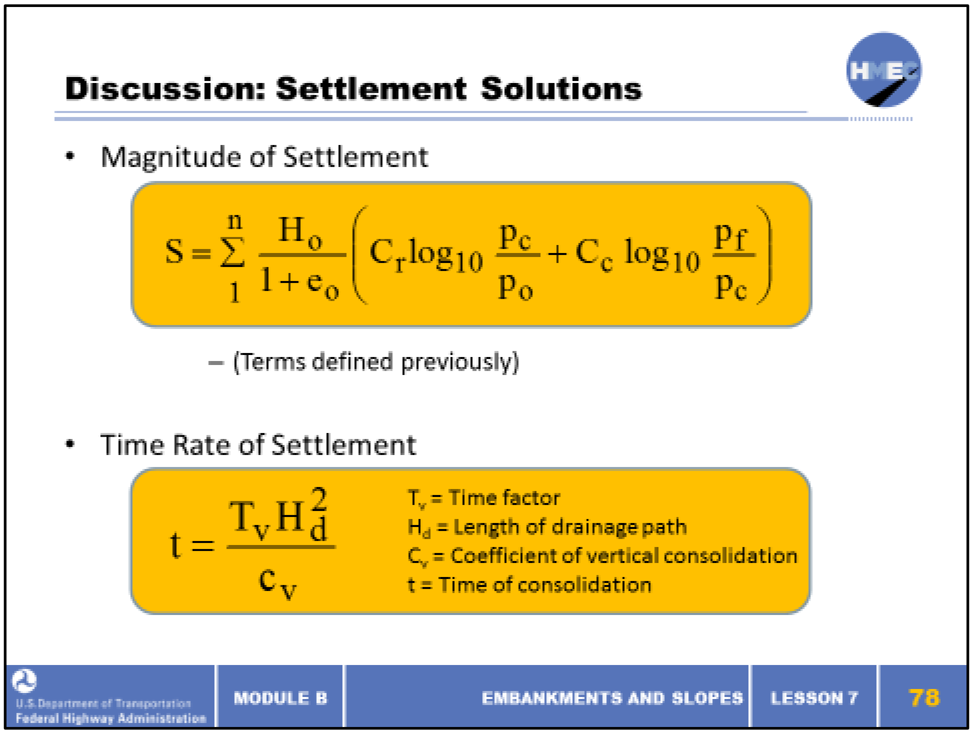
Just a quick overview of some triggers that can initiate stability problems. Tie them into the increasing driving forces and decreasing resistance.

Slide 77



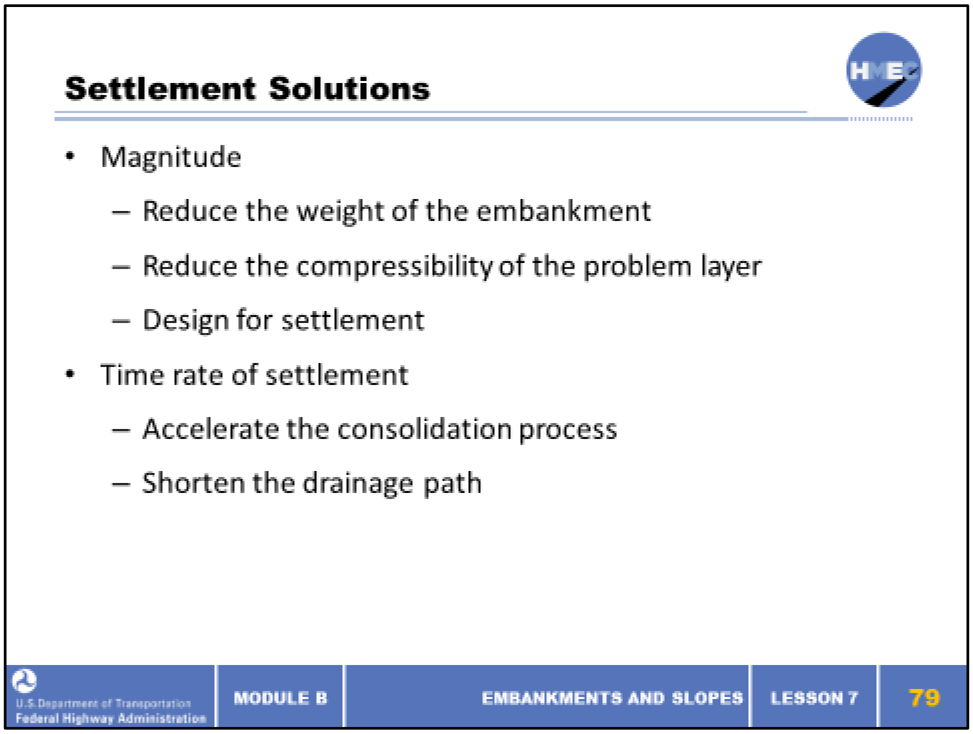
How about solutions for settlement.

Slide 78



Now that we have solved all of our stability problems, don't forget embankments are also subject to settlement.

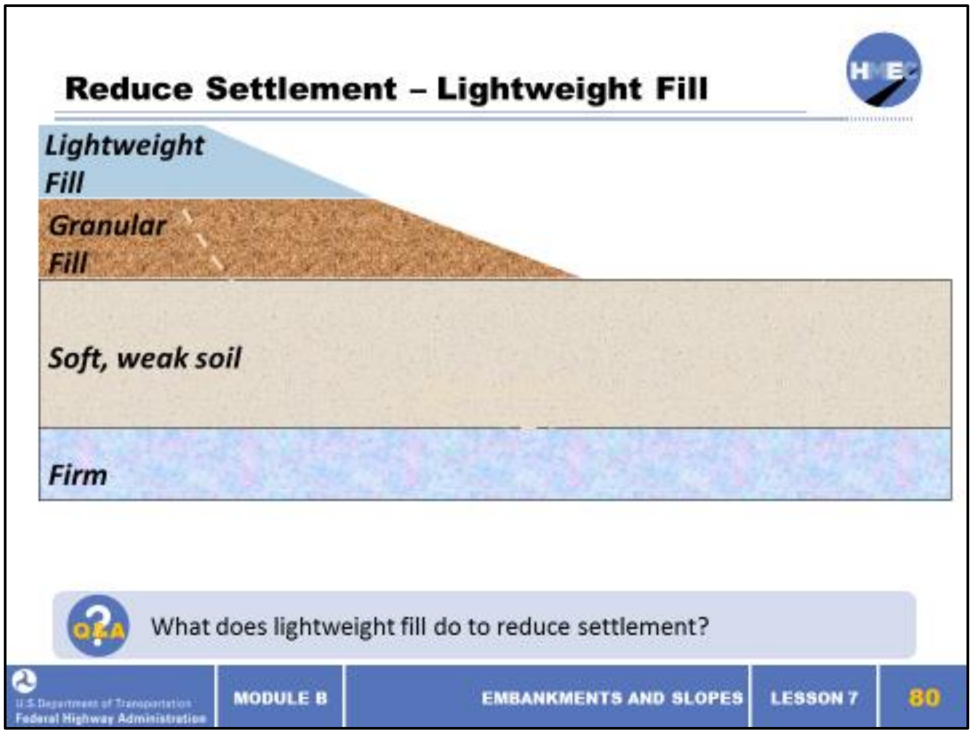
Slide 79



If the magnitude of settlement is the problem, we must reduce the forces causing settlement, reduce the compressibility of the problem layer, and design the geotechnical feature to handle settlement, or bypass the soft soils with a column/pile supported embankment. We could also incorporate a combination of those solutions.

If the time rate of settlement is the problem, we must either shorten the drainage path or accelerate the consolidation process.

Slide 80


Reduce Settlement – Lightweight Fill
HMEC
Lightweight Fill
Granular Fill
Soft, weak soil
Firm
Q&A
What does lightweight fill do to reduce settlement?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
80

____

____

____

____

____

____

____

____

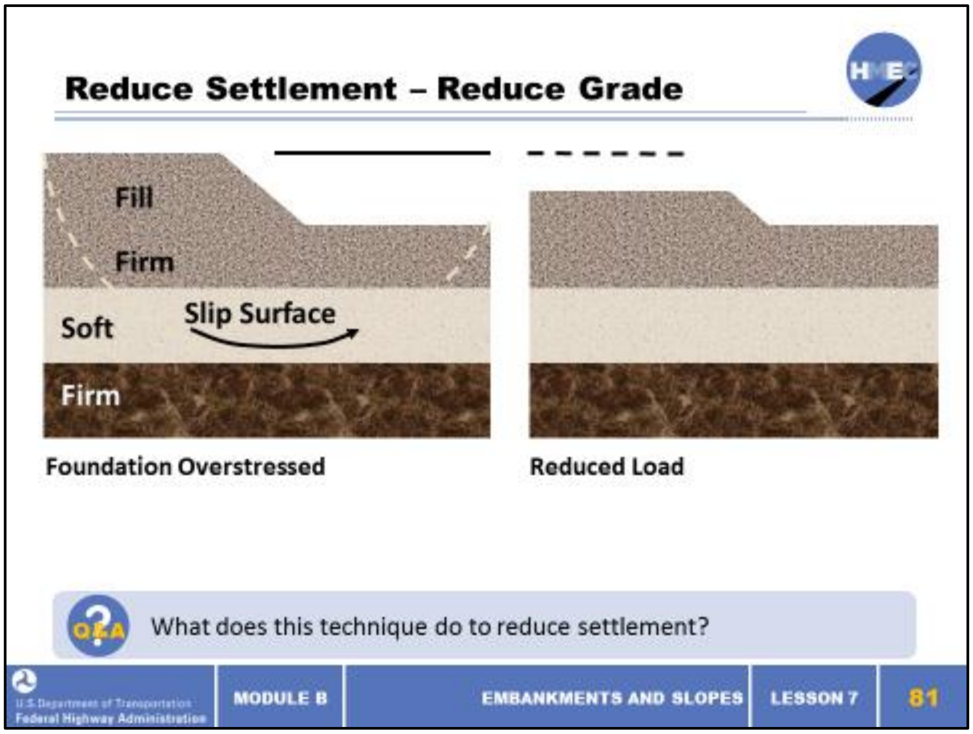
____

____

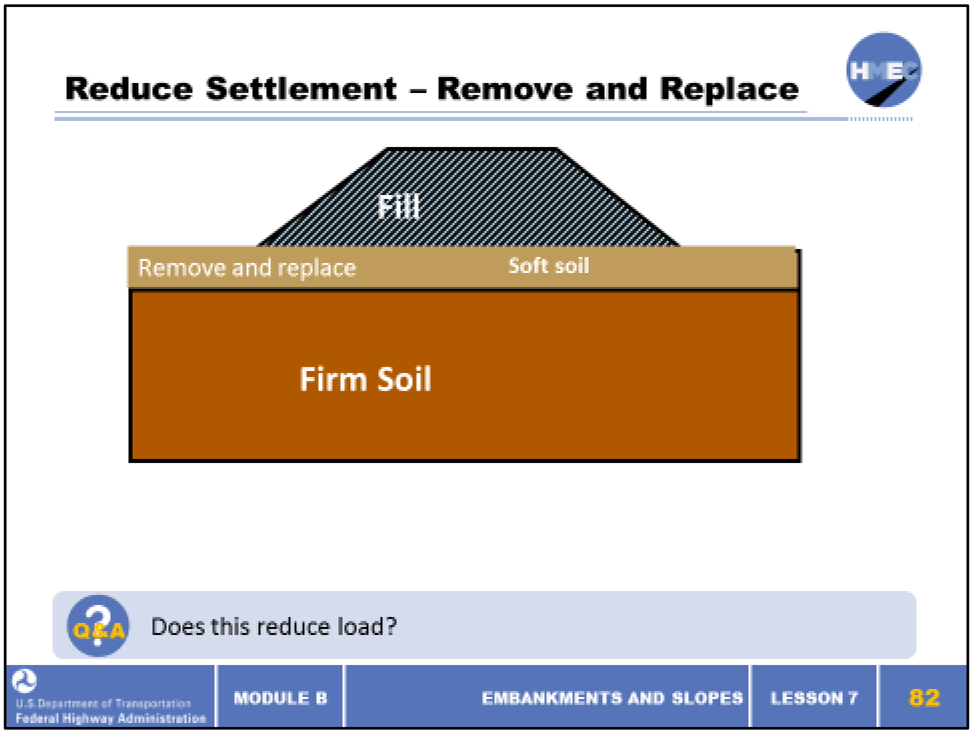
____

____

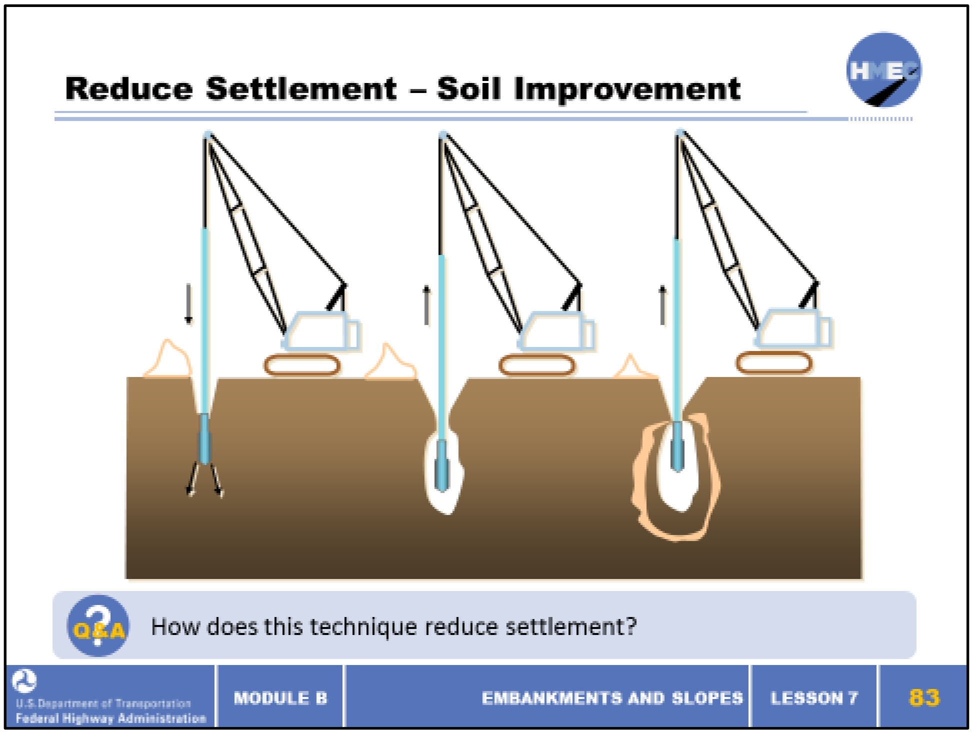
Slide 81


Reduce Settlement – Reduce Grade
HMEC
Fill
Firm
Soft
Slip Surface
Firm
Foundation Overstressed
Reduced Load
What does this technique do to reduce settlement?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
81

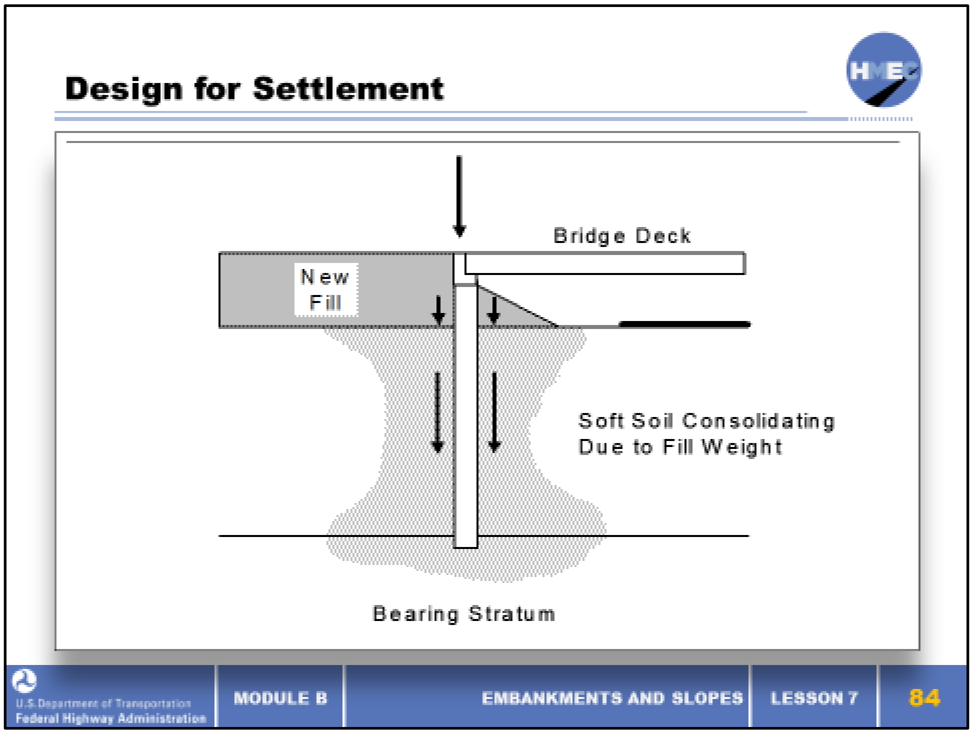
Slide 82


Reduce Settlement – Remove and Replace
HMEC
Fill
Remove and replace
Soft soil
Firm Soil
Q&A
Does this reduce load?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
82

Slide 83


Reduce Settlement – Soil Improvement
HMEC
Q&A
How does this technique reduce settlement?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
83

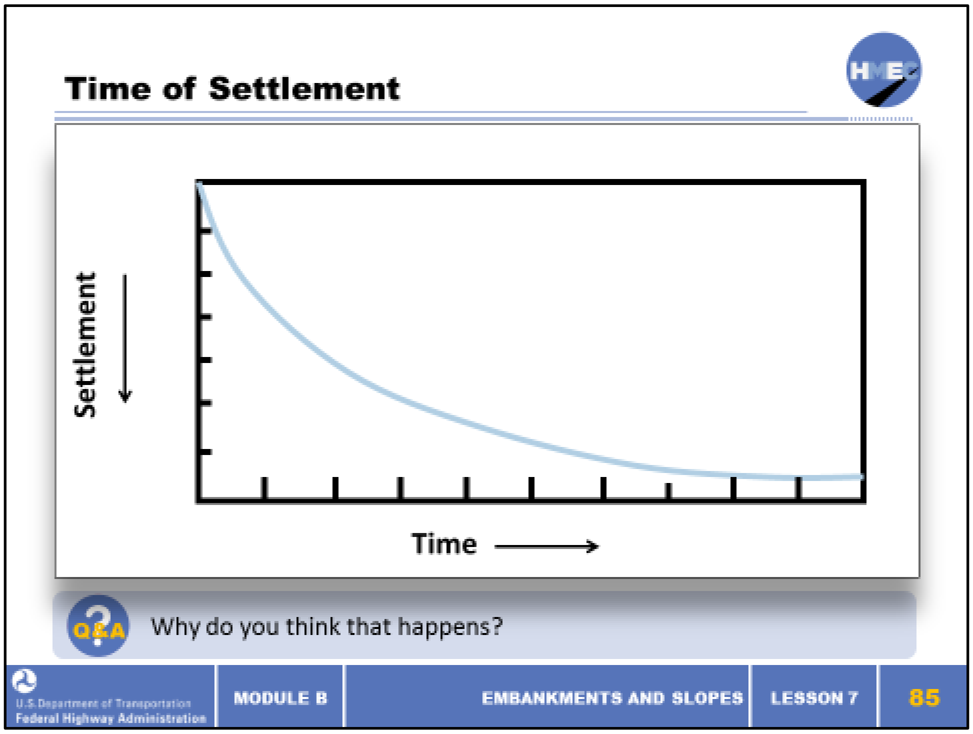
Slide 84



As little as ½ in. of settlement can create a down drag load on bridge piling. Since it might take months or years to get to the percent consolidation that would have less than ½ in. of settlement remaining, we may have to design the piling to withstand the increased load.

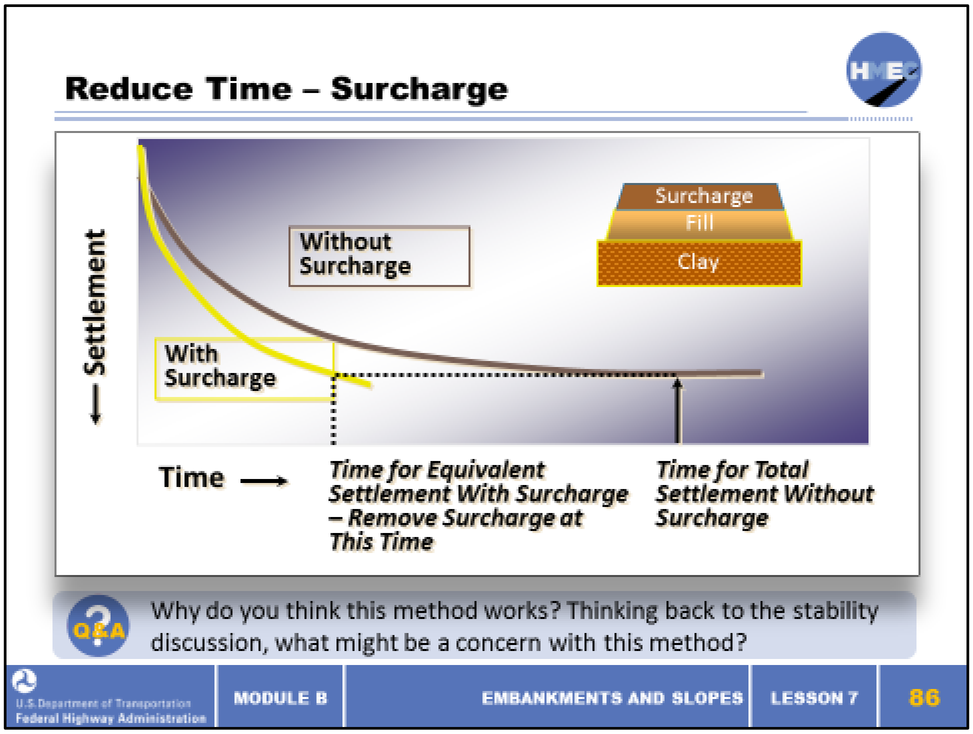
Solutions include letting the settlement occur before driving the pile, designing the pile to handle the increased load, pre-boring through the soft layer and isolating the pile, and coating the pile to reduce soil adhesion to the pile

Slide 85

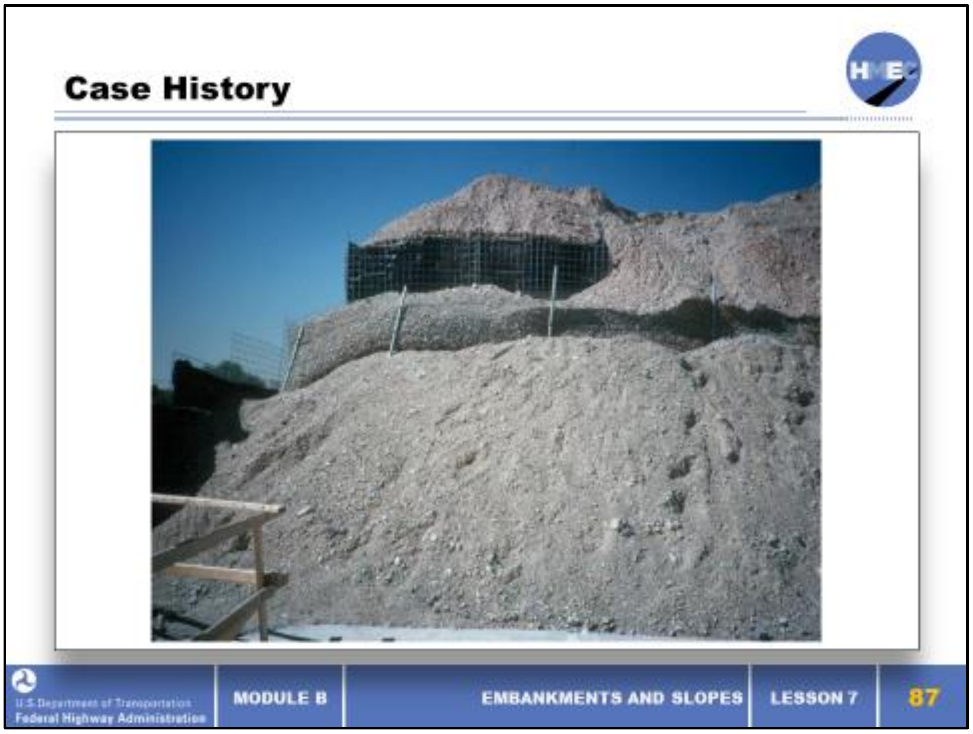


As can be seen by the time of settlement curve, as more time is allowed, more settlement takes place. However, the rate of settlement decreases with time. Soil strength also increases with consolidation, this is why stage construction works.

Slide 86


Reduce Time – Surcharge
HMEC
Settlement
Without Surcharge
With Surcharge
Surcharge
Fill
Clay
Time
Time for Equivalent Settlement With Surcharge – Remove Surcharge at This Time
Time for Total Settlement Without Surcharge
Q&A
Why do you think this method works? Thinking back to the stability discussion, what might be a concern with this method?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
86

Slide 87



This is a surcharge placed on the Salt Lake City, Utah I-15 project. Note that the lateral extent of the surcharge was maximized by using a soil reinforcement system to build the surcharge with a vertical face. Also note that a relatively high embankment was being surcharged at this location. The design attempted to maximize the surcharge height as the relative proportion of surcharge load was small compared to the total embankment load.

It doesn't have to look good, it will be removed.

____

____

____

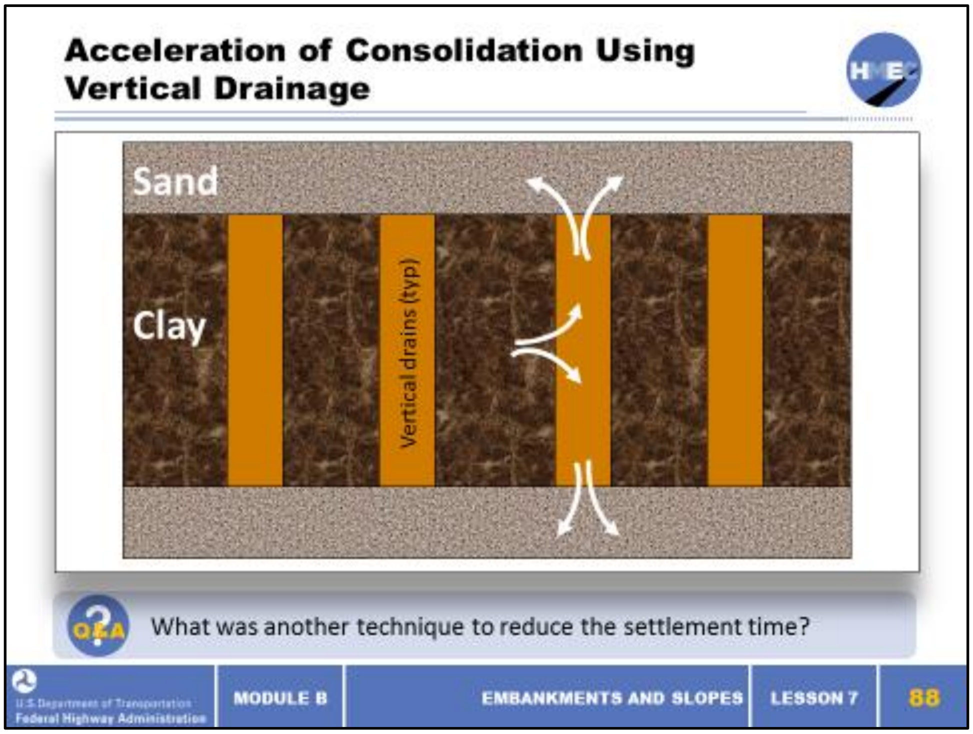
____

____

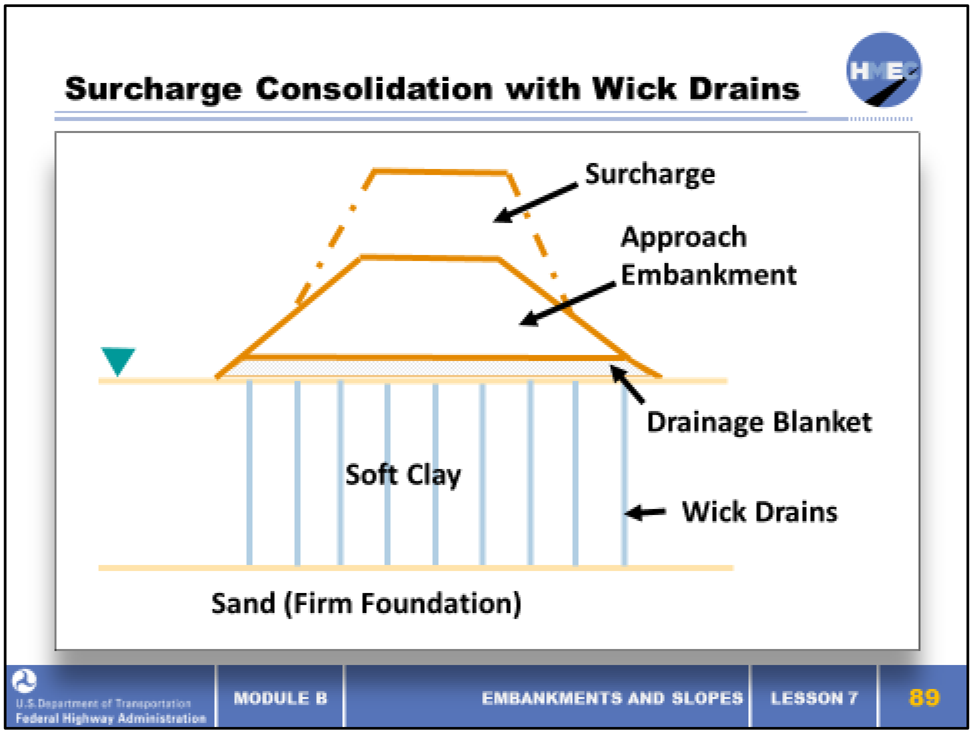
____

____

Slide 88

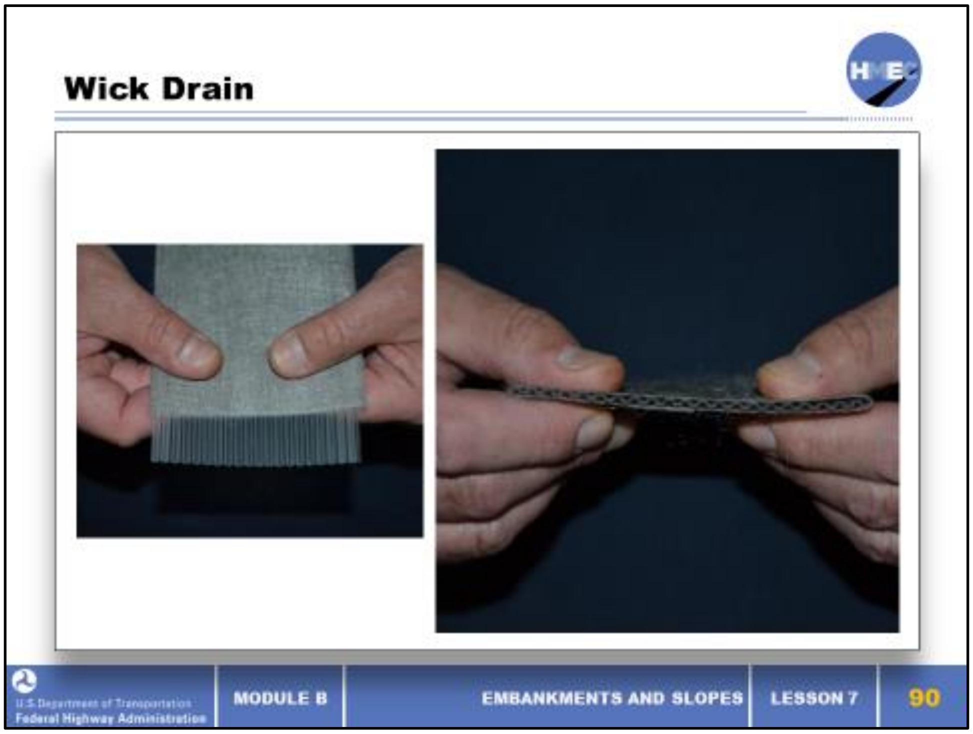

Acceleration of Consolidation Using Vertical Drainage
Sand
Clay
Vertical drains (typ)
What was another technique to reduce the settlement time?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
88

Slide 89

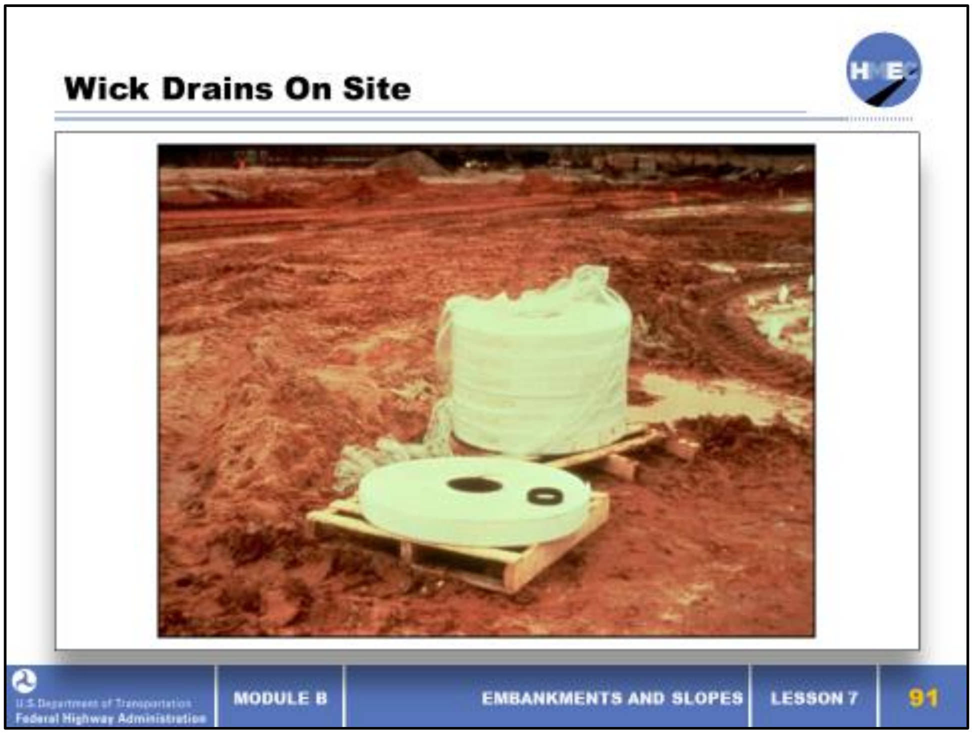


Wick drains accelerate the consolidation process by reducing the drainage path of the groundwater. The reduction of the length of the drainage path is a function of the wick drain spacing. The design can therefore optimize the spacing of the wicks to achieve a certain amount of consolidation in a defined time. By accelerating the consolidation, the amount of preload may also be reduced.

Slide 90


Wick Drain
HMEC
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
90

Slide 91



A 2–3-ft. sand blanket is placed over the site then the drains are placed. The sand blanket placed on top of the soft soil, to collect and drain water, also serves as the construction loading platform to support this installation equipment.

Because they don't have much structure as can be seen here, they are pushed/vibrated into the ground with a mandrel.

____

____

____

____

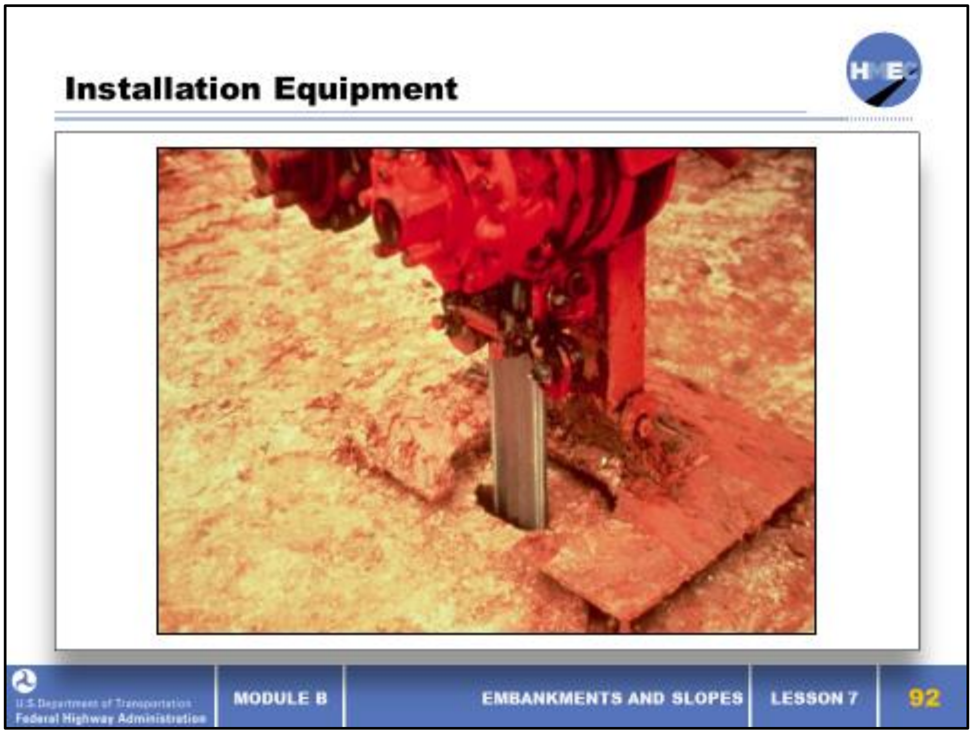
____

____

____

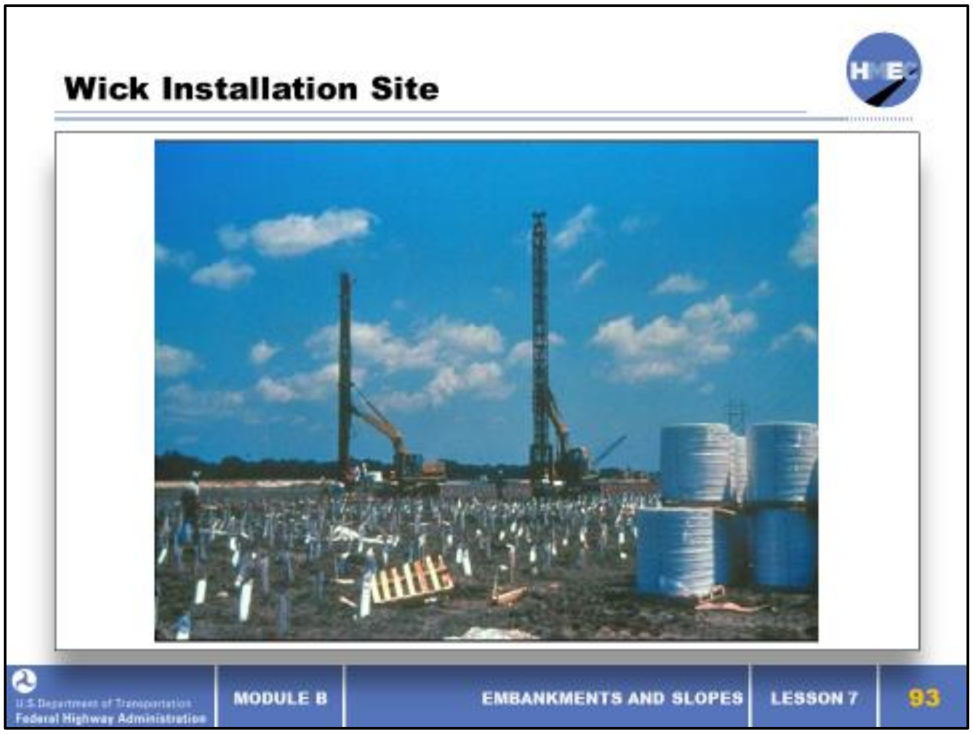
____

Slide 92



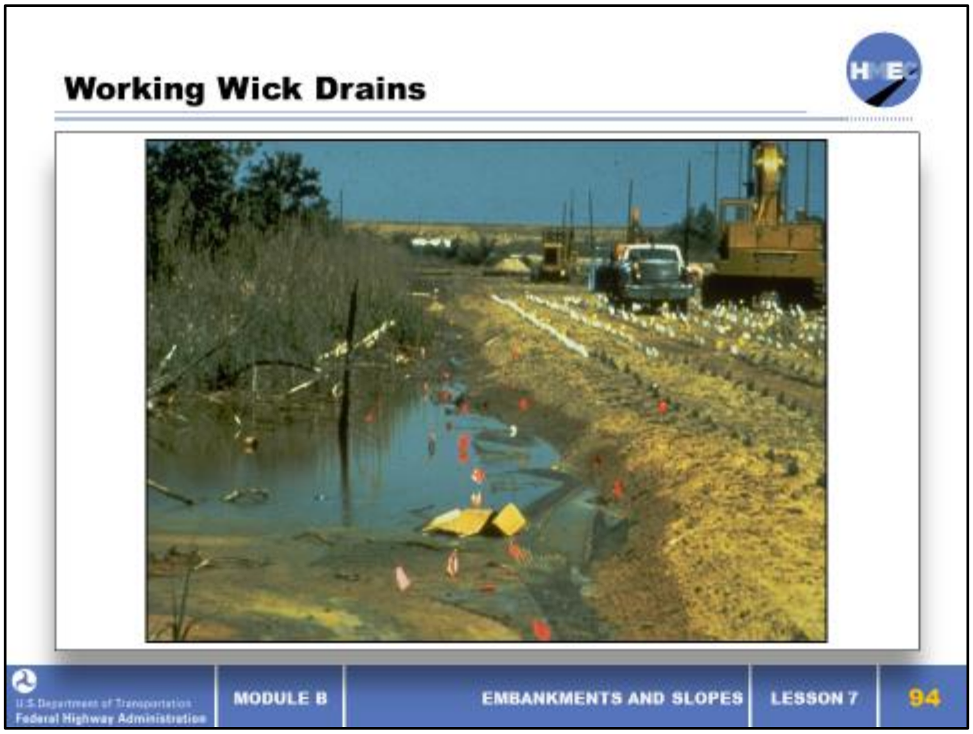
This shows the mandrel fully inserted into the ground. Note that a prefabricated drain can be placed at average rates of 40 ft. per minute depending on conditions. Length of the drain may be limited by the equipment as the mandrels tend to be relatively thin and flexible. Also compact surface deposits may need to be pre-augered to allow mandrel penetration.

Slide 93



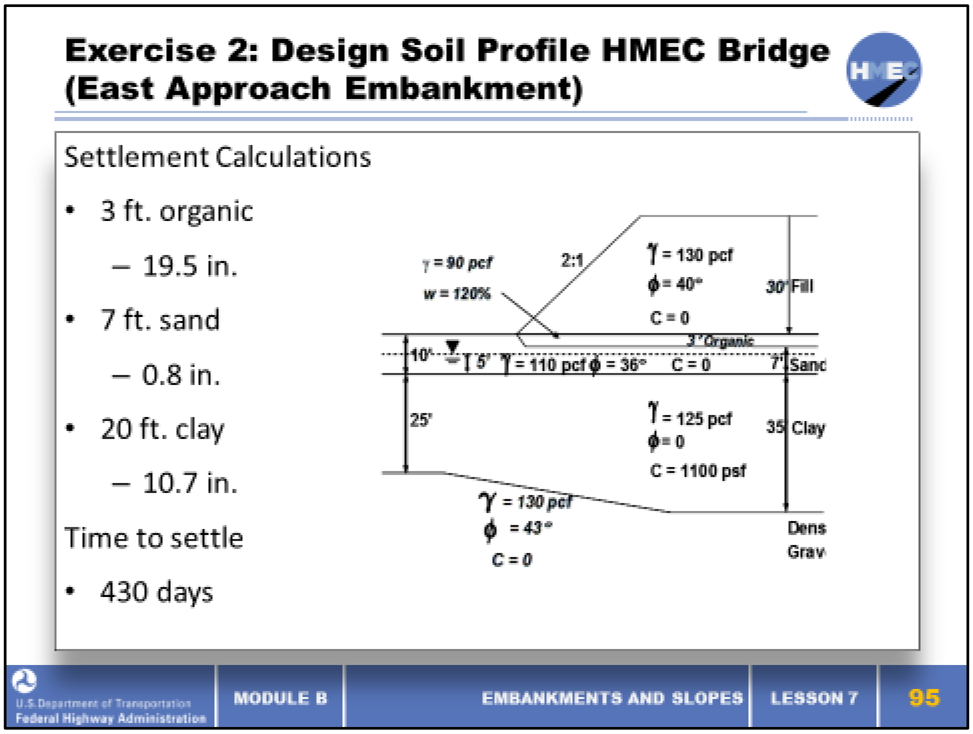
Installation in progress.

Slide 94



A drainage layer is required between the foundation soil and the surcharge soil to facilitate drainage. Here you see drainage occurring with just the placement of the drainage layer. The full approach embankment load has not been applied yet and already water is being squeezed out of the soft foundation soil.

Slide 95



Our approach embankment at the HMEC Bridge had significant settlement issues. Let's apply what we have learned about reducing settlement and time rate of settlement to our HMEC project.

____

____

____

____

____

____

____

____

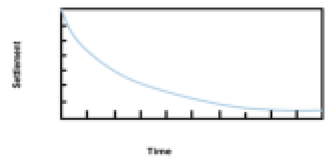
____

____

Slide 96

Exercise 2: Settlement Solution Matrix

$$t = \frac{T_v H_d^2}{c_v}$$



	Reduce Settlement			Reduce Time rate of settlement	
Embankment Type	Remove and Replace	Lower Grade	Lightweight Fill	Surcharge	Wick Drains
Side Slope at HMEC Bridge					
End slope at HMEC Bridge					

Let's break into groups for an exercise. Take 10 minutes to complete the matrix.

U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 96

Complete the matrix with techniques appropriate for the site that will reduce settlement and reduce the time for settlement to occur.

____

____

____

____

____

____

____

____

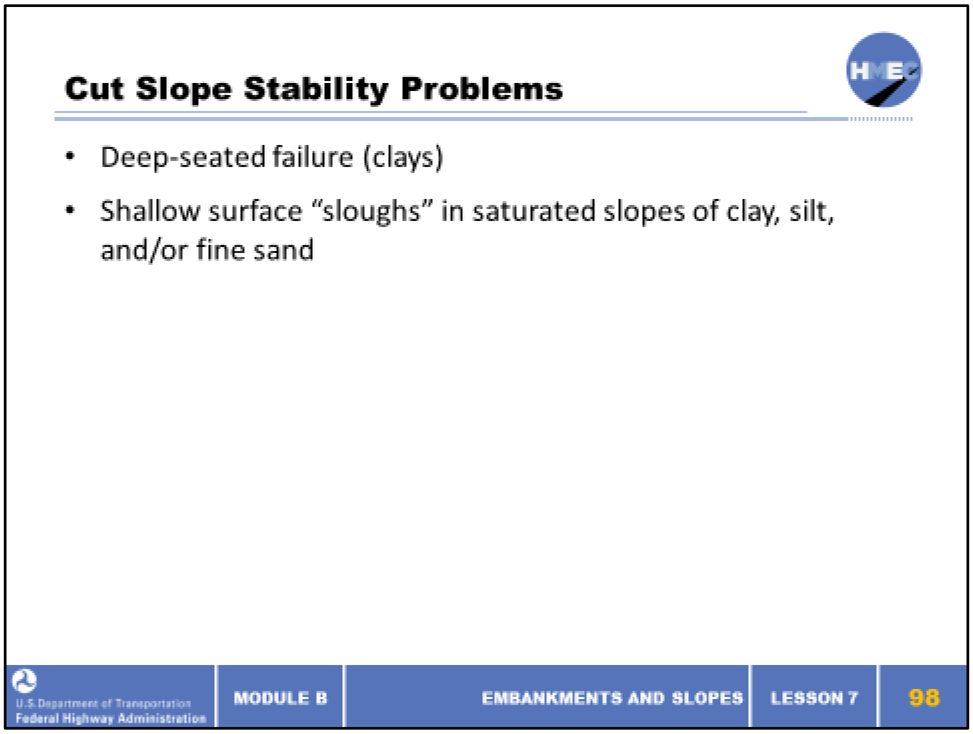
____

____

Slide 97

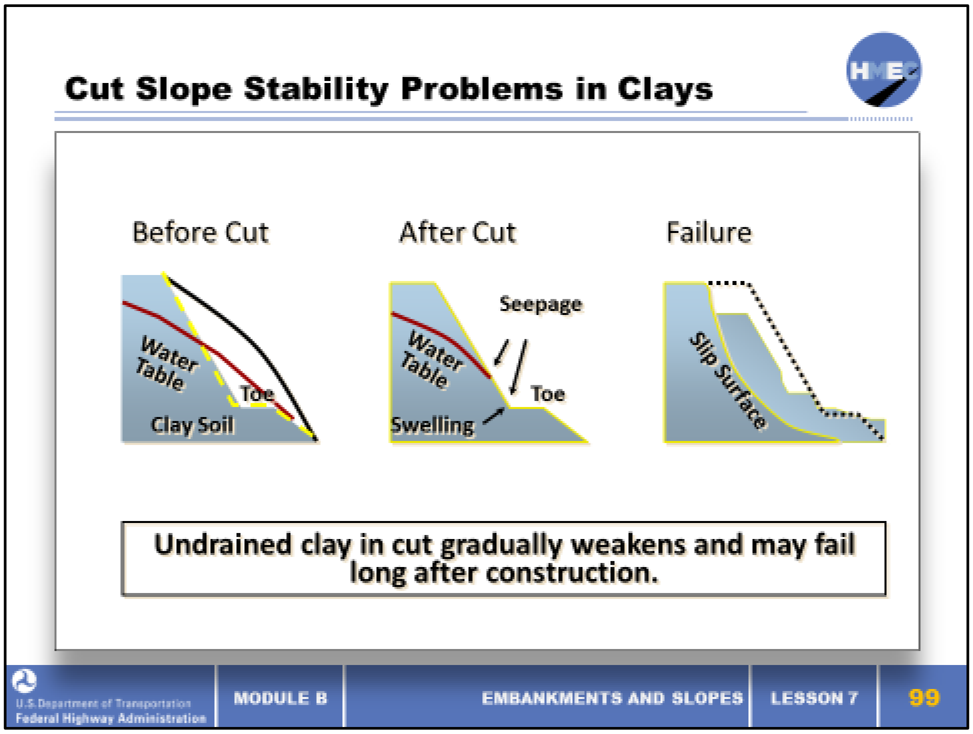

Exercise 2: Possible Settlement Solutions - Debrief
HMEC
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
97

Slide 98



Cut slope stability problems can be shallow sloughs, which we have already discussed, or they can be deep-seated failures in clay. You often find that water, pre-existing failure planes, and impermeable layers are involved in cut slope failures.

Slide 99



You need to know where the long-term water table is to ensure long-term stability of a cut slope. As we discussed earlier, after we remove the overburden, any water present now or in the future will weaken the clay slope. Hopefully impermeable layers will be detected during the site investigation, but if not, construction personnel needs to pay close attention to cut slope excavation. They are the last line of defense against a failure that results from an impermeable layer that daylights on the cut slope and carries water to the slope.

Cutoff ditches and sub drains can be effective techniques to intercept water before it becomes a problem.

____

____

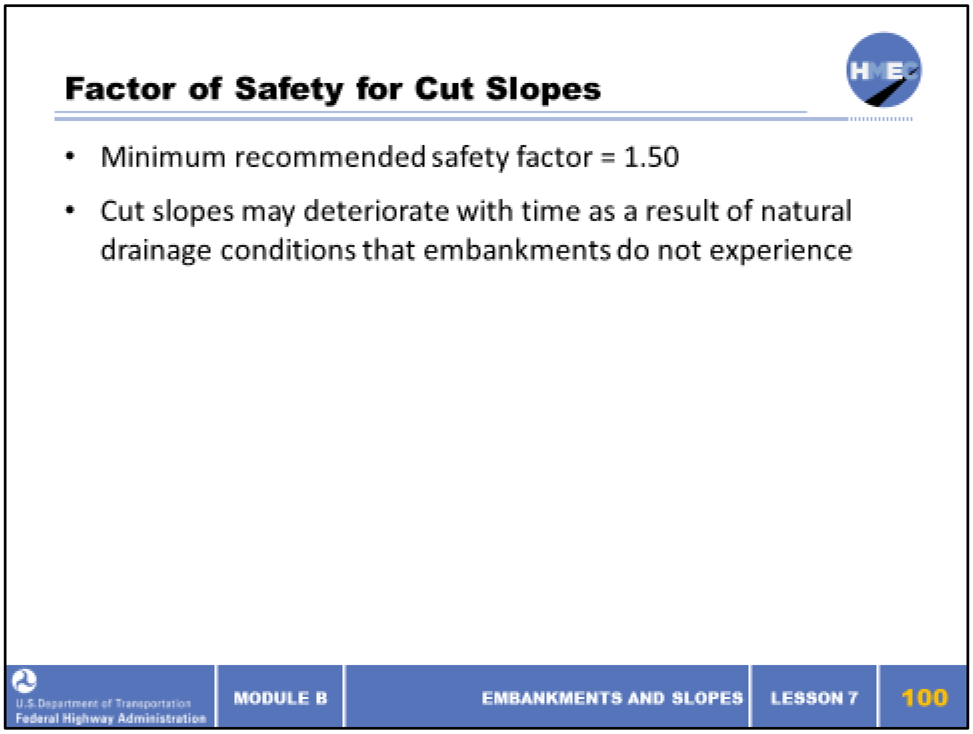
____

____

____

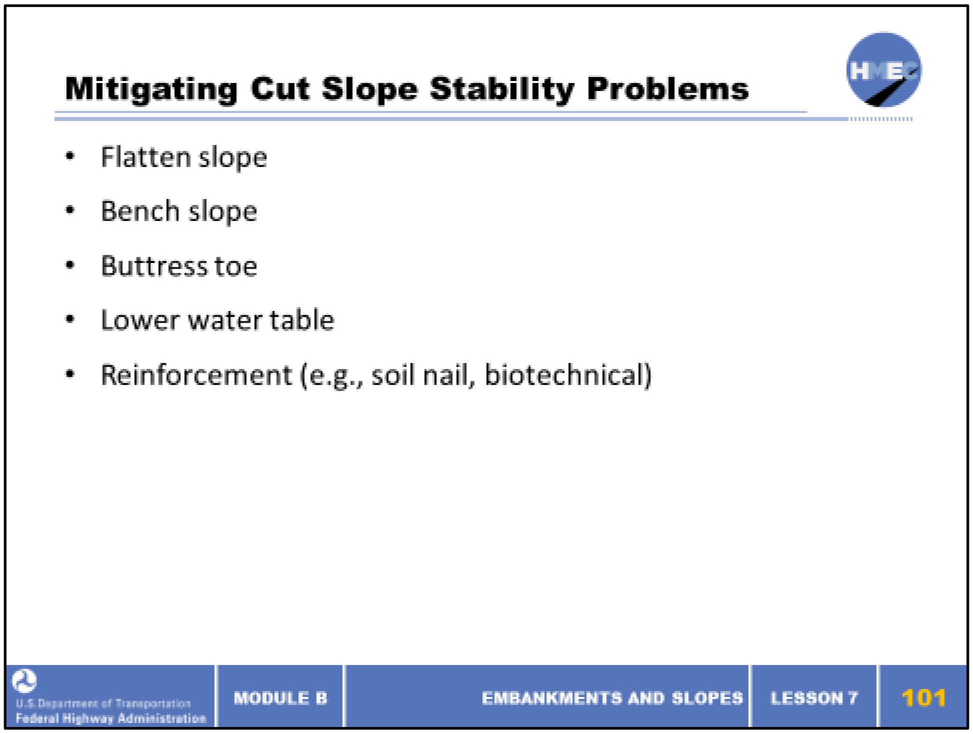
____

Slide 100



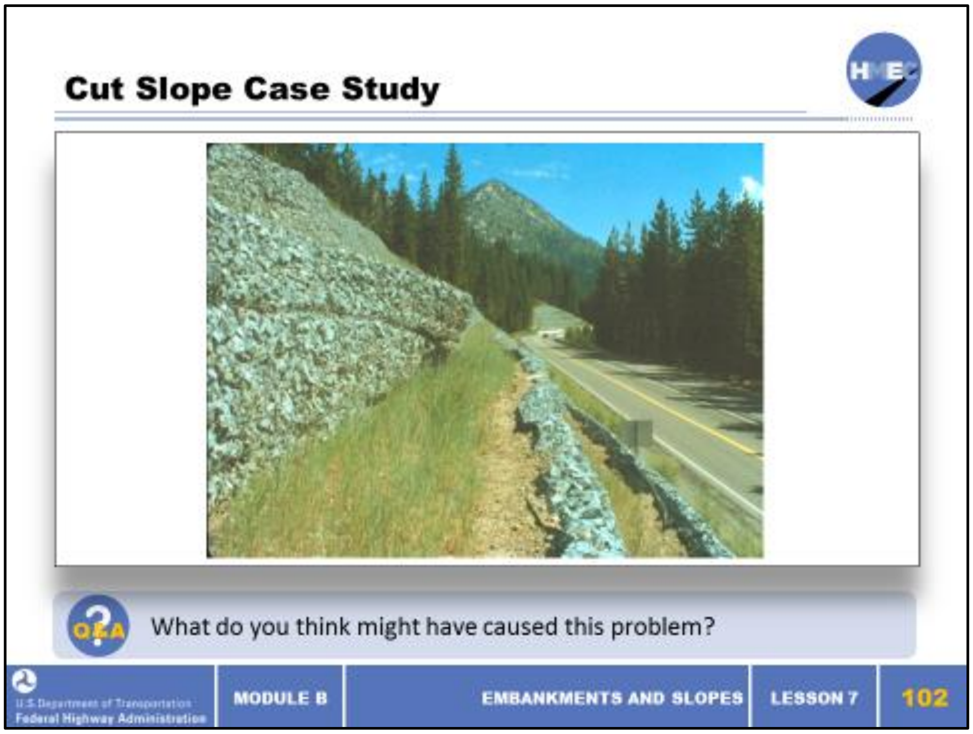
The effect of water and loss of strength justify the higher FS for cut slopes.

Slide 101



The first three solutions are the bread-and-butter solutions used by highway agencies. The latter two solutions are methods that may require specialists for both the design and construction of the solution.

Slide 102



Point out the bulge in the gabion wall on this cut slope.

___

___

___

___

___

___

___

___

___

___

___

Slide 103

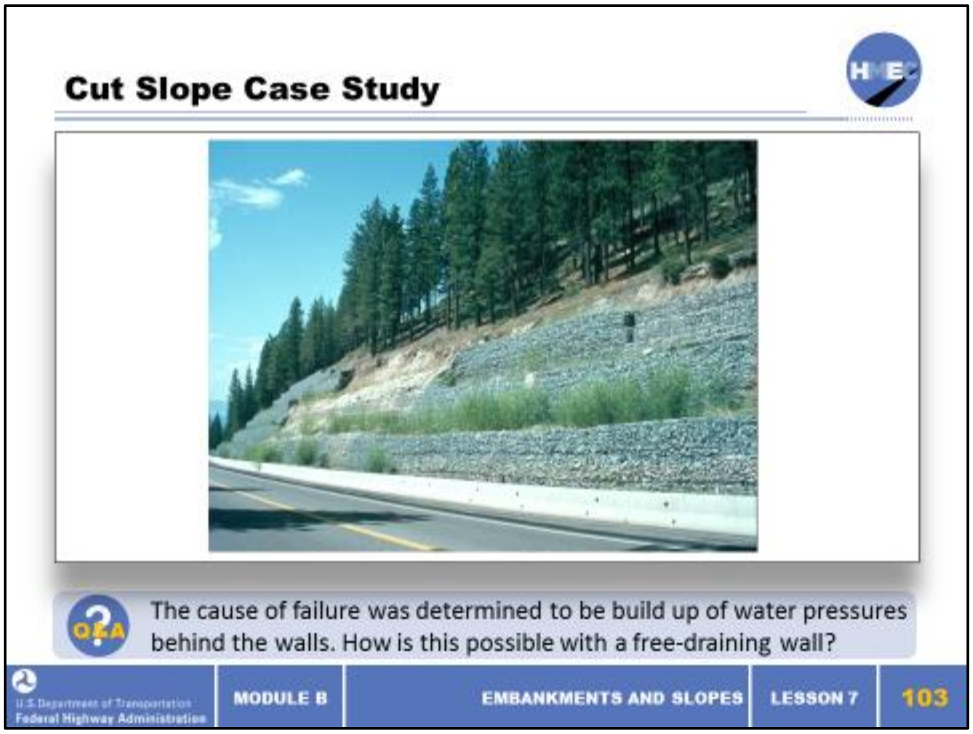


Photo of the same case study cut slope, showing the failed section of gabions.

____

____

____

____

____

____

____

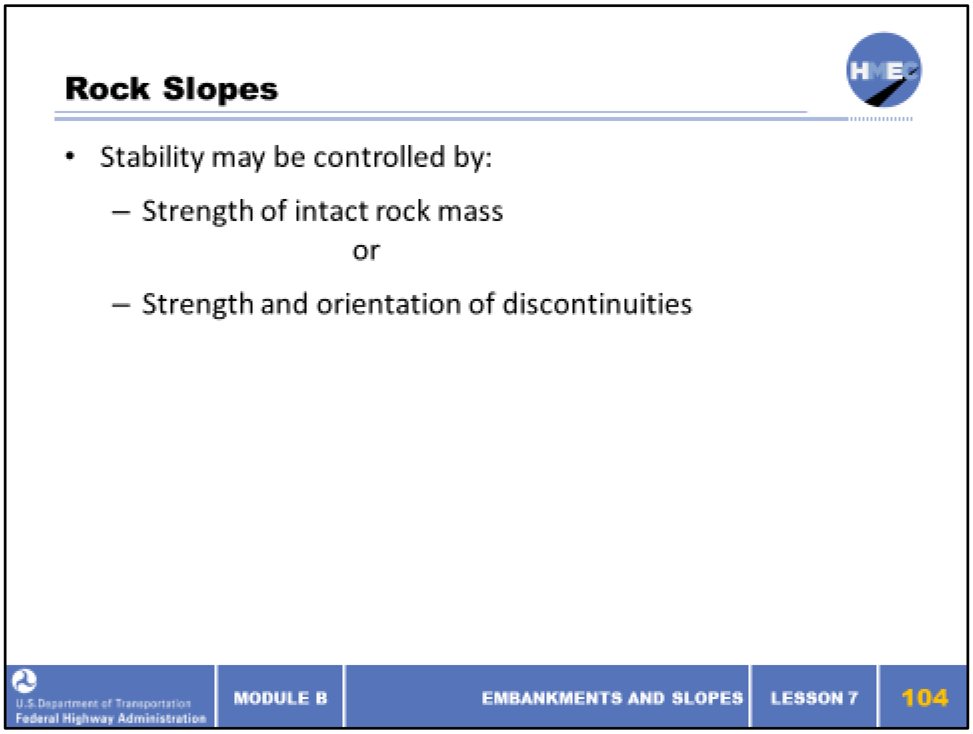
____

____

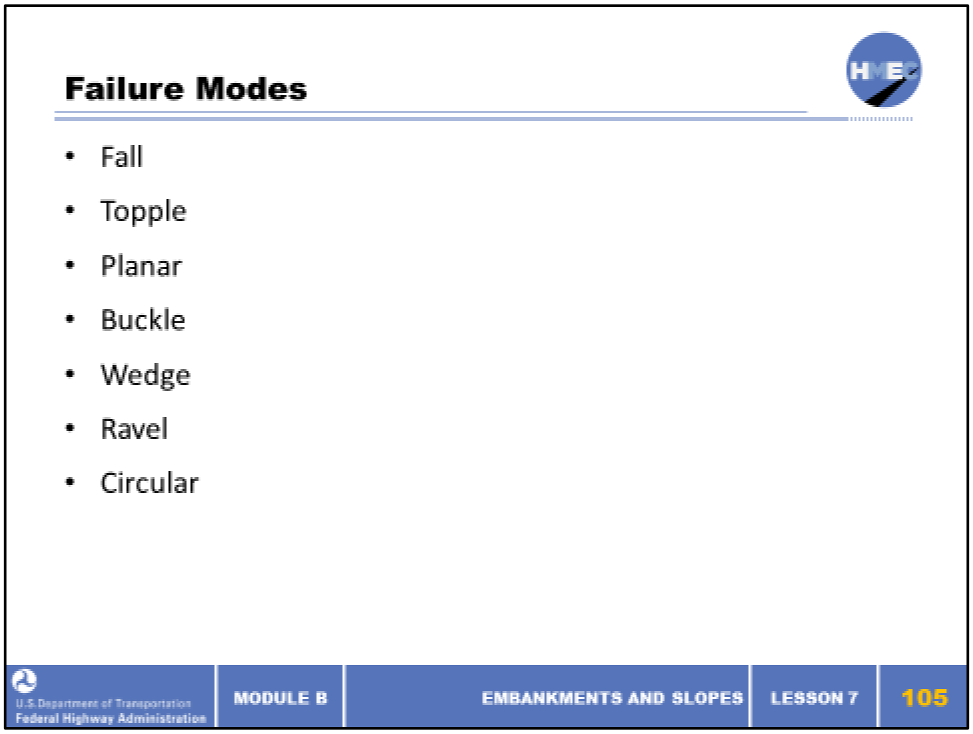
____

____

Slide 104

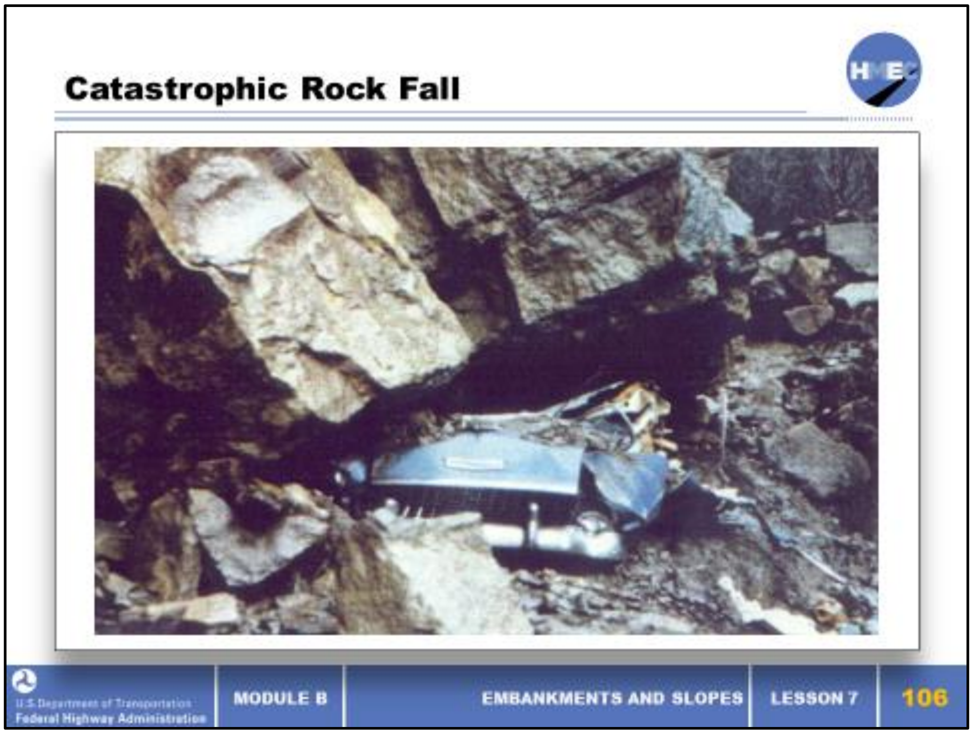

Rock Slopes
• Stability may be controlled by:
– Strength of intact rock mass
or
– Strength and orientation of discontinuities
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
104

Slide 105



Listed are several failure modes for rock slopes, we will only discuss a few of the more common failures modes.

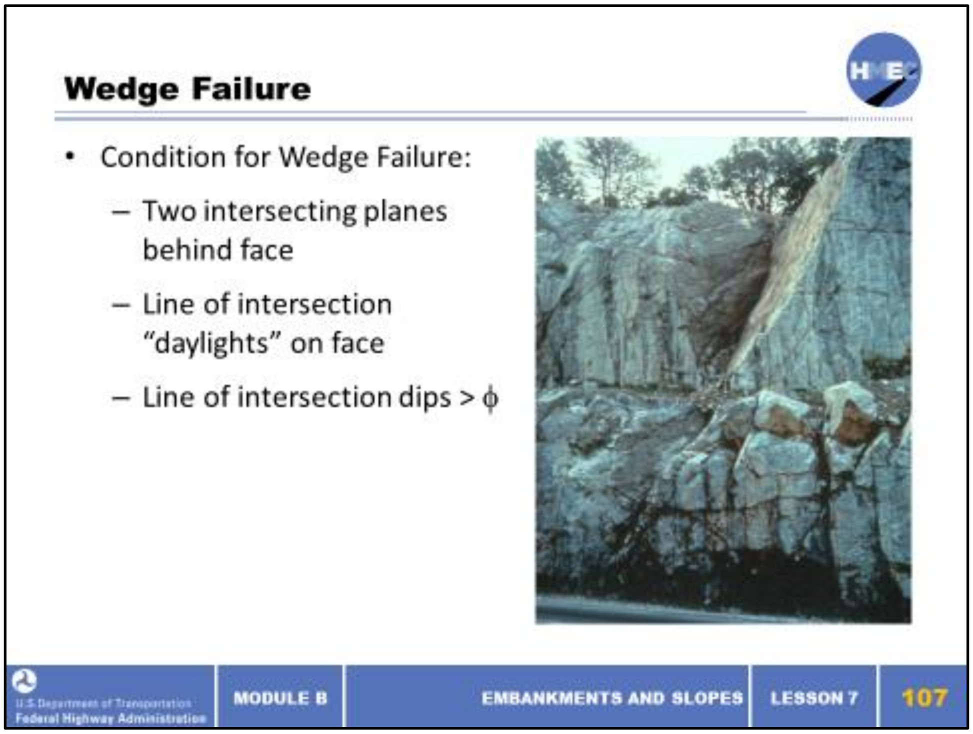
Slide 106



Rock falls are a major geological hazard in many States with mountainous or hilly terrain. Catastrophic failures of rock cuts can result in property damage, injury, and even death. Water is the major cause of rock falls. For example, rain, freeze-thaw, snowmelt, erosion, seepage, and root growth. Most rock falls occur during high rainfall periods, and spring and fall when the slopes thaw. California records show that about two-thirds of the falls were triggered either directly or indirectly by water, which is the single greatest cause of rock falls.

Earthquakes can cause rock falls and landslides, and ground motions should be taken into account when designing cuts in seismically active areas.

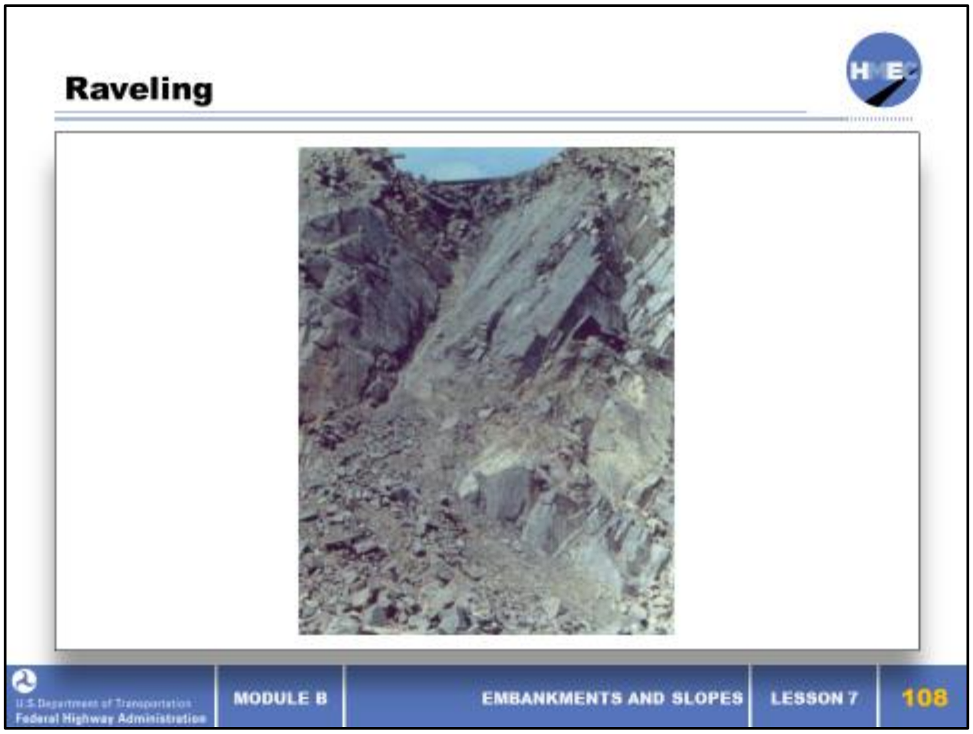
Slide 107



This is an example of a wedge failure. The wedge has been formed by two joints that intersect behind the face such that the line of intersection "daylights" in the face. Sliding has occurred because both joints are continuous over the full height of the upper bench, and the line of intersection dips steeper than the friction angle. It is likely that wedges have not formed elsewhere on the face because the other joints are not persistent over the full height of the face; the portions of intact rock on the joint surfaces provide sufficient shear strength to prevent failure.

The friction angle of the joints play a role in the stability of this feature, pointing out the contribution that water and water pressure will have on stability.

Slide 108



Most rock falls are of the raveling type and not conducive to stability calculations, and even the failure mechanisms are not well understood.

In many terrains, the discontinuities are oriented in such a way that they contribute to create wedge, planar sliding, or toppling failures, which have well-accepted methods to analyze stability. Raveling failures, defined as time dependent regressive displacement failures, are more problematic. There are no analysis techniques for prediction, and remediation designs are typically based on engineering judgment, balancing the risk in terms of probability of failure and consequence of failure against the cost of effective remediation.

Slide 109

Scaling and removal involves the removal of rock that has the potential to fall and impact the safety of the traveling public or the function of the transportation facility. Mechanical support is often used to hold the rock mass in place on the rock slope. Protection and catchments are also frequently used to prevent the failing rock mass from encroaching on the roadway and/or endangering the traveling public.

Rock slope stabilization may be required in strong massive rock as well. Rock fall in very strong, massive rock are more dangerous than falls of weak, closely jointed rock. The boulders did not break up significantly as they fell down the slope and their energy is sufficient to demolish an unreinforced concrete wall at the toe of the slope; this demonstrates that protection measures should be flexible to absorb energy.

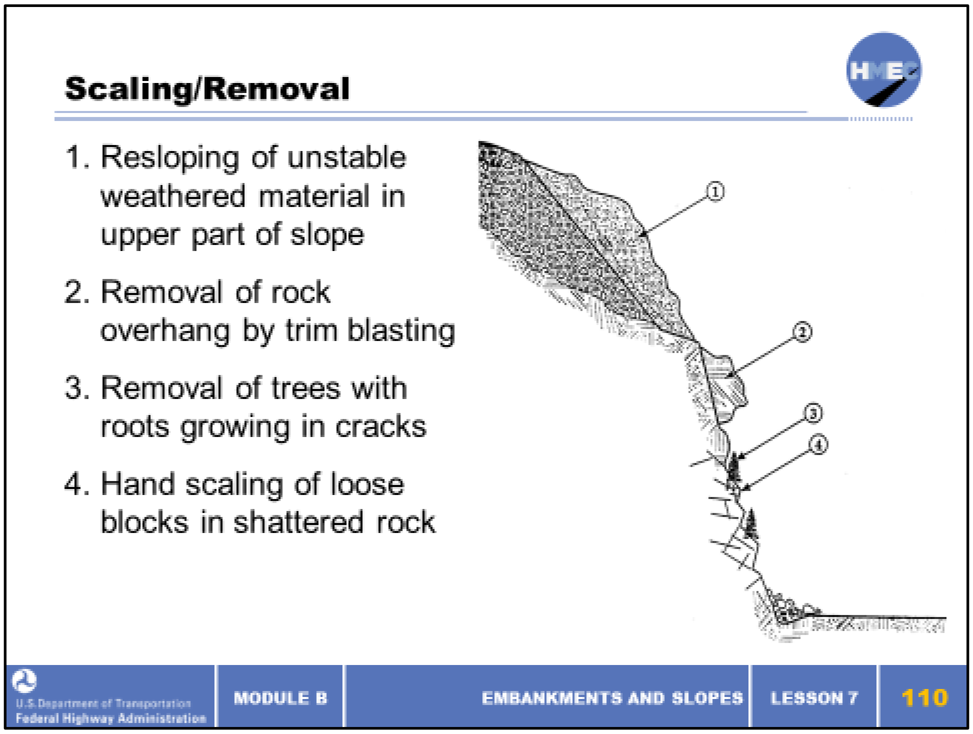
____

____

____

____

Slide 110



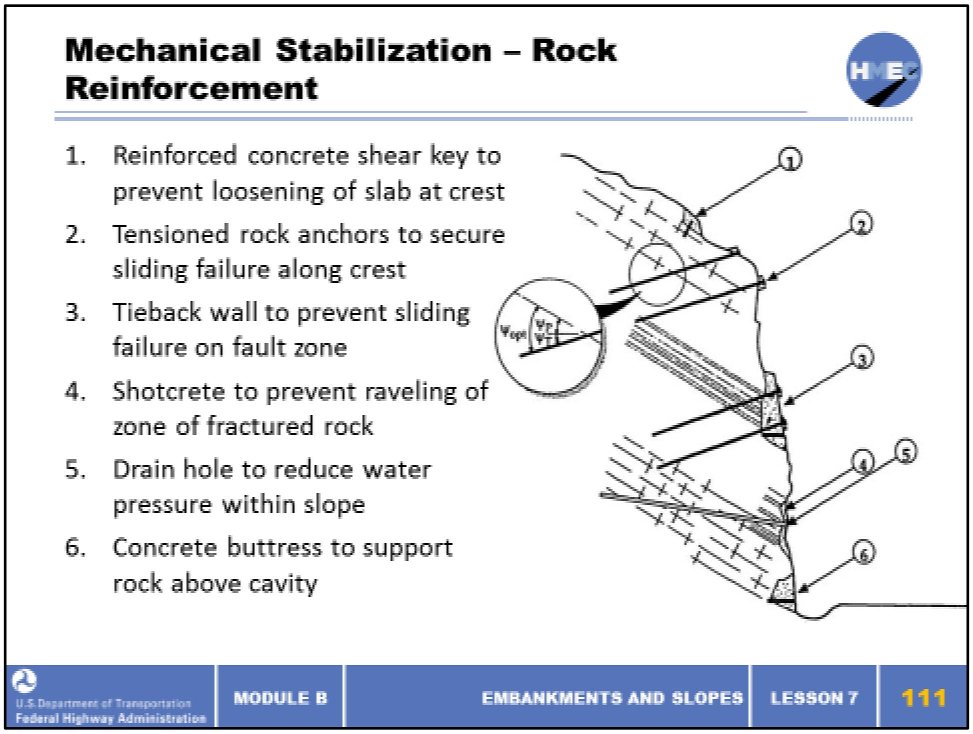
Instability of unconsolidated material overlying rock cuts is often a greater contributor to rock fall than the rock slope itself. Too often slope designers pay attention to the rock cut design while ignoring these overlying soil or weathered rock materials. If the natural slope above the cut is not overly steep, slope reduction in the unconsolidated material may be possible.

Trim blasting is analogous to cushion blasting and involves the firing of small-diameter holes along a neat excavation line, for example to remove an overhang. If there are any vibration sensitive structures in the vicinity of a blast, vibration monitoring should be stipulated.

Trees on rock slopes have two destabilizing properties. Firstly, as the tree grows the roots seek out the discontinuities where the infillings retain moisture. This results in a long term wedging action across the discontinuity as the root grows. Secondly, as the tree grows to tens of feet in height it experiences wind loading that produces a pry-bar effect on the rock.

Scaling is the simplest, least expensive, and in many cases, the most effective means of slope stabilization.

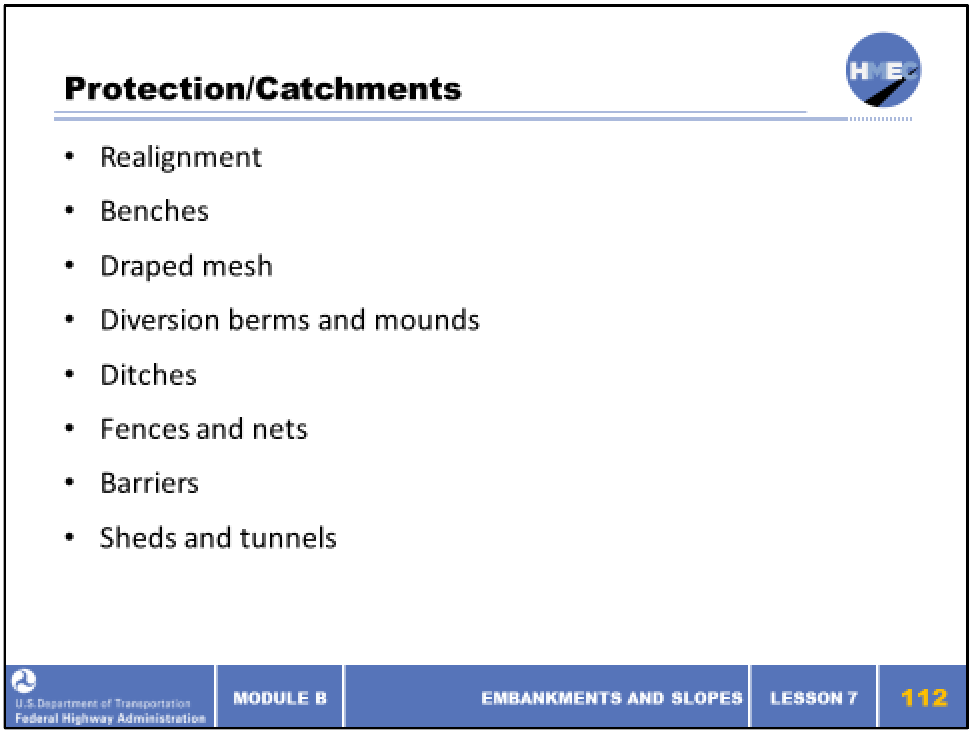
Slide 111



All of the techniques listed here and others are common to stabilize rock slopes. Read the bullets and comment on a few. For instance, note the drain to prevent groundwater from building up excess pressure, and the buttress to prevent further undercutting and possible collapse.

NHI-132035, Chapter 10 is a good reference.

Slide 112



Many of these methods to protect the facility have been tried, all with different degrees of success depending on the conditions.

Mesh: Unless the mesh is vinyl coated to match the adjacent rock, its reflectivity can be a visual intrusion as in this case. The objective of mesh and cable net is to enable the rock to work its way to the bottom of the slope in a controlled manner beneath the loose mesh. The intent is not to pin the mesh to the slope at regular intervals so that the loose rock is suspended on the face. Such a practice leads to local overloading of the mesh as rock accumulates and leading eventually to failure. Twisted wire mesh is used. Chain link is also used but it has a tendency to unravel if it is cut. At this time there is not a design procedure for mesh installations. A rule of thumb is that mesh is effective up to a block size of about 0.7 meters. Other design issues include anchorage capacity along the crest, corrosion protection, and “stitching” of adjacent panels with wire rope to create a continuous blanket. The mesh should extend to within a few meters of the ditch

Where the block size is larger, a cable net installation is preferable. Cable net consists of a wire rope grid (typically 200 x 200 mm up to 300 x 300 mm) backed with chain link mesh.

Ditches: Ditches are to protection measures as scaling is to rock removal methods: highly functional and cost effective. The effective depth of a catchment area (ditch) can be increased by the placement of a barrier on the outside. The barrier should be flexible and energy absorbent rather than rigid. Gabion baskets can be effective in this regard. The application is probably more suited to railroads than to highways where snowplows or other equipment could breach the basket wire. Another method of augmenting the ditch depth is to place a fence along the outside. This application also has the drawback of potential interference with maintenance equipment, particularly snow removal.

Interlocking concrete blocks (also known as ecology blocks) are an effective way to construct a rock fall catchment area. The blocks are not physically attached to each other and hence have the ability to yield if struck by a rock. If a block is damaged, it is a relatively straightforward process to remove and replace it, especially on a railroad.

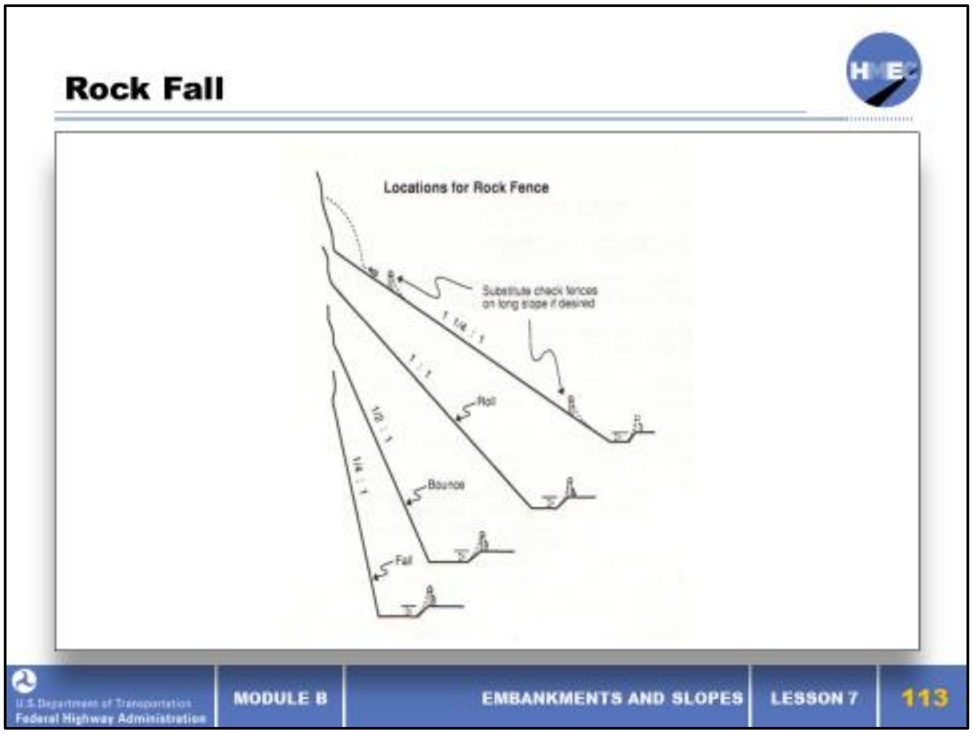
Cast-in-place concrete should be avoided as a rock fall protection measure. The concrete is rigid and usually less strong than the rocks that impact it, leading to the results shown.

In the 1980s, much work was done in Colorado to increase the utilization of recycled materials in rock fall protection designs. One of these included this MSE wall faced with tires for energy dissipation. Though highly effective, the required footprint is larger than most sites will accommodate. There may also be a slight aesthetic concern.

Fences: Upon impact, the fence is design to deform to absorb the translational energy of the rock. Most of the energy absorption takes place on the panel. Fencing can be placed on top of walls (such as MSE) to improve the performance of the rock catchment. Provides flexibility and storage.

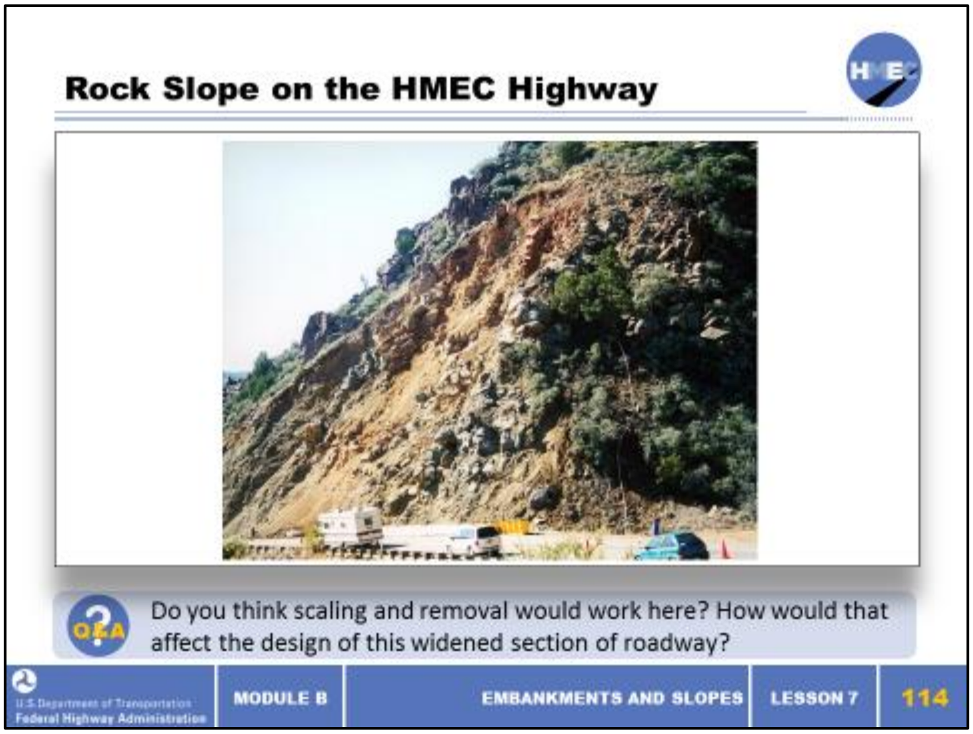
Sheds are built over the roadway. This technique can be effective if the rock fall area is narrow and rock fall can funneled to the shed. Tunnels are similar to realignment, expensive, and used only in extreme cases.

Slide 113

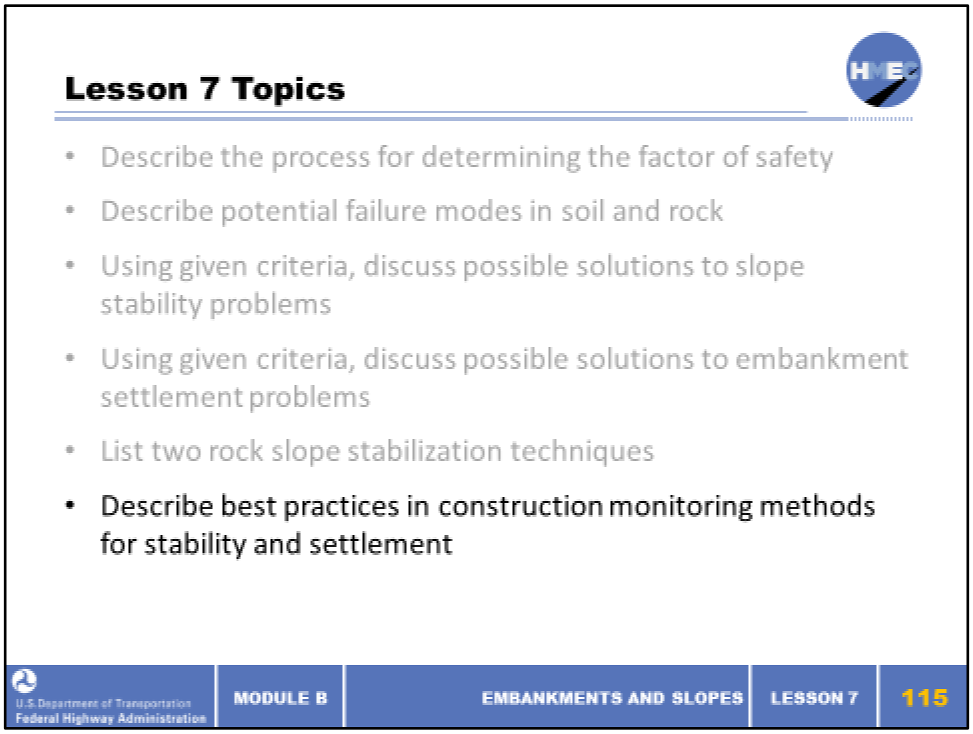


This is a diagram to provide insight into how the slope influences design of a fence or catchment.

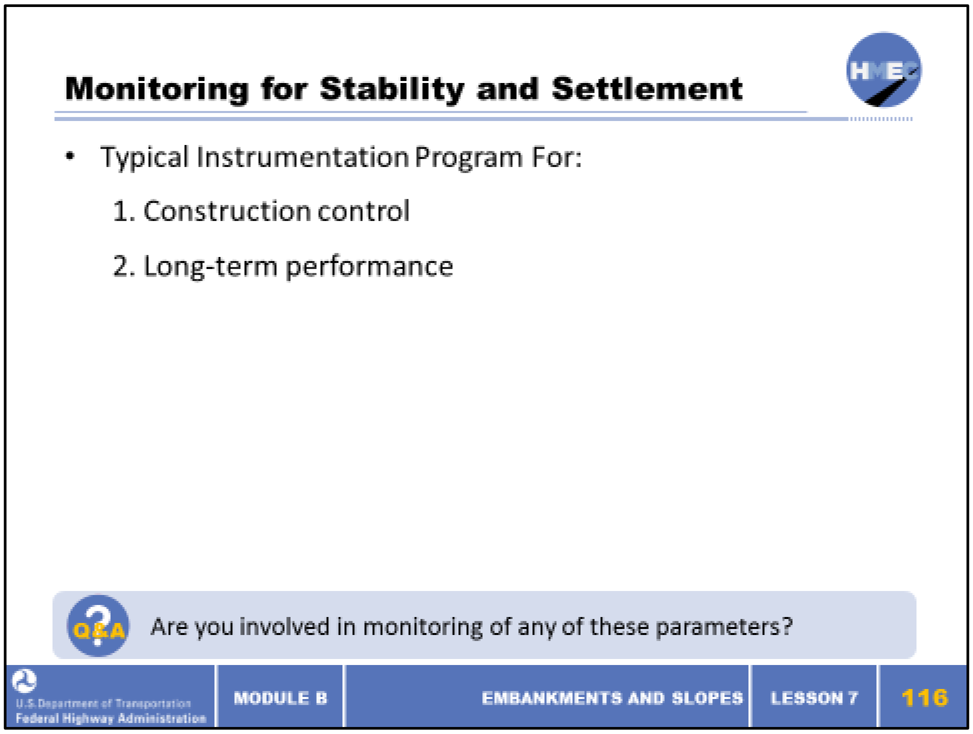
Slide 114


Rock Slope on the HMEC Highway
HMEC
Q&A
Do you think scaling and removal would work here? How would that affect the design of this widened section of roadway?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
114

Slide 115

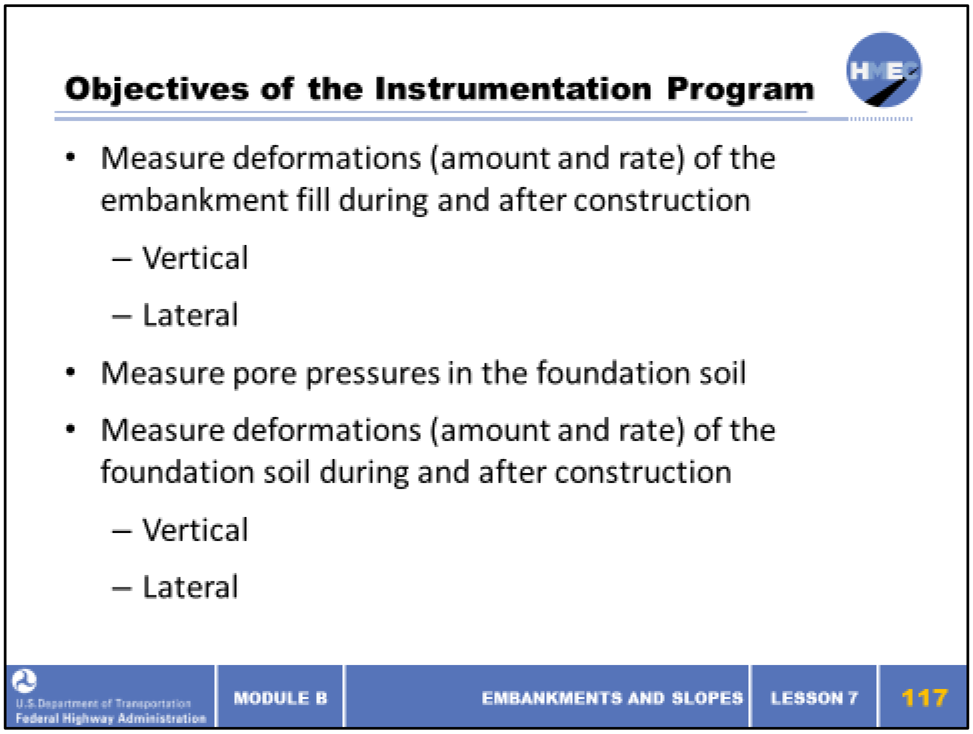

Lesson 7 Topics
HMEC
• Describe the process for determining the factor of safety
• Describe potential failure modes in soil and rock
• Using given criteria, discuss possible solutions to slope stability problems
• Using given criteria, discuss possible solutions to embankment settlement problems
• List two rock slope stabilization techniques
• Describe best practices in construction monitoring methods for stability and settlement
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
115

Slide 116



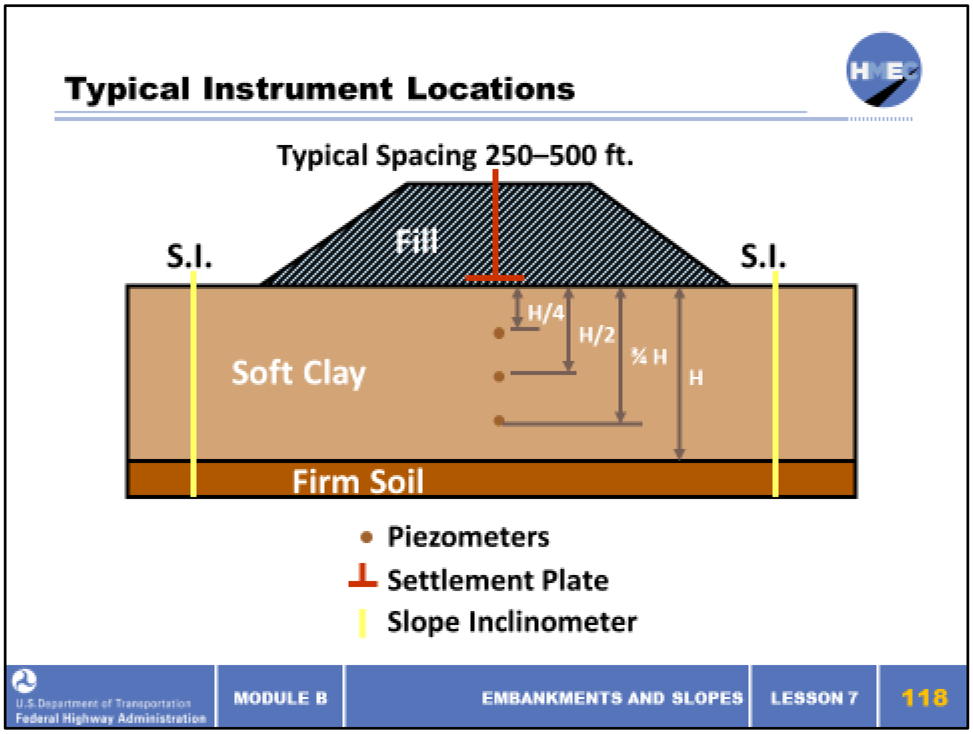
The most critical stage for slope stability may occur during construction. If we are waiting on settlement to reach a tolerable limit before proceeding with construction, we need to monitor settlement. If we are using stage construction or a surcharge, we will need to monitor pore water pressure. If slope stability is a concern, we may need to monitor horizontal movement of the slope.

Slide 117



These are the parameters we want to be able to measure. Let's look at how that is achieved. Note that if we are able to specify quality materials for embankment construction, the first bullet will not typically be a concern. However, often because of cost or availability, we are not able to specify quality materials and must use on-site material.

Slide 118



Shown on this slide is common instrumentation that may use to measure those parameters we need.

____

____

____

____

____

____

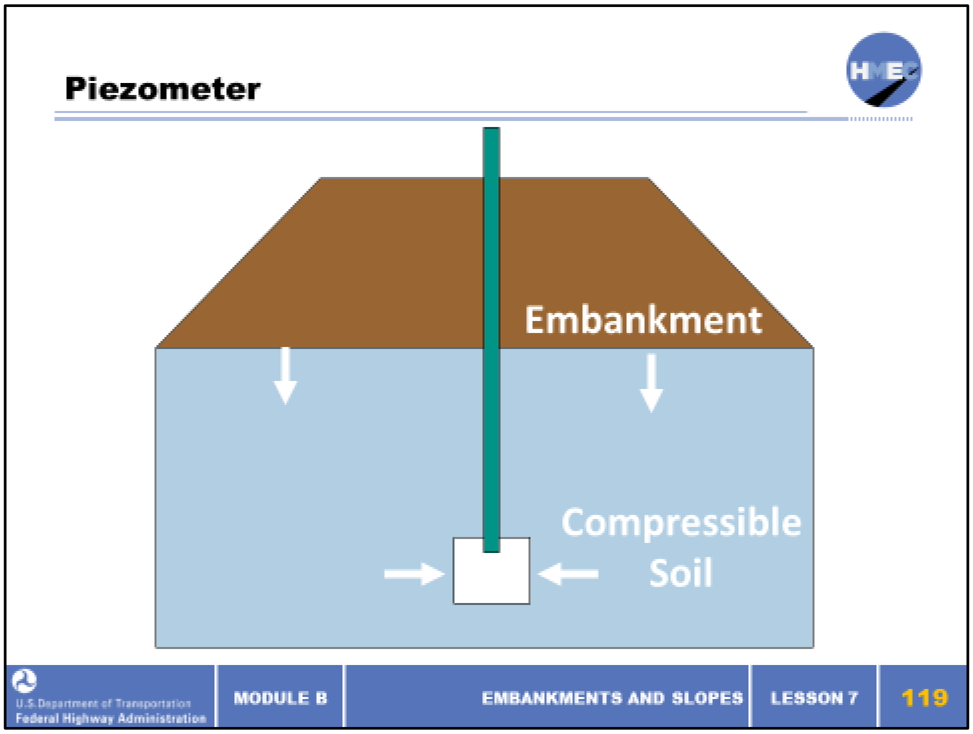
____

____

____

____

Slide 119



Piezometers are essential in determining the stability during staged construction. Pneumatic or electric piezometers (including vibrating wire) respond to changes in pore pressure more rapidly than the Casagrande type piezometers. A combination of both can be used to build redundancy into the instrumentation program.

Take this opportunity to discuss the philosophy about having redundant instruments in any instrumentation program to account for damage, malfunction of the instruments, and poor data. Remember, these instruments are often on the middle of a fill being constructed with large equipment driven by operators who could care less about the little orange flags you placed around your instrumentation.

____

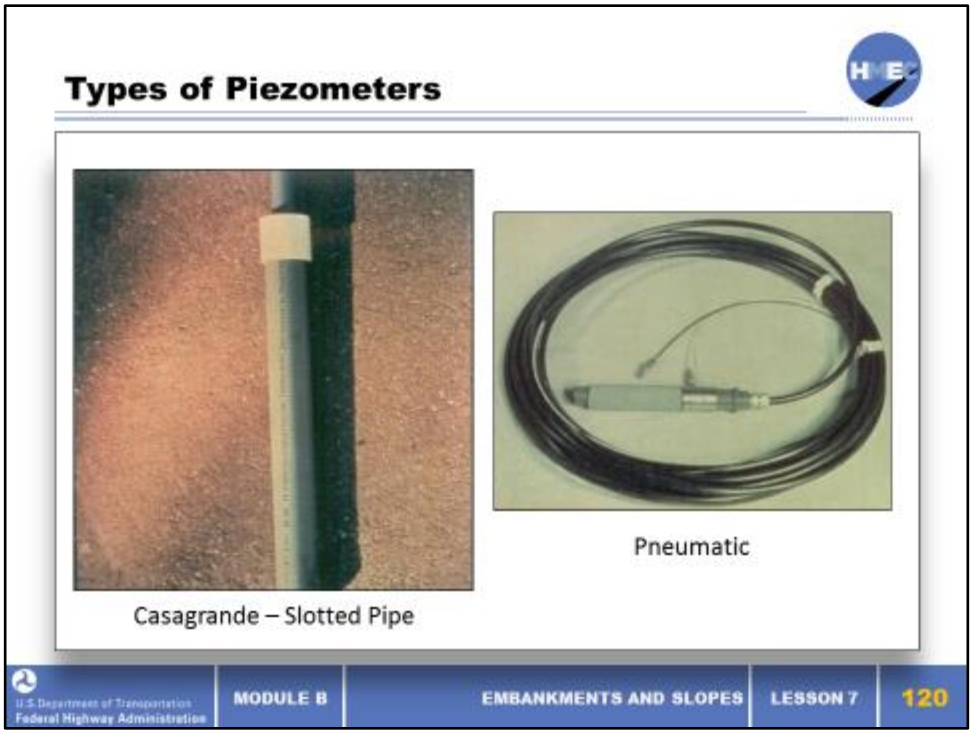
____

____

____

____

Slide 120



The slotted pipe is robust, however, the pneumatic type or the electric type piezometer allows for remote readouts to be obtained off the embankment grade, since it does not need to extend vertically up through the embankment, but can be run under the embankment to a readout box. Vibrating wire piezometers are one of the more common types. A diaphragm within the piezometer reacts to changes in water pressure which changes the tension in the vibrating wire and sends a signal to the remote readout.

____

____

____

____

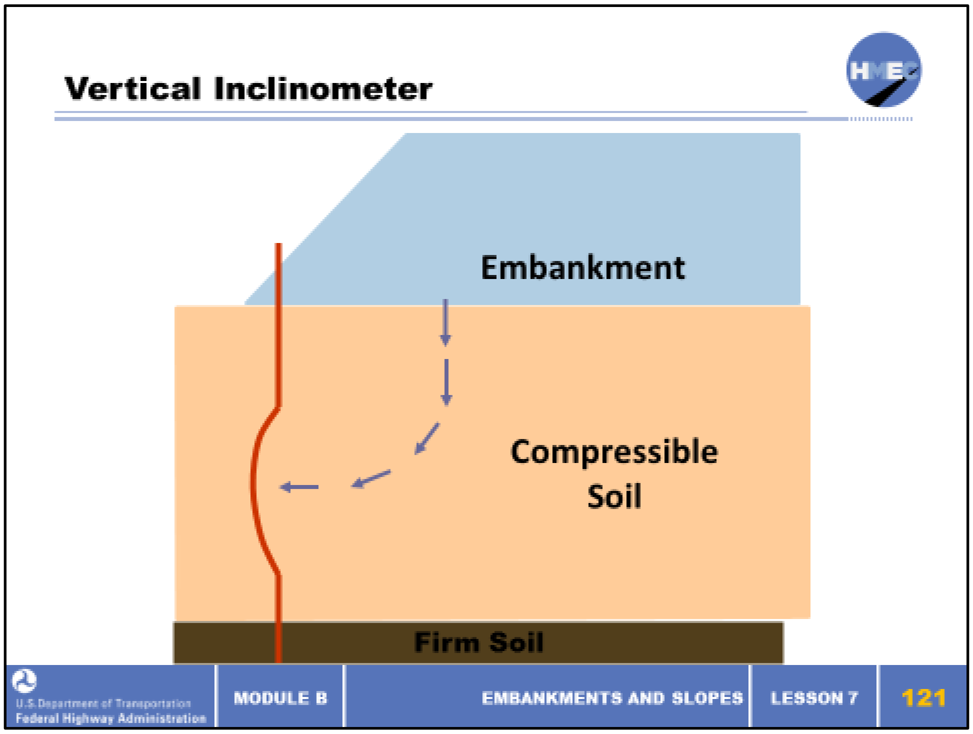
____

____

____

____

Slide 121



Slope inclinometers are used to monitor subsurface lateral deformation. A slope inclinometer typically consists of a 3-in. (75 mm) internal diameter (ID) plastic tube with four grooves cut at 90-degree intervals around the inside. The slope inclinometer tube is installed in a borehole. The bottom of the slope inclinometer tube must be founded in firm soil or rock. A readout probe that fits into the grooves is lowered down the tube and angular deflection of the tube is measured. The amount and location of horizontal movement in the foundation soil can then be measured.

If a large amount of vertical settlement is expected, telescoping casing is used.

____

____

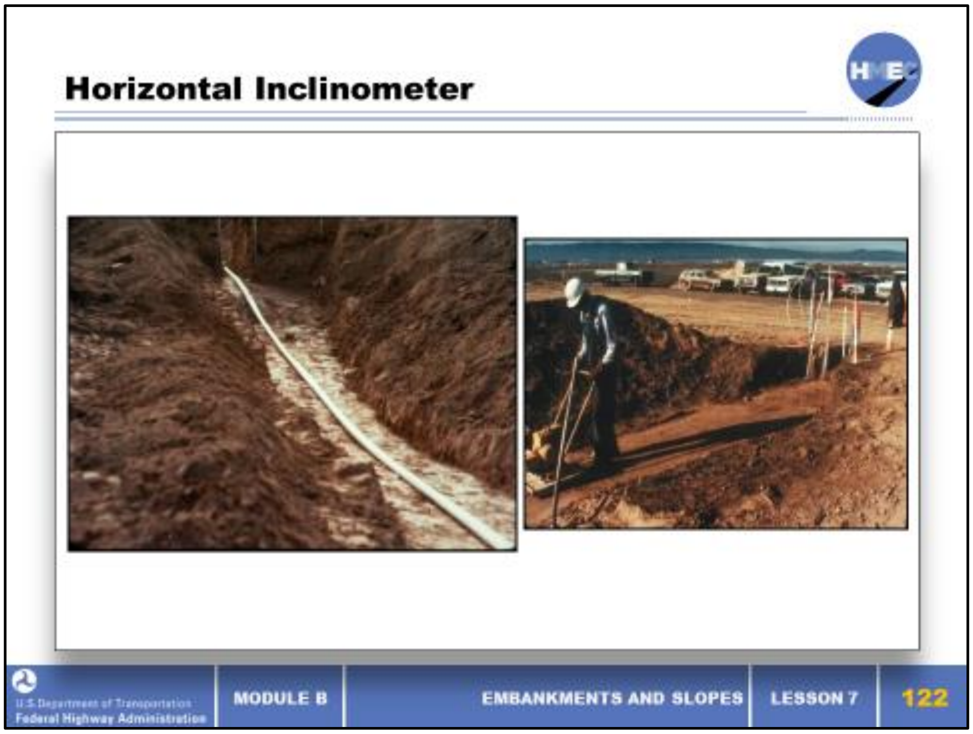
____

____

____

____

Slide 122

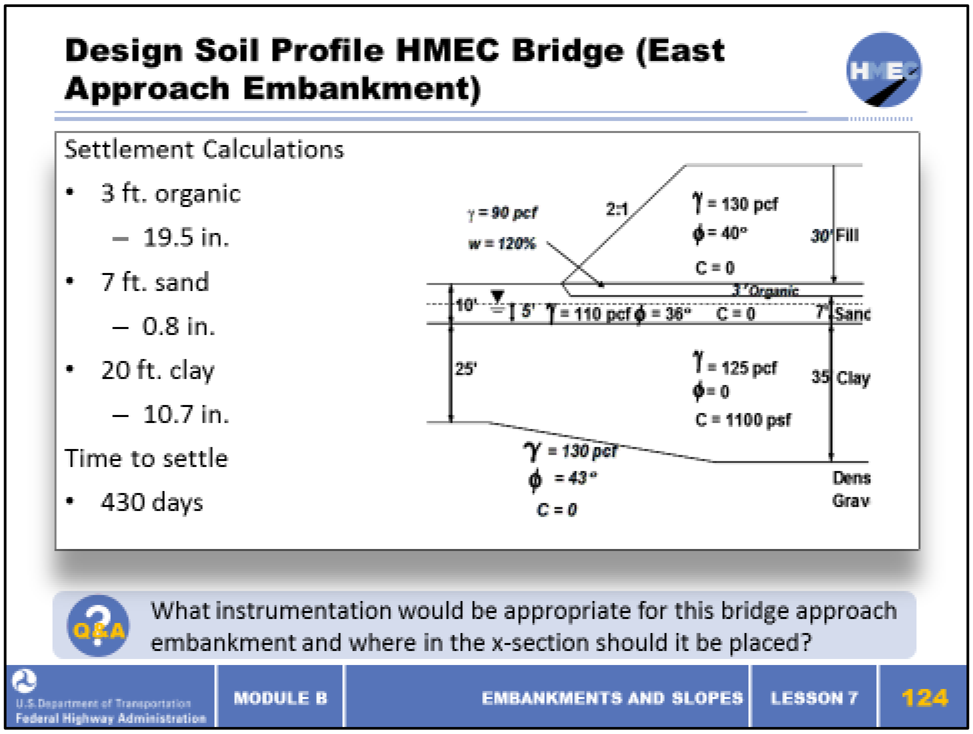


A horizontal profile gauge (horizontal inclinometer) is another tool that may be used to obtain information on the settlement profile across the width of the embankment. Note the instrument cluster on the right side of the right photo, which must be protected from damage.

Slide 123

This shows a settlement plate before construction of the embankment has begun. It is typically a ¾–2-in. steel riser pipe attached to a base plate of steel or plywood.

Slide 124


Design Soil Profile HMEC Bridge (East Approach Embankment)
HMEC
Settlement Calculations
• 3 ft. organic
– 19.5 in.
• 7 ft. sand
– 0.8 in.
• 20 ft. clay
– 10.7 in.
Time to settle
• 430 days
γ = 90 pcf
w = 120%
2:1
γ = 130 pcf
ϕ = 40°
C = 0
30' Fill
3' Organic
10'
5'
γ = 110 pcf ϕ = 36° C = 0
7' Sand
25'
γ = 125 pcf
ϕ = 0
C = 1100 psf
35 Clay
γ = 130 pcf
ϕ = 43°
C = 0
Dens
Grav
Q&A
What instrumentation would be appropriate for this bridge approach embankment and where in the x-section should it be placed?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
EMBANKMENTS AND SLOPES
LESSON 7
124

____

____

____

____

____

____

____

____

____

____

____

____

Slide 125

Learning Outcomes Review

You are now able to:

- Describe the process for determining the factor of safety
- Describe potential failure modes in soil and rock
- Using given criteria, discuss possible solutions to slope stability problems
- Using given criteria, discuss possible solutions to embankment settlement problems
- List two rock slope stabilization techniques
- Describe best practices in construction monitoring methods for stability and settlement

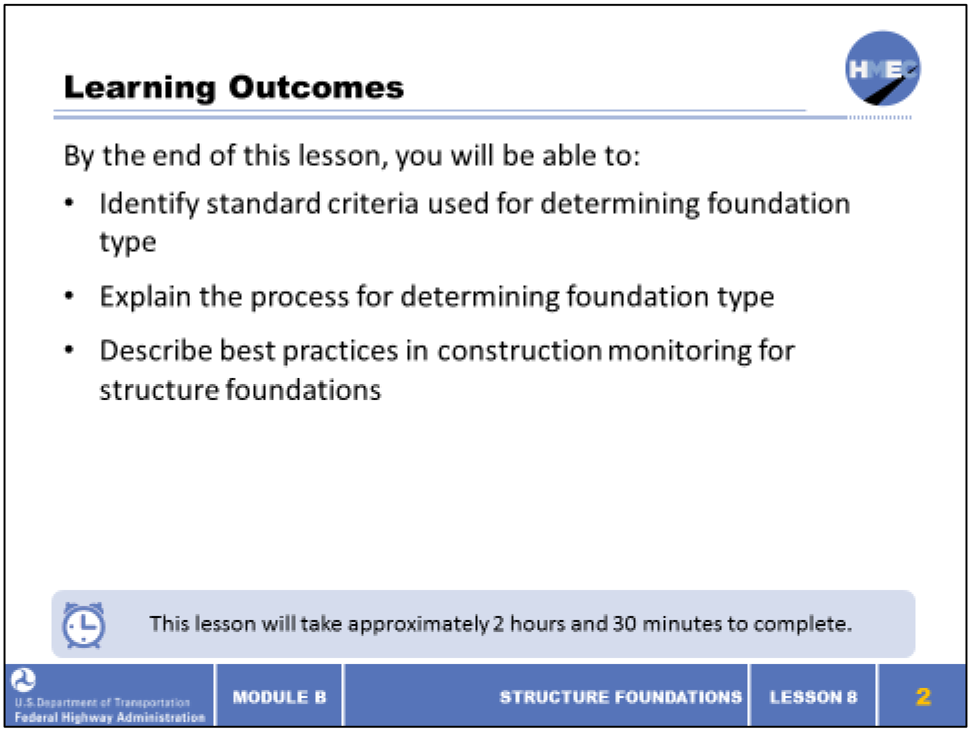
U.S. Department of Transportation Federal Highway Administration | MODULE B | EMBANKMENTS AND SLOPES | LESSON 7 | 125

This page is intentionally blank.

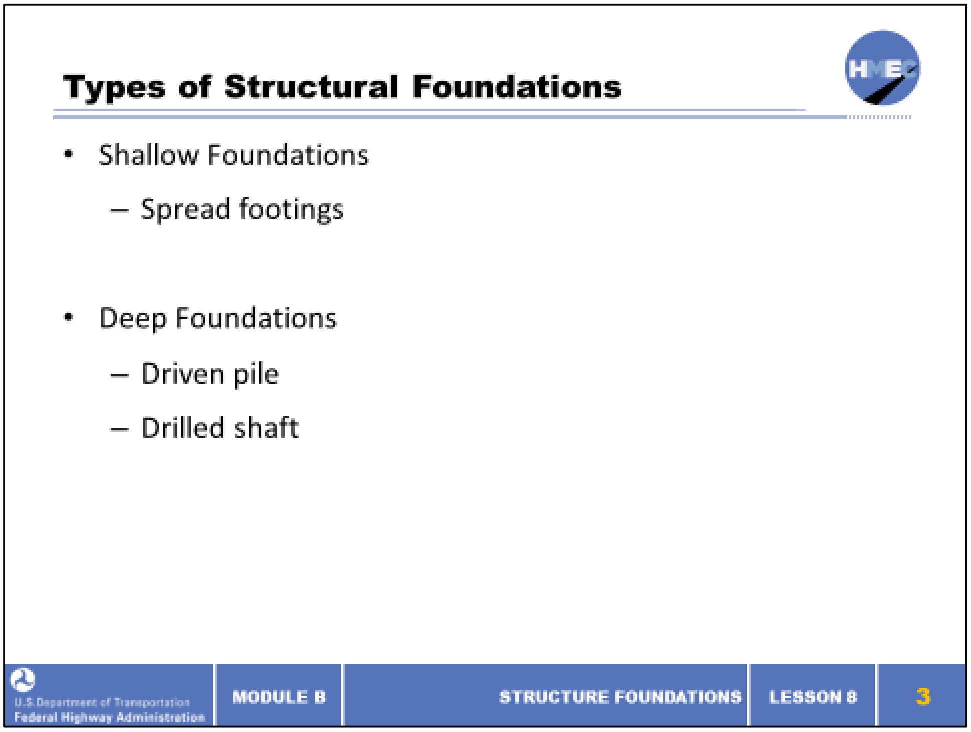
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 8: Structure Foundations
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2

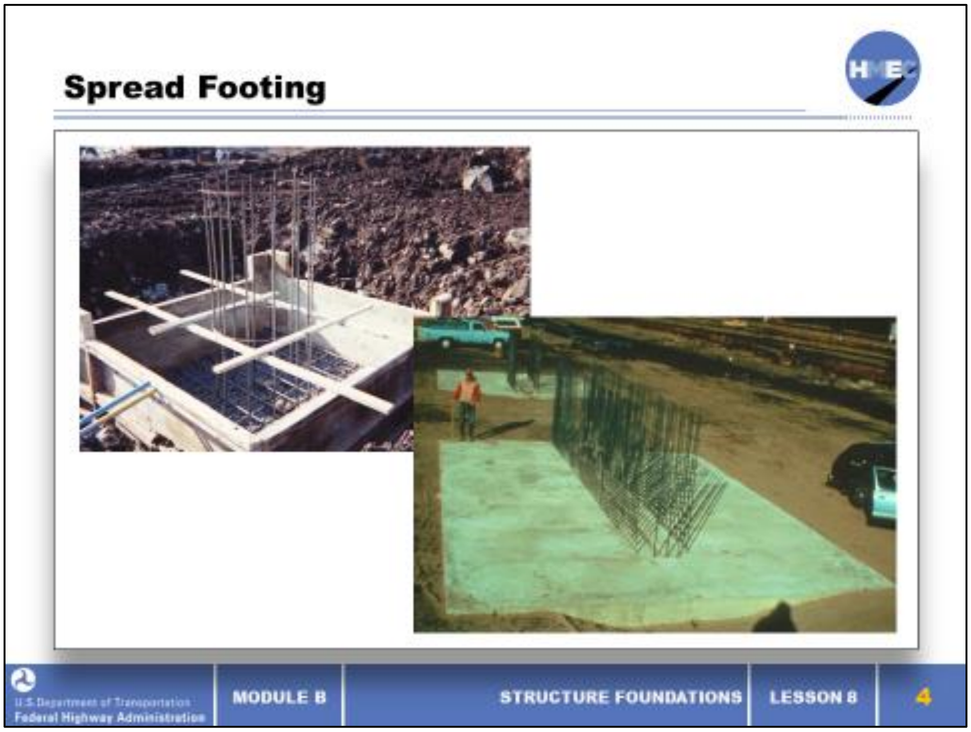

Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
• Identify standard criteria used for determining foundation type
• Explain the process for determining foundation type
• Describe best practices in construction monitoring for structure foundations
This lesson will take approximately 2 hours and 30 minutes to complete.
U.S. Department of Transportation Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
2

Slide 3



The duty of the foundation design specialist is to establish the most economical design that safely conforms to the prescribed structural criteria and properly accounts for the intended function of the structure. Essential to the foundation engineer's study is a rational method of design, whereby various foundation types are systematically evaluated and the optimum alternative is selected.

Slide 4



One is founded on weathered rock and the other on compacted engineered fill. They are reinforced and support columns or walls. Their vertical capacity comes from the shear strength of the soil and rock. Their resistance to lateral loads is from the friction between the concrete and the soil. Their size is determined by the structural loads and the strength and compressibility of the soil.

____

____

____

____

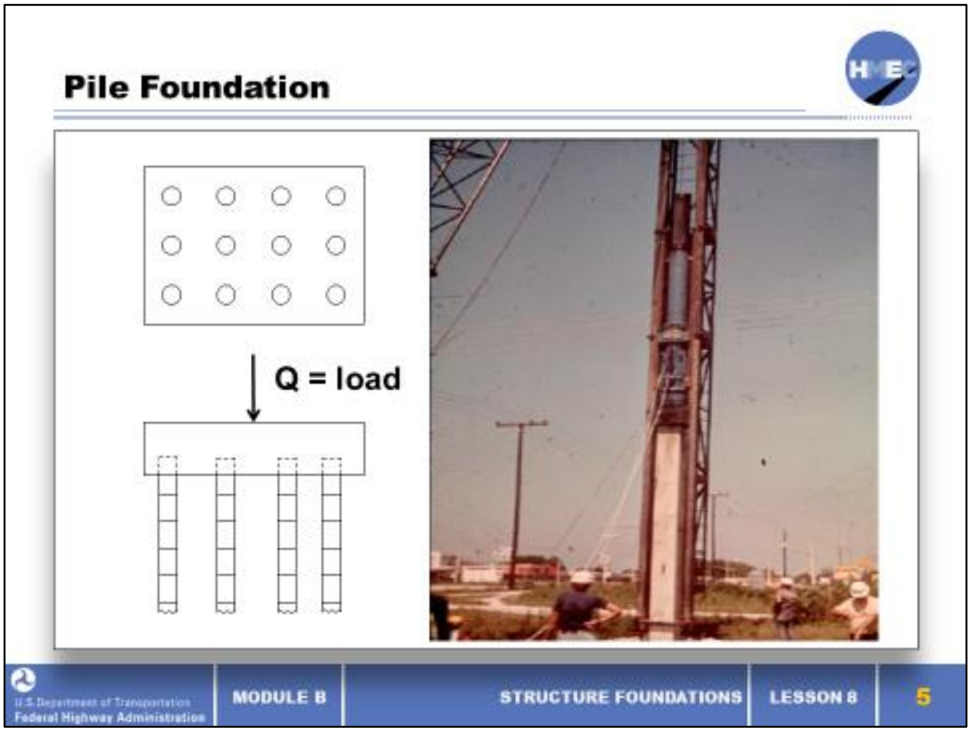
____

____

____

____

Slide 5

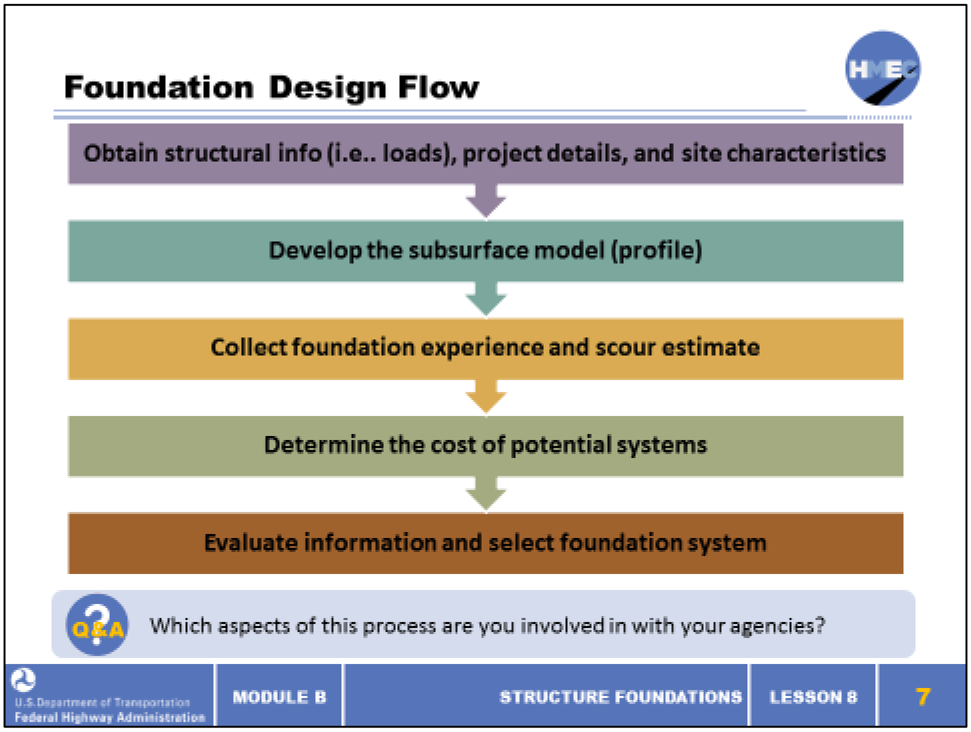


There are several types of piling available, and they are different sizes, shapes, and material. The one in this photo is a pre-cast, reinforced concrete pile. The load transfer mechanism also varies with the pile type.

Slide 6

Drilled shafts can be constructed in a variety of diameters and depths. They have large capacity and can replace several driven piles. Their applications range from small shafts for noise walls to very high capacity shafts for long-span bridges.

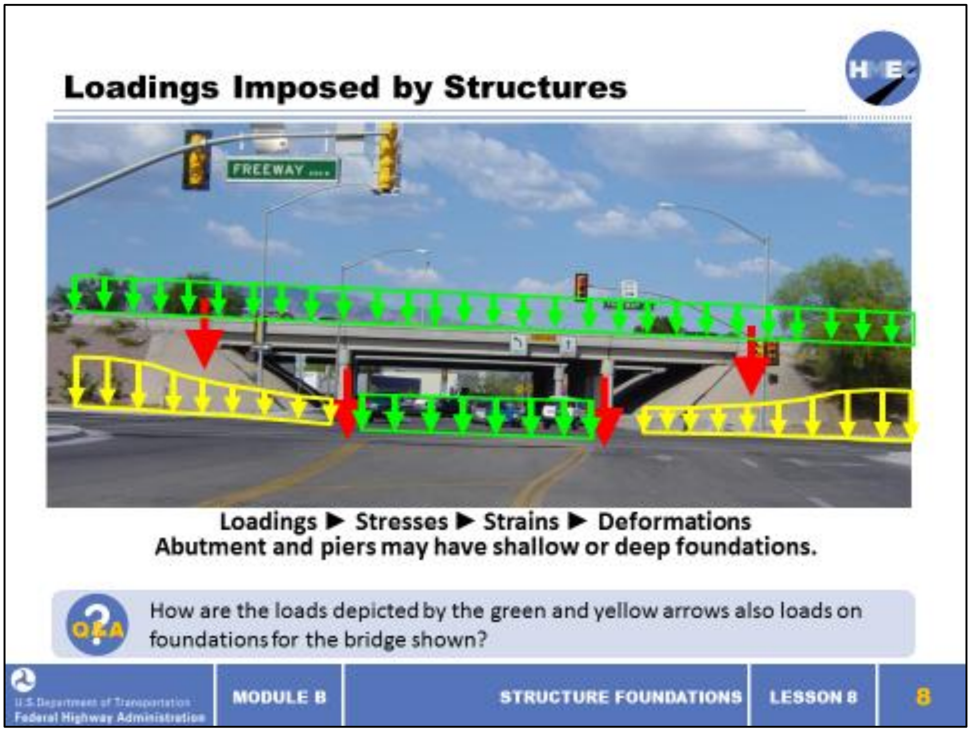
Slide 7



A systematic process should be followed in order to choose a foundation. Many of the project details we have discussed in previous lessons, plus some new ones, will be used to determine the foundation type.

____

Slide 8


Loadings Imposed by Structures
FREEWAY
Loadings ▶ Stresses ▶ Strains ▶ Deformations
Abutment and piers may have shallow or deep foundations.
Q&A
How are the loads depicted by the green and yellow arrows also loads on foundations for the bridge shown?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
8

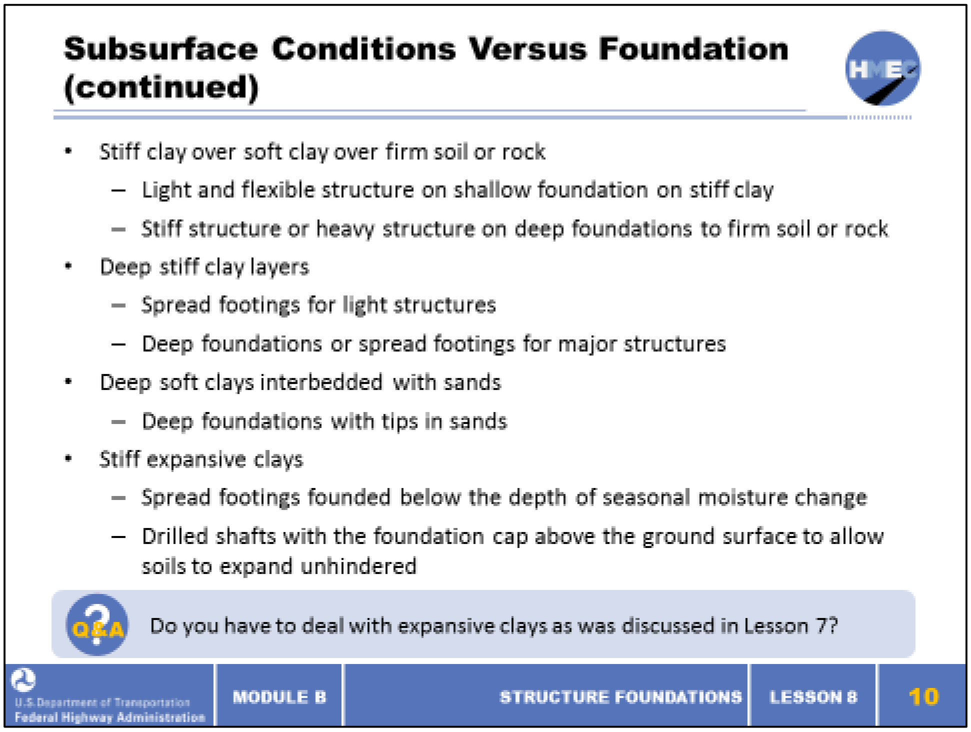
Slide 9

Subsurface Conditions Versus Foundation

- Thin layer of very soft clay over firm soil or rock
  - Set foundation on competent soil or rock
- Thick soft layer over firm soil or rock
  - Use deep foundations – piles
  - Apply soil improvement techniques (Lesson 11) and use spread footing
- Stiff clay over rock
  - Shallow foundation for light loads
  - Shallow foundations or deep foundations for heavier loads
- Deep sand stratum
  - Light loads on spread footings
  - Heavy loads on footings or piles

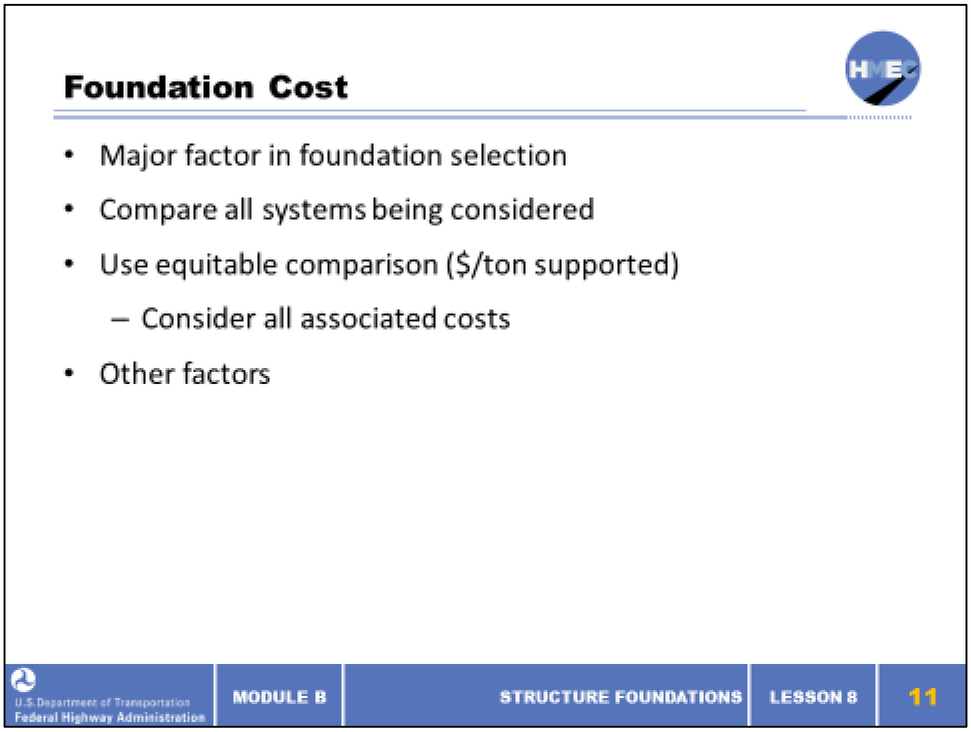
U.S. Department of Transportation Federal Highway Administration | MODULE B | STRUCTURE FOUNDATIONS | LESSON 8 | 9

Slide 10



Deep foundations affected by expansive clays will need to extend well below the expansive layer and be designed to resist uplift from the expansive layer. For drilled shafts, care must be exercised to ensure that the upper portion of the shaft does not increase in diameter. A common construction technique is to use a surface casing, which is a larger diameter than the design shaft. That should not be allowed unless considered in design.

Slide 11



Our evaluation of the project information should also include a comparison of costs for each foundation system being considered. Whether it is for shallow or deep foundations, it is recommended that foundation support cost be defined as the total cost of the foundation system divided by the load the foundation supports in tons. Thus, the cost of the foundation system should be expressed in terms of dollars per ton of load that will be supported. For an equitable comparison, the total foundation cost should include all costs associated with a given foundation system, including the need for excavation or retention systems, environmental restrictions on construction activities, e.g., vibrations, noise, disposal of contaminated excavated spoils, pile caps and cap size, etc. For major projects, if the estimated costs of alternative foundation systems during the design stage are within 15% of each other, then alternate foundation designs should be considered for inclusion in contract documents.

Cost analyses of all feasible alternatives may lead to the elimination of some foundations that were otherwise qualified under the engineering study. Other factors that must be considered in the final foundation selection are the availability of materials and equipment, the qualifications and experience of local contractors and construction companies, as well as environmental limitations/considerations on construction access or activities.

____

____

____

____

____

____

____

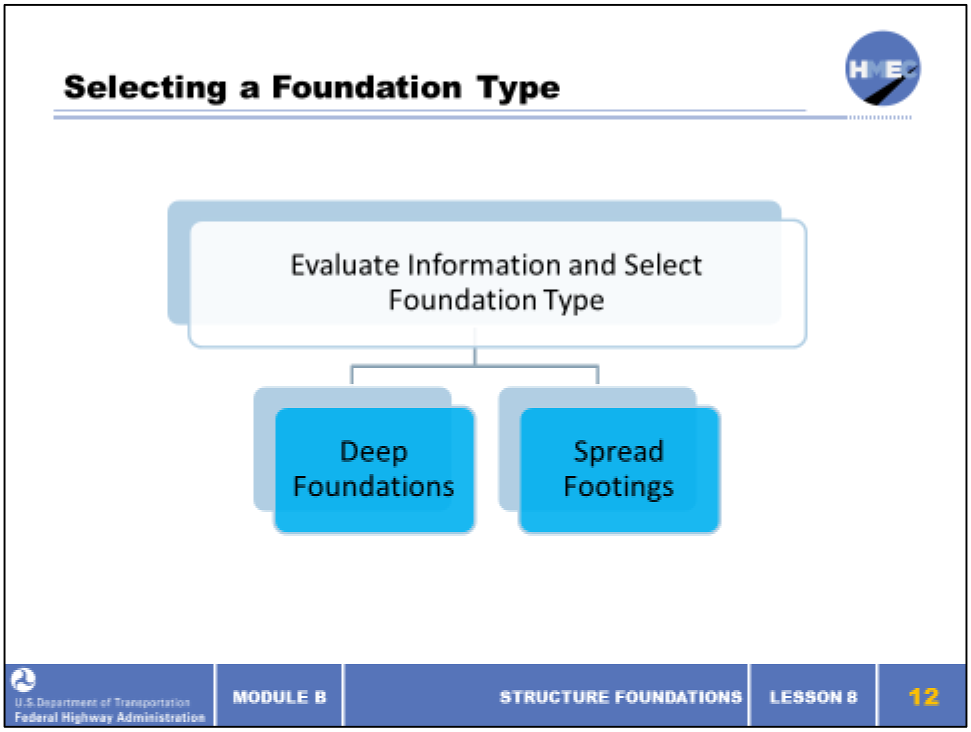
____

____

____

____

Slide 12



We will limit our discussion to generic foundation types; however, proprietary foundation systems should not be excluded as they may be the most economical alternative in a given set of conditions.

____

____

____

____

____

____

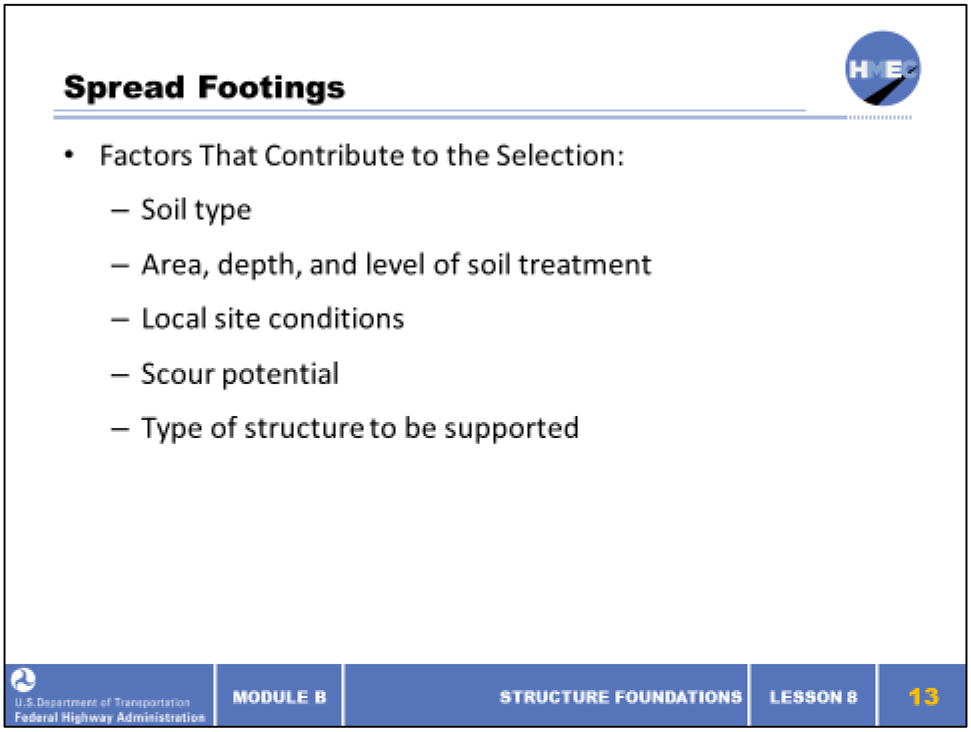
____

____

____

____

Slide 13



Since spread footings are supported by near surface soils or rock, that material must be capable of providing adequate bearing and limiting settlement. If weak soils can be excavated and replaced with competent engineered fill, spread footings may be appropriate in those situations as well. Scour potential of any significant depth will often require selection of a deep foundation system instead of a spread footing. Very high loads and/or moments may also dictate a deep foundation unless the spread footing can be founded on rock.

Slide 14

Spread footings founded on rock are an excellent choice for a bridge foundation, therefore if near-surface competent rock exists at the site, a spread footing foundation may the appropriate system.

Construction of spread footings on an embankment constructed with high quality granular material and compaction control will have no issues with bearing capacity or settlement within the embankment. An appropriate amount of time will need to be allowed for settlement if the embankment is built over a compressible subsurface layer. Studies have shown that bridges supported by spread footings on engineered fills or improved natural soils have few problems. However, a 2007 survey indicated that only a few DOTs (< 15) use spread footings founded on soil for more than 10% of their bridges.

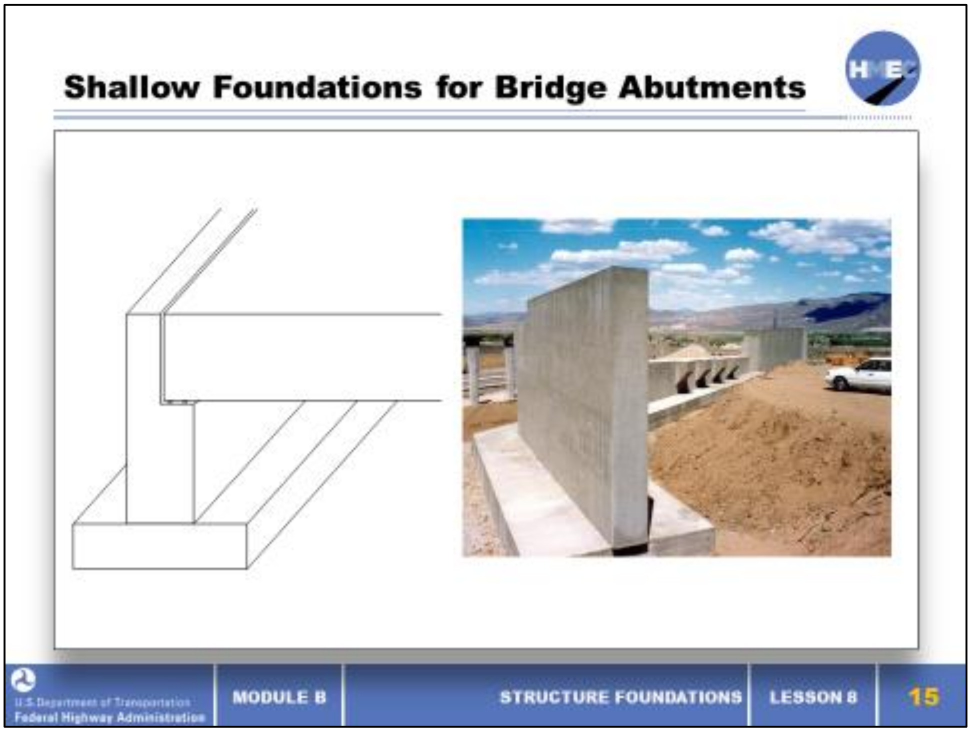
____

____

____

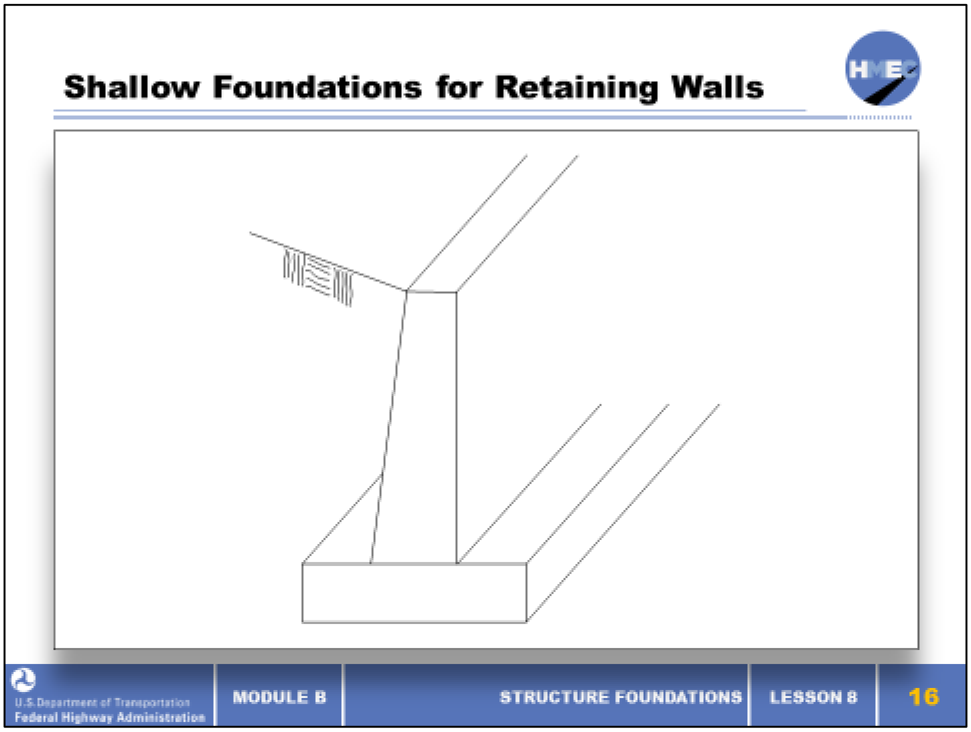
____

Slide 15



A national FHWA survey of the geotechnical practices of the state DOTs was developed and distributed in 2007. The States' geotechnical engineers in 44 States responded to this survey. Survey results indicate that the average distribution of bridge foundation types considered by State DOTs across the United States is approximately 24% spread footings (11.5% founded on soils, 12.5% founded on rock) and 76% deep foundations (56.5% driven piles and 19.5% drilled shafts). Similar information was recently reported by Paikowsky et al. (*7*). The FHWA national survey identified the States with significant and moderate use (> 10%) of spread footings on soils to support highway bridges (Table 1) and the States with limited or no use (< 5%). Based on this survey, the FHWA concluded that a number of State DOTs could save time and cost if spread footings are incorporated in their selection process and used when appropriate to support bridges.

Slide 16



Several types of retaining walls require structurally reinforced footings to support the wall portion of the structure. Cast-in-place (CIP) cantilever and counterfort walls are two examples—a cantilever wall is shown in this illustration. If the underlying soil is competent, a spread footing may be used.

Slide 17

Scour undermined the spread footings on the Schoharie Creek in New York, causing it to collapse during a record flood in 1987. However, the investigation revealed that the scour had begun soon after construction and had continued slowly for over 30 years. The spread footings were founded on a dense gravel interbedded with layers of hard glacial till. The foundation material scoured a little during each flood over the 30-year period until the footing was finally undermined during the 1987 flood. The bridge was a simple span design so when the pier collapsed, the two spans of the superstructure also collapsed. Ten people lost their lives due to the collapse.

This 1987 collapse was a call to action on addressing scour potential on our highway structures.

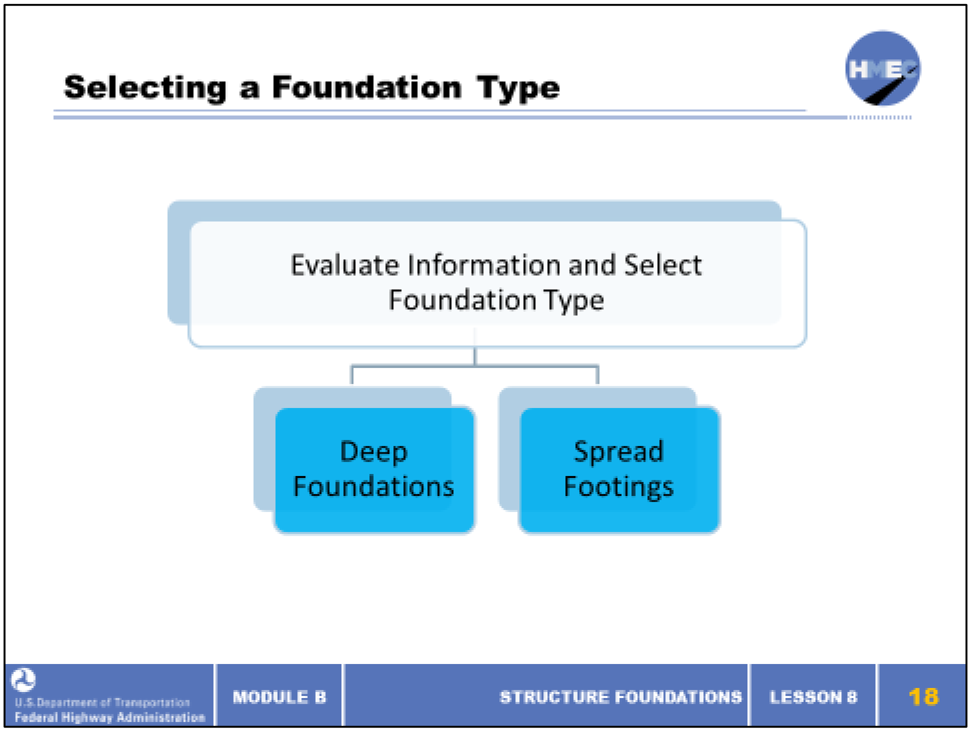
____

____

____

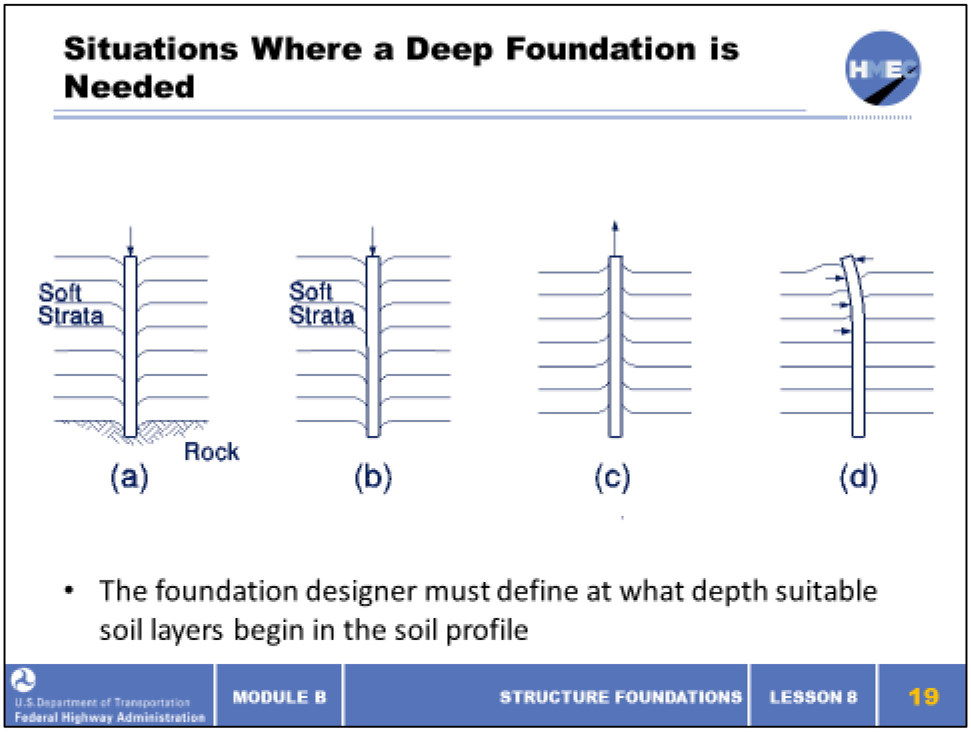
____

Slide 18



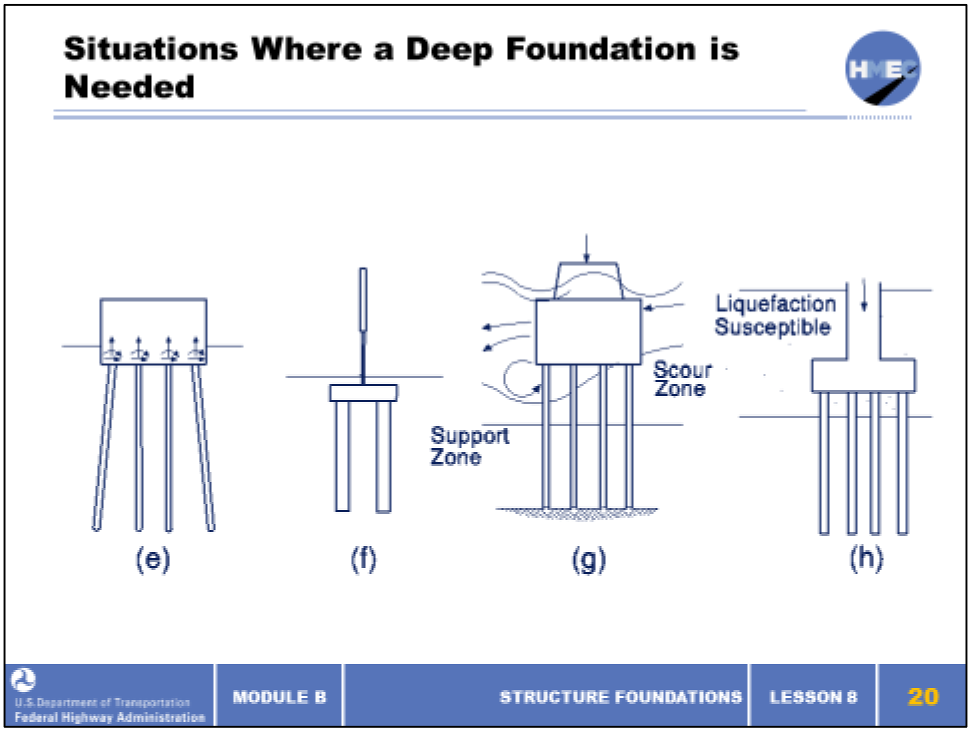
This is a transition slide to deep foundations.

Slide 19



That soil strength often increases with depth and the designer must use soil properties obtained through subsurface investigation and testing to determine at what depth suitable soil layers exist within the soil profile.

Slide 20

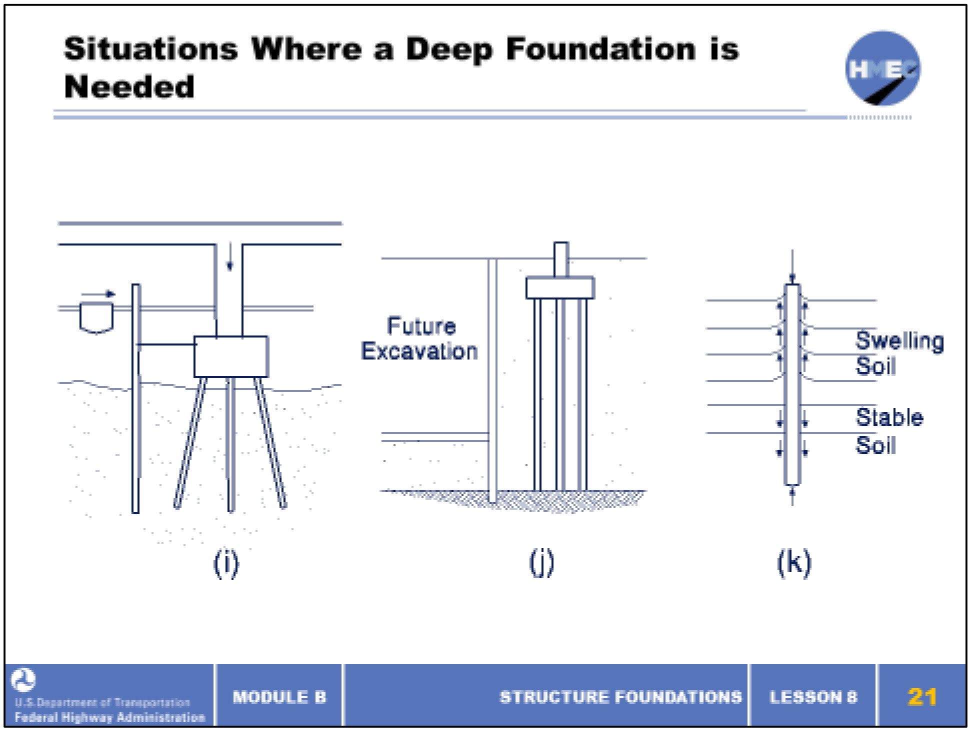


Lateral loads are resisted by groups of vertical and battered foundations, which combine the axial and lateral resistances of all deep foundations in the group, Figure 9-l (e). Lateral loads from overhead highway signs and noise walls may also be resisted by groups of deep foundations.

Deep foundations are often required when scour around footings could cause loss of bearing capacity at shallow depths. In this case the deep foundations must extend below the depth of scour and develop the full capacity in the support zone below the level of expected scour. FHWA (2001c) scour guidelines require the geotechnical analysis of bridge foundations to be performed on the basis that all stream bed materials in the scour prism have been removed and are not available for bearing or lateral support. Costly damage and the need for future underpinning can be avoided by properly designing for scour conditions.

Soils subject to liquefaction in a seismic event may also dictate that a deep foundation be used. Seismic events can induce significant lateral loads to deep foundations. During a seismic event, liquefaction-susceptible soils offer less lateral resistance as well as reduced shaft resistance to a deep foundation. Liquefaction effects on deep foundation performance must be considered for deep foundations in seismic areas.

Slide 21

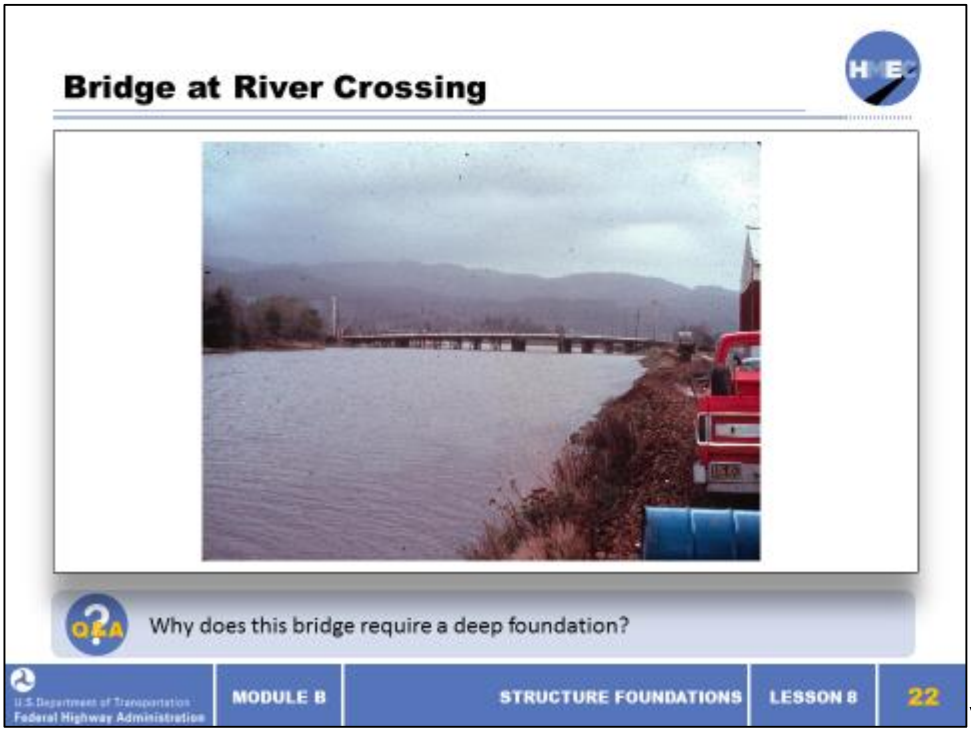


Deep foundations are often used as fender systems to protect bridge piers from vessel impact. Fender system sizes and group configurations vary depending upon the magnitude of vessel impact forces to be resisted. In some cases, vessel impact loads must be resisted by the bridge pier foundation elements. Single deep foundations may also be used to support navigation aids.

In urban areas, deep foundations may occasionally be needed to support structures adjacent to locations where future excavations are planned or could occur, Figure 9-1(j). Use of shallow foundations in these situations could require future underpinning in conjunction with adjacent construction.

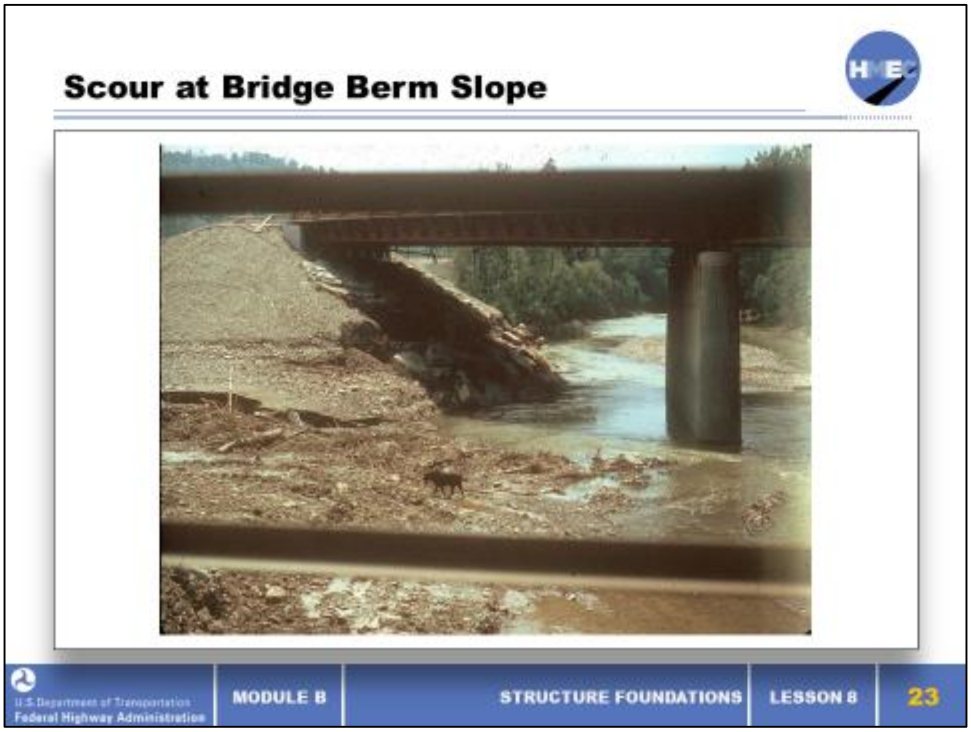
Deep foundations are used in areas of expansive or collapsible soils to resist undesirable seasonal movements of the foundations. Deep foundations under such conditions are designed to transfer foundation loads, including uplift or down drag, to a level unaffected by seasonal moisture movements.

Slide 22



This picture is of a bridge located in a flood plain. Emphasize that the hydraulics engineer should be involved in foundation designs in the vicinity or water crossings.

Slide 23



This abutment was affected by scour forces that eroded the end fill to a point where the footing was undermined. The point is that embankment material is not sufficient protection against scour forces.

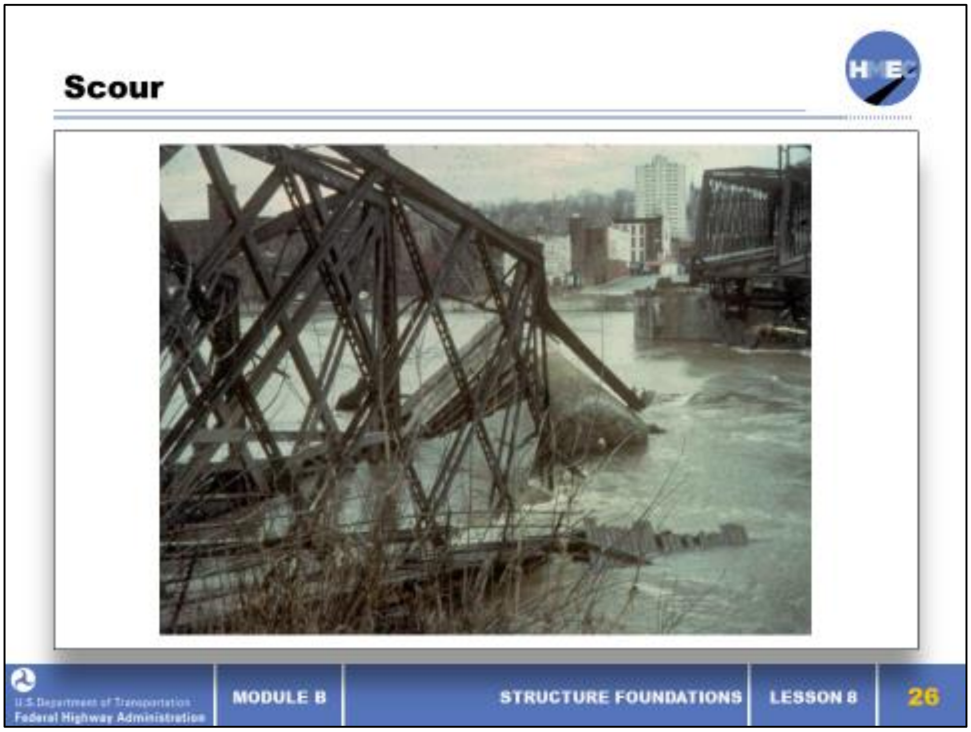
Slide 24

The foundation design must account for future removal of soil by water and foundations elements extended sufficiently below the scour depth to mobilize the required resistance for the foundation loads. For abutments, the added issue of the erosion of roadway fill behind the abutment and therefore collapse of the roadway pavement must be considered. Studies have shown that even large riprap must be designed with care if it is to resist the forces of scour. Point out that several FHWA publications and NHI courses are available regarding scour and scour countermeasures (HEC-18, HEC-20 and HEC-23).

Slide 25

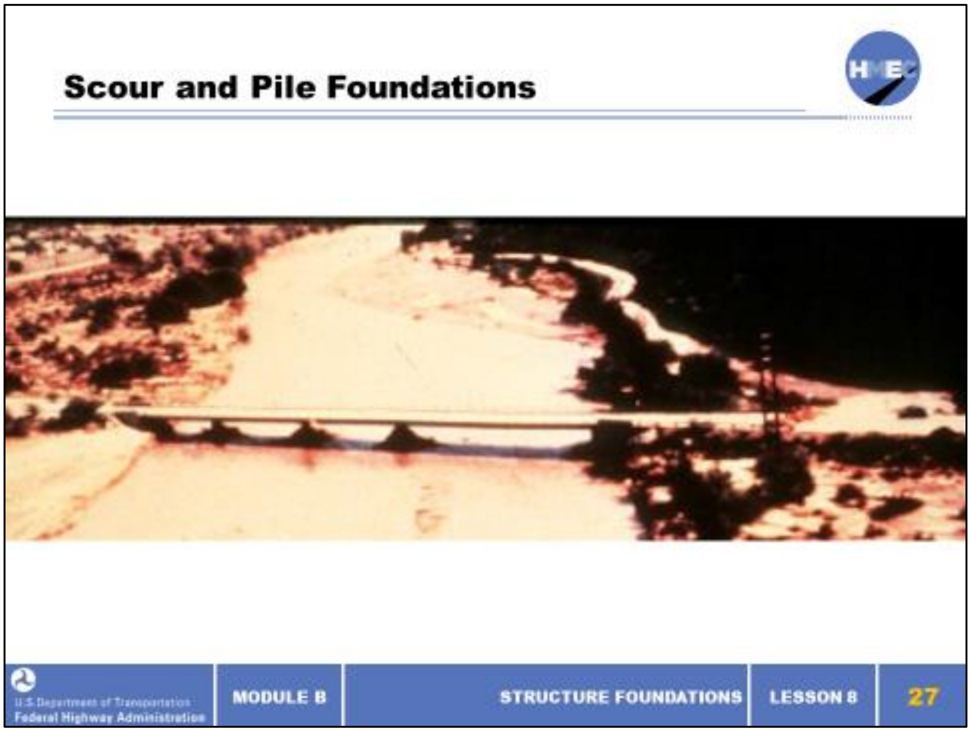
For bridges crossing rivers in northern climates, ice loads must be considered. Often deep foundations are required to resist the substantial loads from ice and ice flows. The loads can be both lateral and uplift.

Slide 26



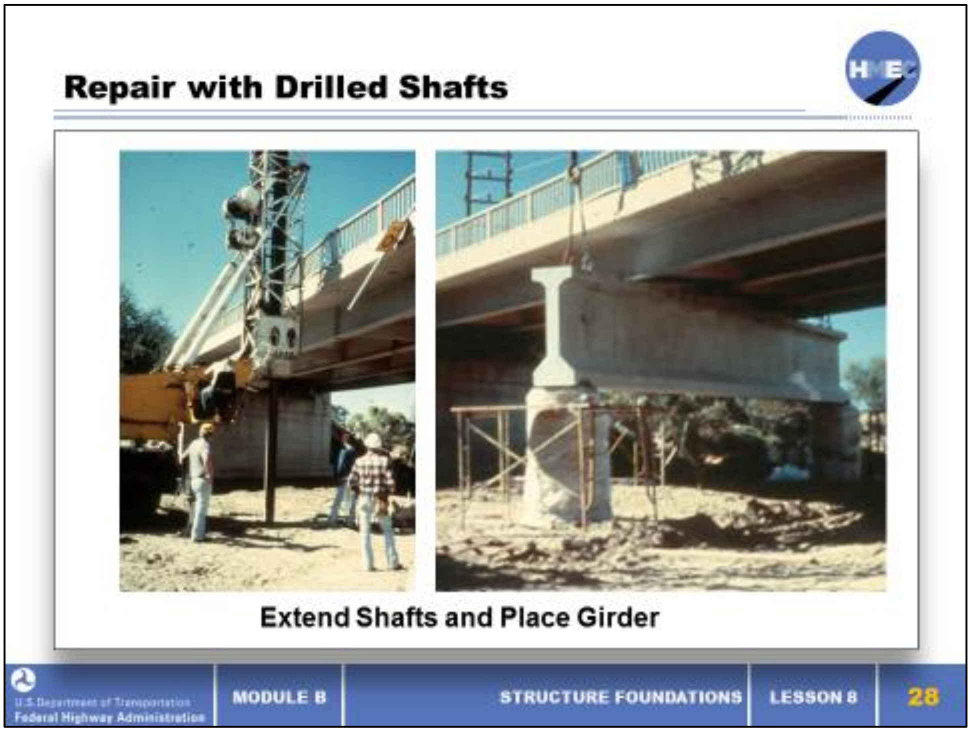
This case history demonstrates how pier footings, even those on piles, can fail if scour is not accounted for. This bridge was founded on short timber piles. A 50-year storm caused scour around the pier to about half the length of the piles. The remaining pile embedment was inadequate to resist the applied structure and water forces. The downstream piles plunged and the pier rotated, broke the upstream piles and fell into the scour hole. The point to make is that rational design needs to account for all factors that influence the foundation.

Slide 27



During a heavy flood, the fast-moving floodwaters scoured away the cohesionless soil from around the pile supported bridge bents, removing lateral support, while at the same time imposing large lateral loads on the bents. One bent collapsed and was washed downstream. Fortunately, the bridge was of continuous span design, so the superstructure remained intact, although it was incapable of remaining in service.

Slide 28



Drilled shafts were installed laterally along the alignment of the bridge at the station of each pile-supported bent by drilling them well into the conglomerate. The shafts were then extended upward to an appropriate elevation, and a heavy transfer beam was put into place on top of the shafts (on the right). This activity was duplicated in sequence for each bent on the bridge, not just the one that collapsed.

The superstructure was then jacked back to its correct position off of the newly placed transfer beams, one bent at a time. The rockers and bearing plates were then installed, and the bridge was ready to return to service.

This is a good example of a situation in which drilled shafts are preferable to driven piles because of their ready ability to penetrate rock and similar non-scourable geomaterial.

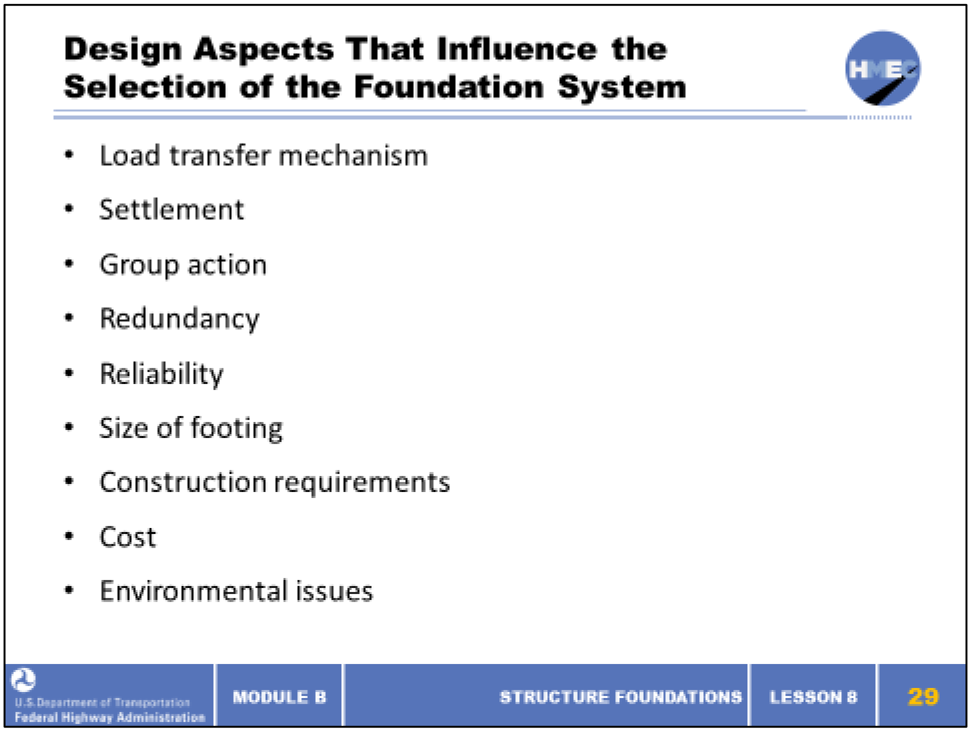
____

____

____

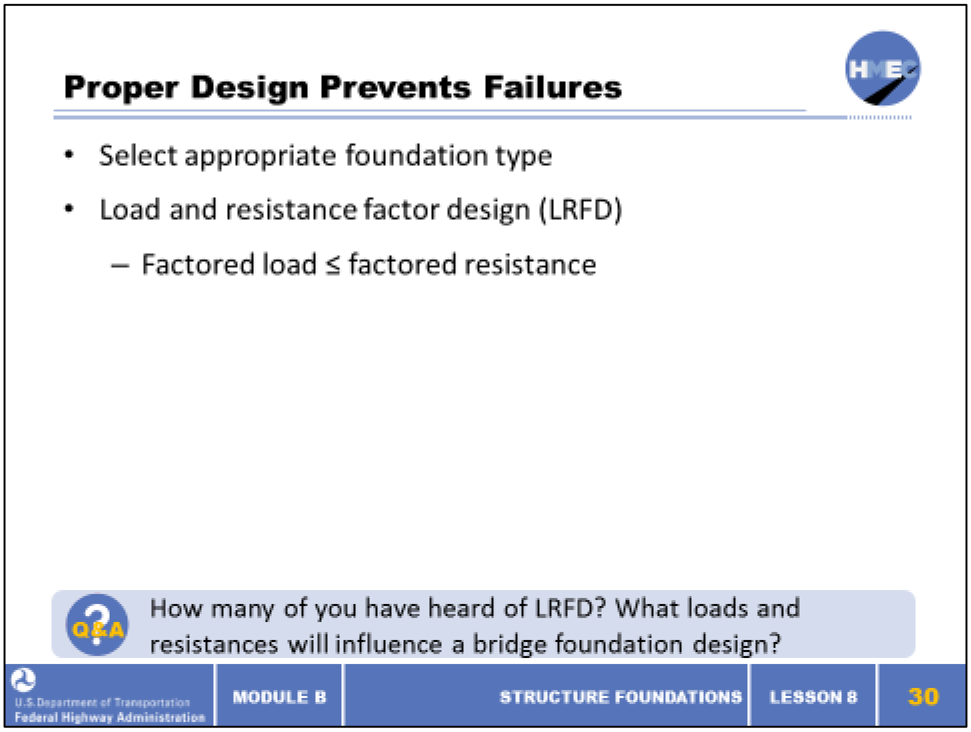
____

Slide 29



The project site may require a small footing size and therefore eliminate some foundation options. Construction and environmental issues may limit certain construction techniques or require disposal of excavated material, which may increase costs. Some systems are more redundant and may be considered more reliable than others.

Slide 30


Proper Design Prevents Failures
HMEC
• Select appropriate foundation type
• Load and resistance factor design (LRFD)
– Factored load ≤ factored resistance
Q&A
How many of you have heard of LRFD? What loads and resistances will influence a bridge foundation design?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
30

Slide 31

Loads Versus Resistance

- Loads
  - Dead loads
  - Live loads
  - Earth pressure
  - Surcharge loads
  - Down drag
  - Earthquake
  - Vehicular or vessel impact
  - Water load
  - Ice load
  - Wind load
- Resistance
  - Material strength
    - Axial
    - Shear
    - Flexural
  - Soil and Rock Strength
    - Shear
      - bearing, skin friction, end bearing, uplift
  - Soil Weight
  - Others
    - Number of cycles, temperature, etc.

U.S. Department of Transportation Federal Highway Administration | MODULE B | STRUCTURE FOUNDATIONS | LESSON 8 | 31

Slide 32

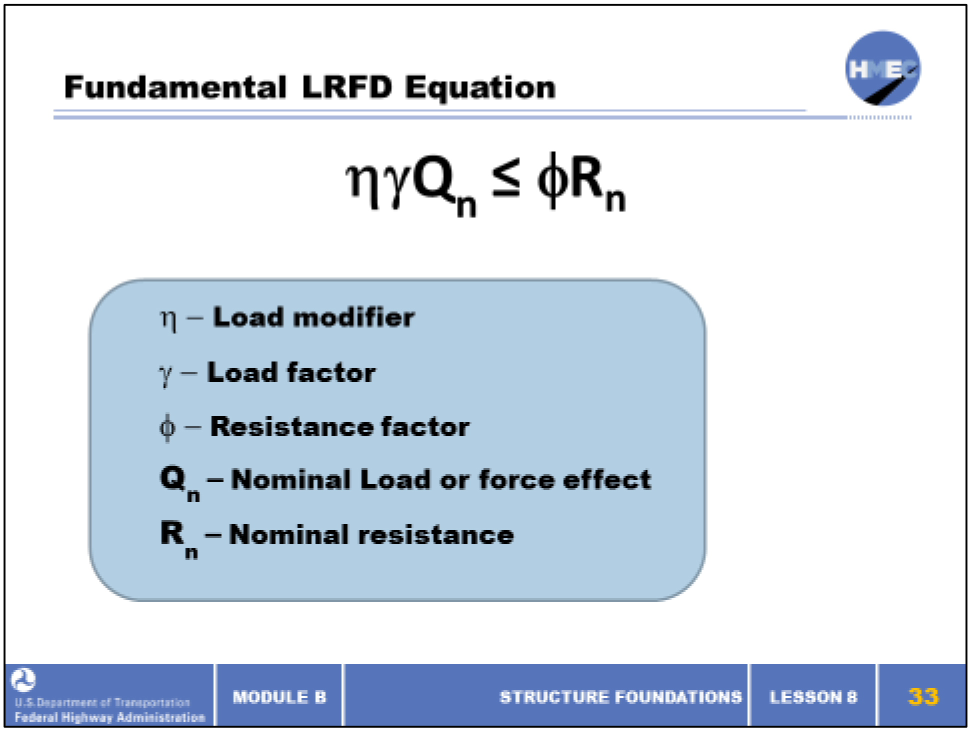
Uncertainty

- Material dimensions and location
- Material strength
- Failure mode and prediction method
- Long- term material performance
- Material weights
- Prediction of potential transient loads
- Load analysis and distribution methods
- General uncertainty associated with structure function

U.S. Department of Transportation Federal Highway Administration | MODULE B | STRUCTURE FOUNDATIONS | LESSON 8 | 32

Certain methods of capacity prediction may be more reliable than others and therefore have less uncertainty.

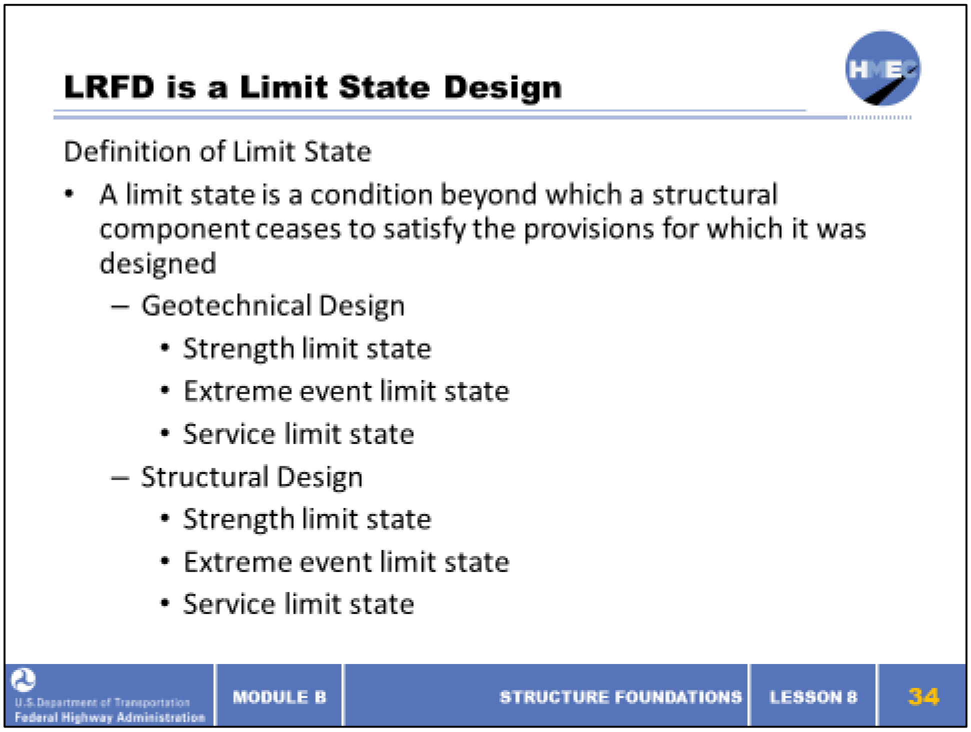
Slide 33



The load modifier is used to adjust the load to account for three issues: redundancy (number of load carrying elements), ductility of the material, and importance of the structure. Load factors are typically 1.0 or greater and are used to increase the load that must be resisted. The larger the uncertainty, the higher the load factor will be. For instance, since the weight of a truck crossing a bridge has a higher level of uncertainty than the weight of concrete in the bridge, the load factor for LL is greater than the load factor for DL. The resistance factor is 1.0 or less. As described earlier, it is used to reduce the strength or capacity of the material to an allowable resistance based on uncertainty.

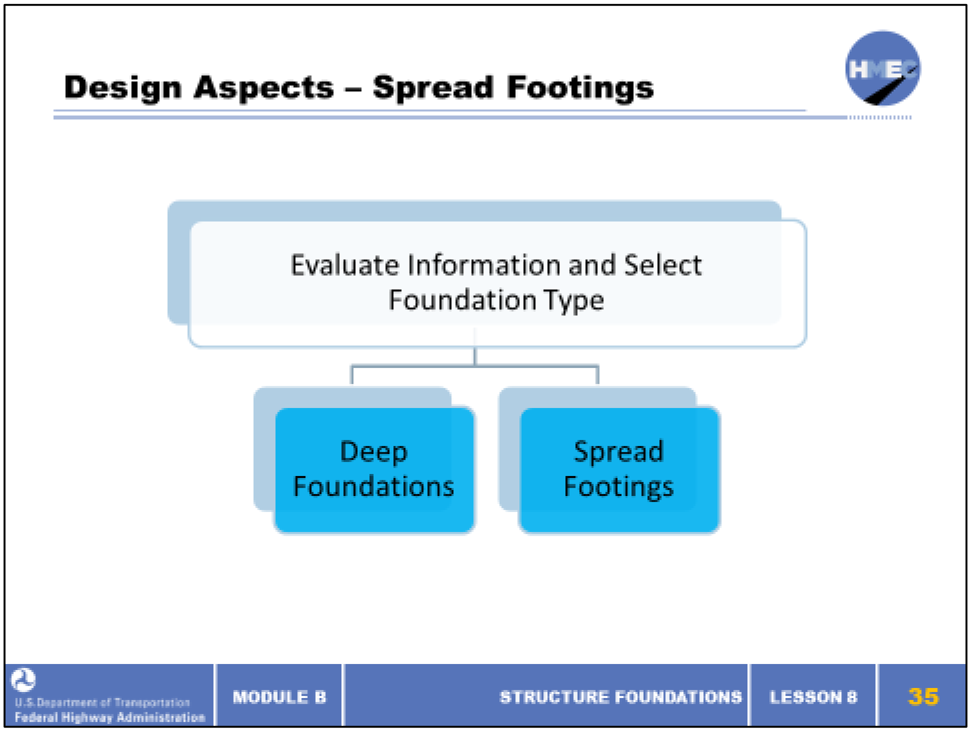
Use of common terminology is critical between project team members and disciplines. With LRFD, we want to be specific about loads and resistance. They are either "nominal" (i.e., unfactored) loads or "factored" loads. Likewise, resistance is either nominal or factored.

Slide 34

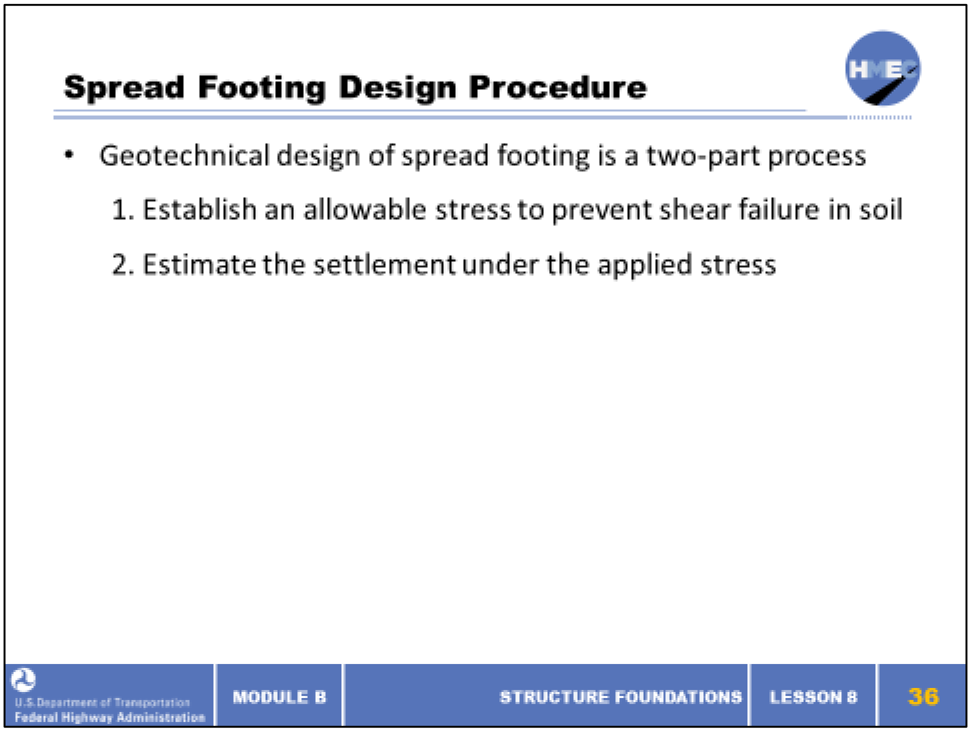


Extreme event limit state is a strength limit that is applied when extreme events, such as floods or earthquakes occur. The load factors, resistance factors, and load combinations are adjusted when evaluating extreme events to reflect the reduced likelihood of various loads occurring during an extreme event. Service limit states are typically evaluated with load and resistance factors of 1.0 to ensure that the structure does not exceed horizontal, vertical, or lateral strain or movement under service loads.

Slide 35

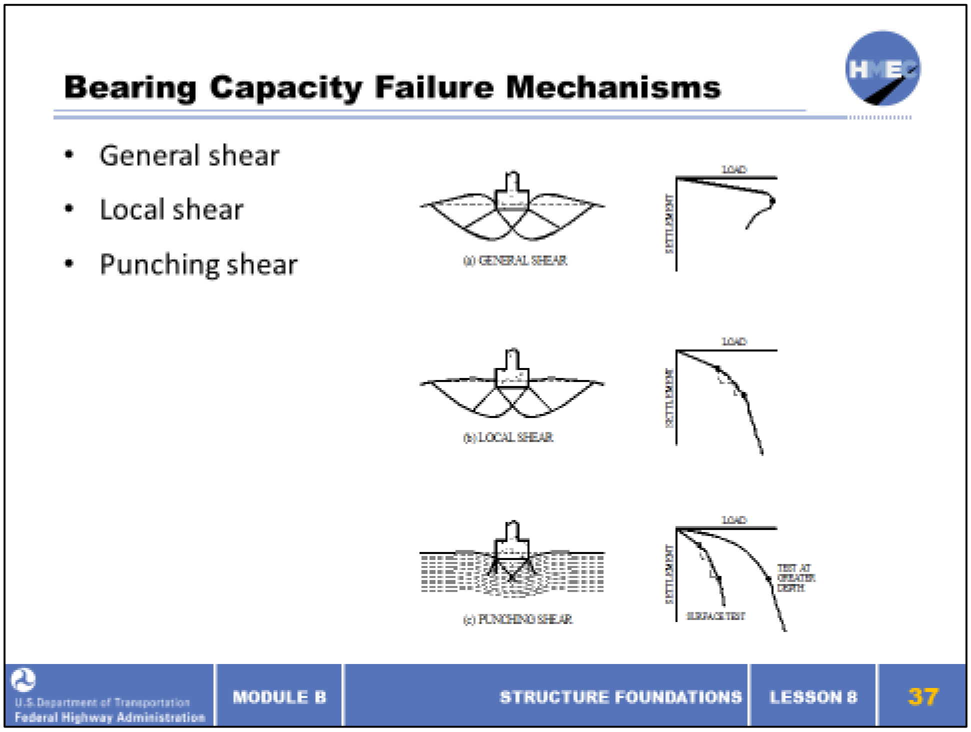

Design Aspects – Spread Footings
HMEC
Evaluate Information and Select Foundation Type
Deep Foundations
Spread Footings
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
35

Slide 36



Since settlement almost always controls the design of a spread footing on soil for transportation structures, conventionally the footing is sized to meet settlement criteria and then bearing pressure is checked. Spread footings founded on competent intermediate geomaterials (IGMs) or rock are not typically controlled by settlement or bearing capacity, but are sized based on structure loads.

Slide 37



Due to the large size of the footings for transportation structures, general shear is the most common type of shear failure that a designer is concerned with. This is because of the limitation on settlement due to structural considerations. The other two failure mechanisms may occur if the footing is founded on weak cohesive soils, which should be avoided

____

____

____

____

____

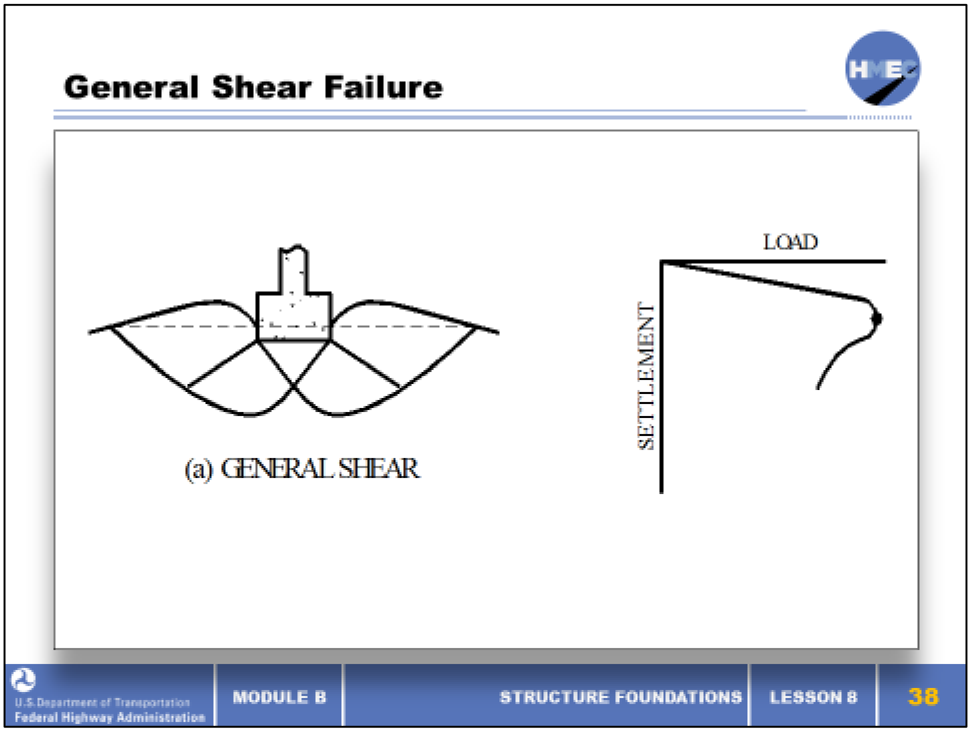
____

____

____

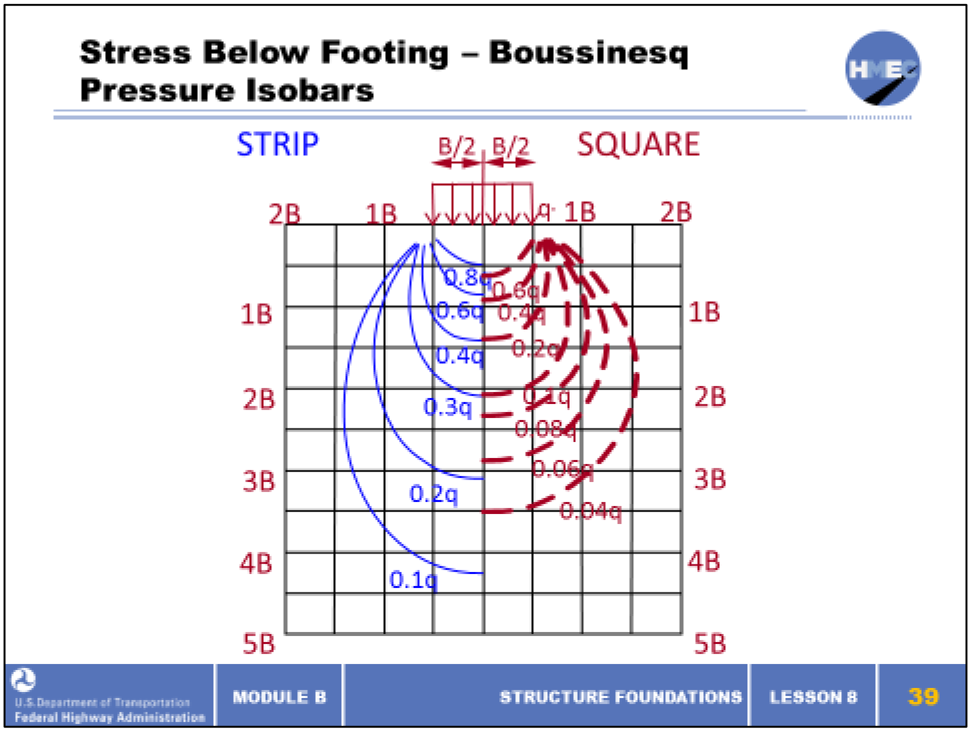
____

Slide 38



To mobilize the full bearing capacity, substantial movement of the footing must occur. Point out the load/settlement curve on the right. Typically, the movement required to cause a shear failure is much more than would be allowed for the structure that the spread footing is supporting. To limit the settlement, the footing size must be increased and when the size of the footing is increased, the unit stress on the footing from the structure must be decreased (we'll discuss that in a minute). This is why the footing is typically sized to meet settlement criteria, which results in a lower pressure under the footing and ensures that bearing capacity will not control the design.

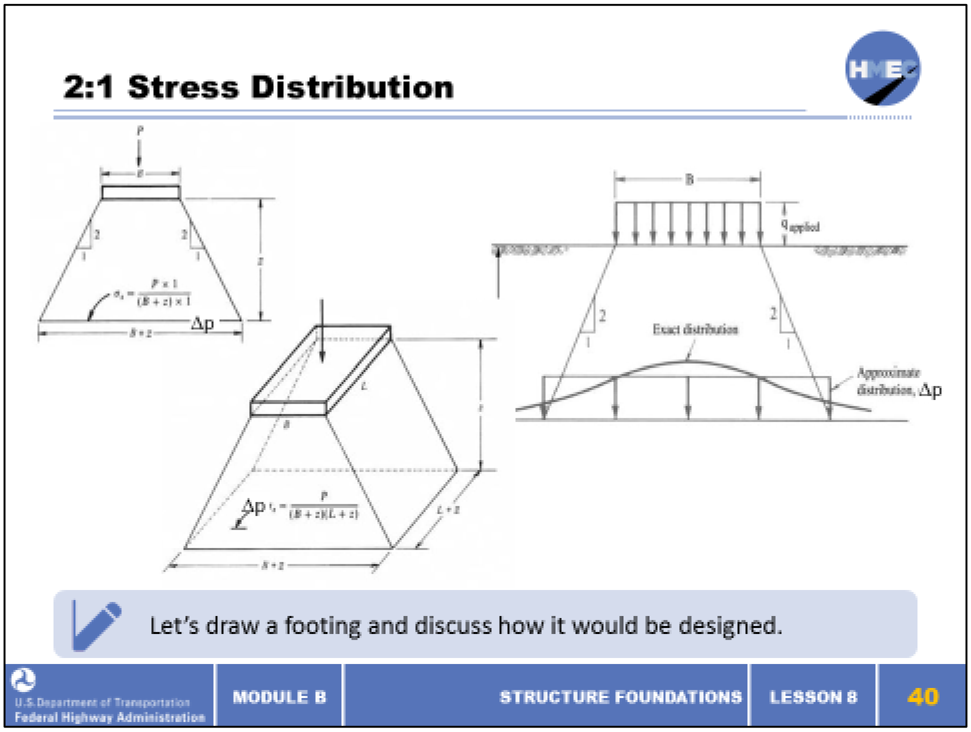
Slide 39



In 1885, Boussinesq developed solutions based on elastic theory for the stress below a point load. These have been applied to soils, which do not meet any of the assumptions made to derive the solutions. However, it has been found that when appropriate modifications and judgment are used they yield acceptable estimates of the stresses below an external load. Also note the increase in stress obtained from the charts must be added to the in-situ stress from existing overburden to get the total stress under a footing.

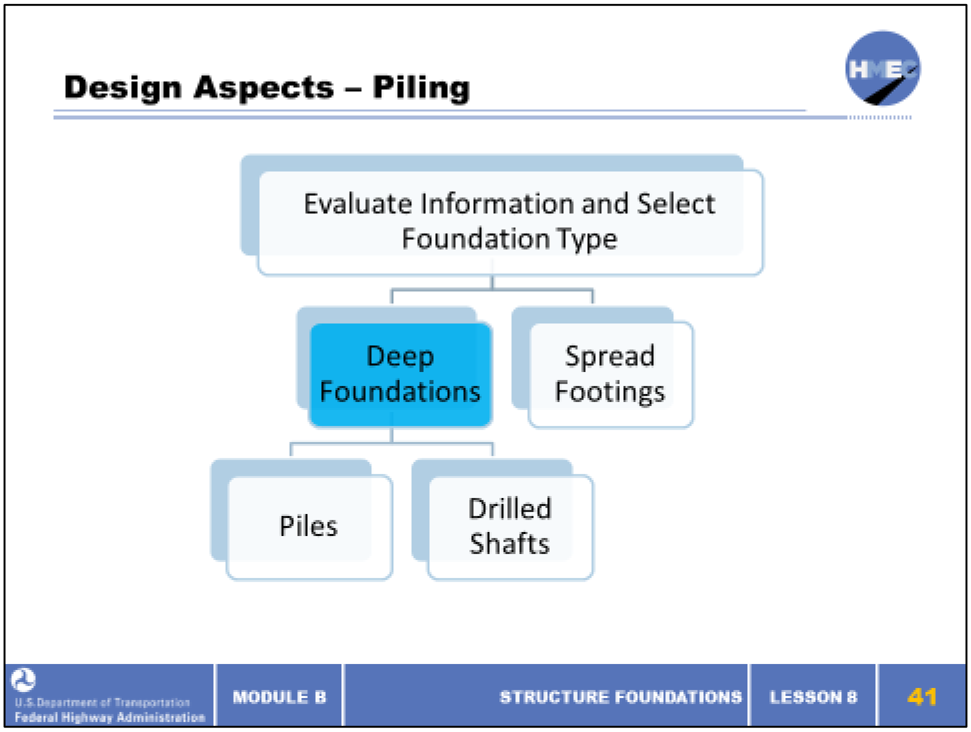
Three key factors can be obtained from the chart, DOSI, spread of load laterally with increasing depth and reduction of load with depth and laterally.

Slide 40



The stress at any depth can be calculated by using the equations shown on the slide. As can be seen on the right diagram, the distribution obtained is a reasonable approximation of the more exact method.

Slide 41



You might think that a deep foundation is a deep foundation and no further evaluation is needed. Over the next several slides we will discuss how they differ in most of the aspects we discussed earlier, including load transfer mechanisms, redundancy, cost, and construction requirements—to mention a few.

Drilled shafts are capable of carrying much more load than piling so they typically replace several piles, resulting in a smaller footing (or perhaps eliminating the footing). However, since one shaft replaces several piling they have less redundancy. Drilled shafts have large lateral load capacity and since they are circular they have the same lateral capacity in all directions, important in seismic events. Drilled shafts require excavation, which can be an issue if the site contains contaminated soils. Piles are driven into the ground, a very noisy operation, which may result in restrictions of working hours. Drilled shafts require special installation equipment and experienced contractors; not all contractors are qualified to install drilled shafts.

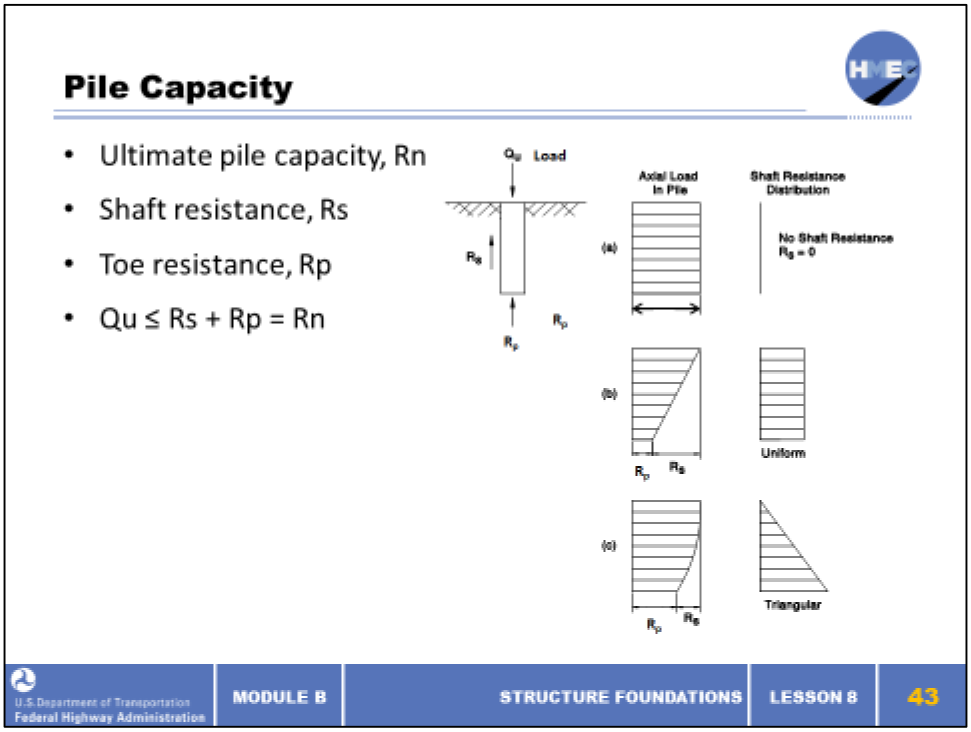
Slide 42

Design and Construction Geotechnical Terminology

- Specific Terminology for Deep Foundations
  - Static driven pile capacity
  - Nominal (ultimate) driven pile capacity
  - Driving capacity
  - Restrike capacity
  - Shaft resistance in driven piles
  - Side resistance in drilled shafts
  - Toe resistance for driven piles
  - Tip or base resistance for drilled shafts

U.S. Department of Transportation Federal Highway Administration | MODULE B | STRUCTURE FOUNDATIONS | LESSON 8 | 42

Slide 43



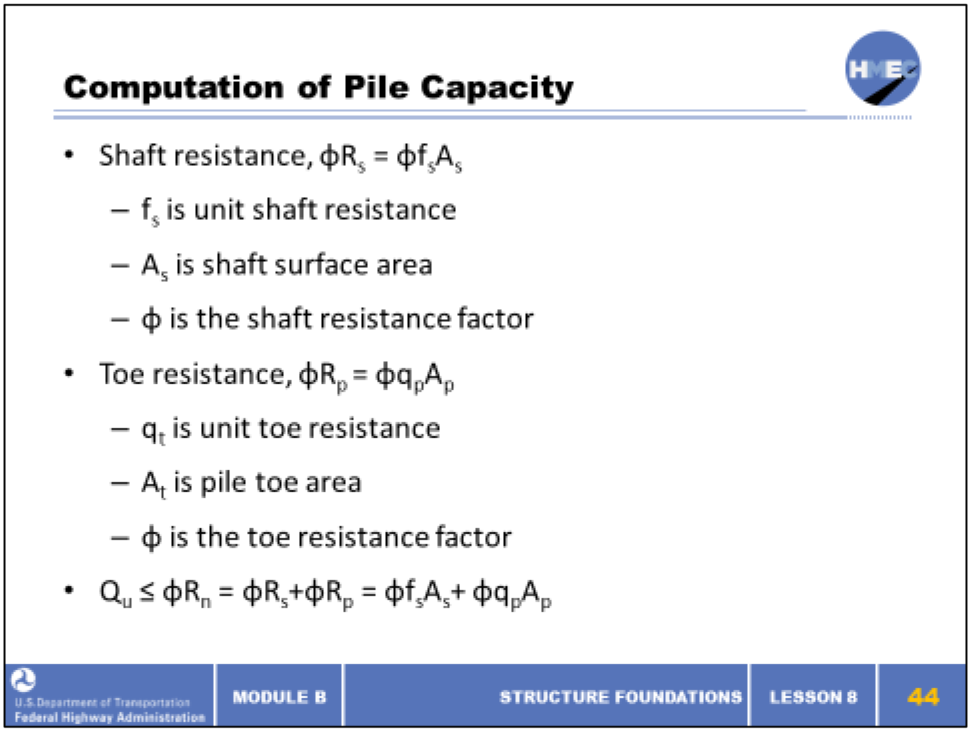
The shaft resistance is a function of pile and soil parameters. The key pile parameters are size, length, shape, and material. The key soil parameters include shear strength, unit weight, and effective stresses. Toe resistance is a function of the tip area and shape of the pile, the bearing capacity factors, and the effective stress. Movement of the pile relative to the soil must occur to develop shaft and toe resistance. The shaft resistance is mobilized (that is developed) during driving of the pile as is most of the toe resistance. Therefore, the nominal (ultimate) resistance for a driven pile is typically taken as the sum of the shaft and toe resistance. This is simple approach is commonly used for all piles except large diameter piles, since all of the toe resistance may not be developed during driving for large diameter piles.

Pile capacity can be estimated in the office by static analysis using soil properties obtained from the soil investigation and lab testing or in the field using load tests conducted on piling, dynamic monitoring of the pile during driving, wave equation analysis, or driving formulas. We will discuss the use of these methods later in this lesson.

____

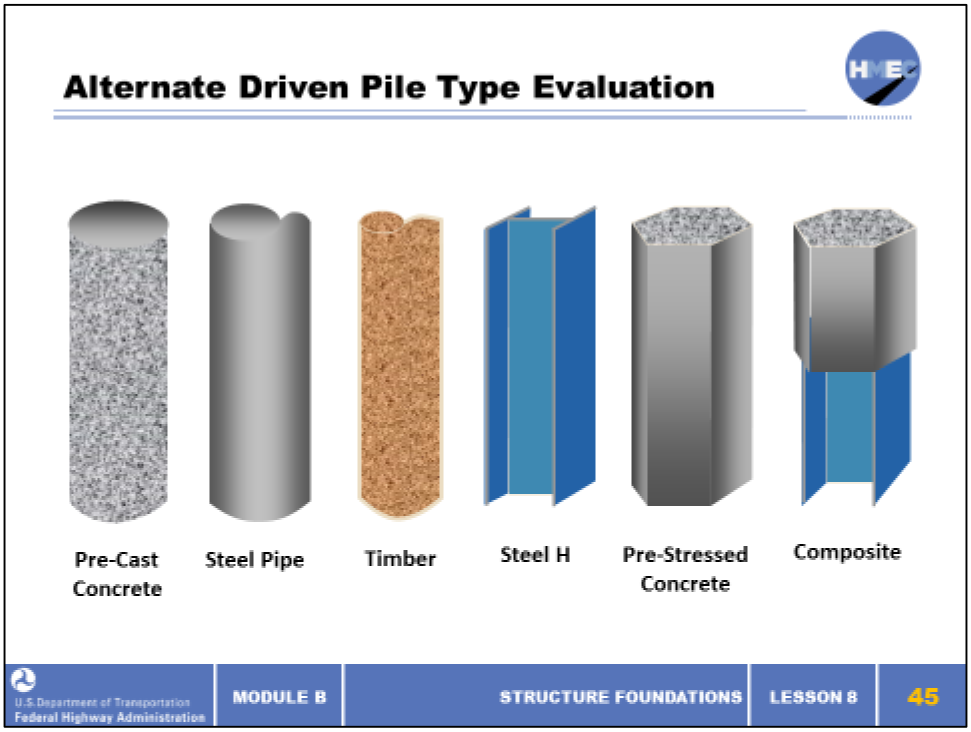
____

Slide 44



This slide shows the soil and pile parameters that influence pile capacity.

Slide 45



Piling is often described by its material composition, shape, displacement, or non-displacement, and the primary load transfer mechanism by which they develop their capacity. Provide a few examples. For instance, a steel H pile driven to point bearing or a pre-stressed concrete displacement friction pile.

Displacement piles are primarily friction piles that is obtaining most of their capacity from shaft resistance. Because they have a large cross-section, they tend to “displace” soil thereby increasing the shaft resistance. Non-displacement piles often are driven to point bearing, obtaining most of their capacity from toe resistance. However, many non-displacement piles develop significant shaft resistance, which should not be ignored as it can improve their cost effectiveness

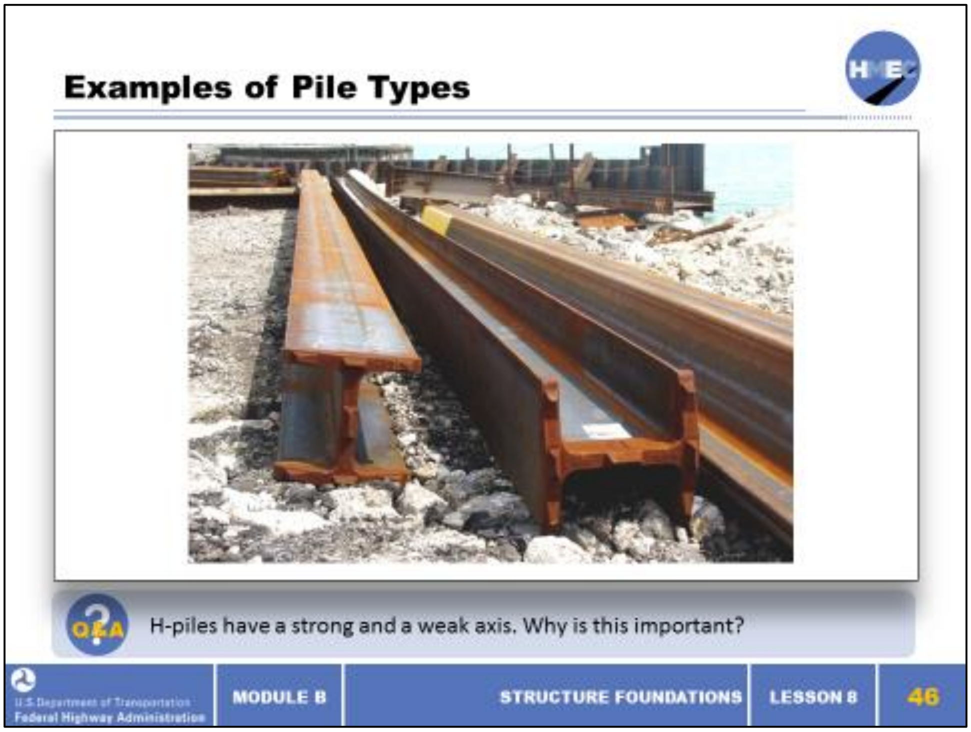
____

____

____

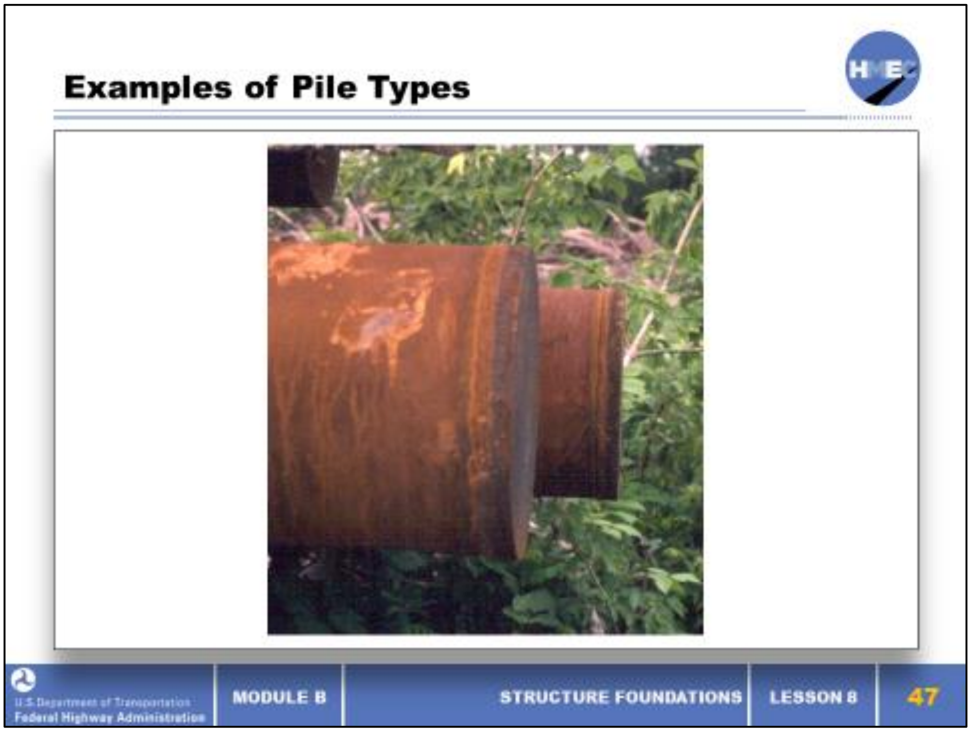
____

Slide 46



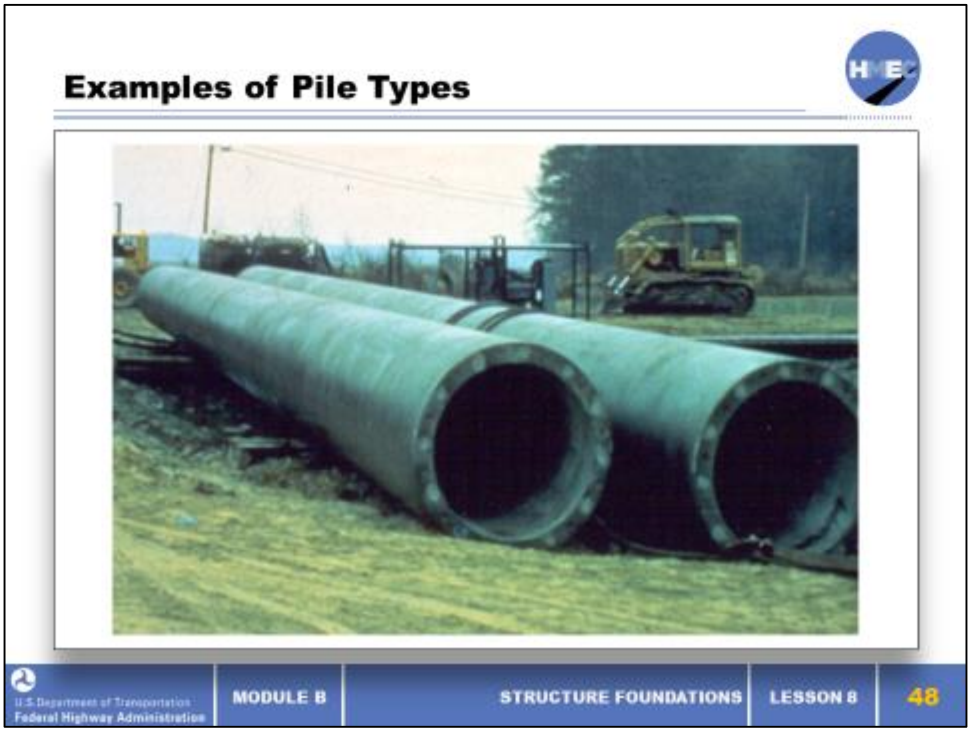
These are steel H piles with driving tips. The H-pile is a non-displacement pile. The driving tip is used when hard driving is expected or when driving into sloping rock.

Slide 47



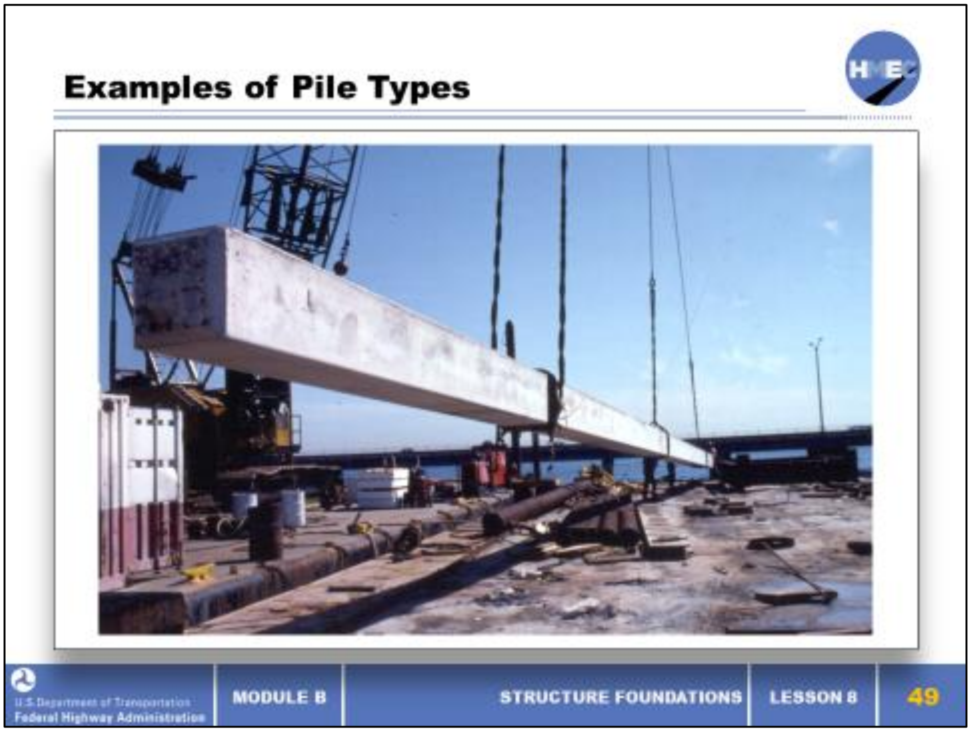
The closed end pipe is a type of displacement pile. The closed end pipe pile can be filled with concrete if added structural capacity is required. Pipe piles are also driven open ended; diameters up to 10 ft. have been driven on large bridge projects.

Slide 48



The post-tensioned cylinder pile may be a non-displacement or a displacement pile, depending on the soil that is being driven through. In some soils and driving conditions, a soil plug may form in the bottom section of the hollow cylinder and the pile will begin to function as a displacement pile. The issue is predicting if and when the soil plug will form. A geotechnical specialist should be involved. Cylinder pile up to 66 in. in diameter are common.

Slide 49



The pre-stressed concrete pile is a type of displacement pile.

Slide 50

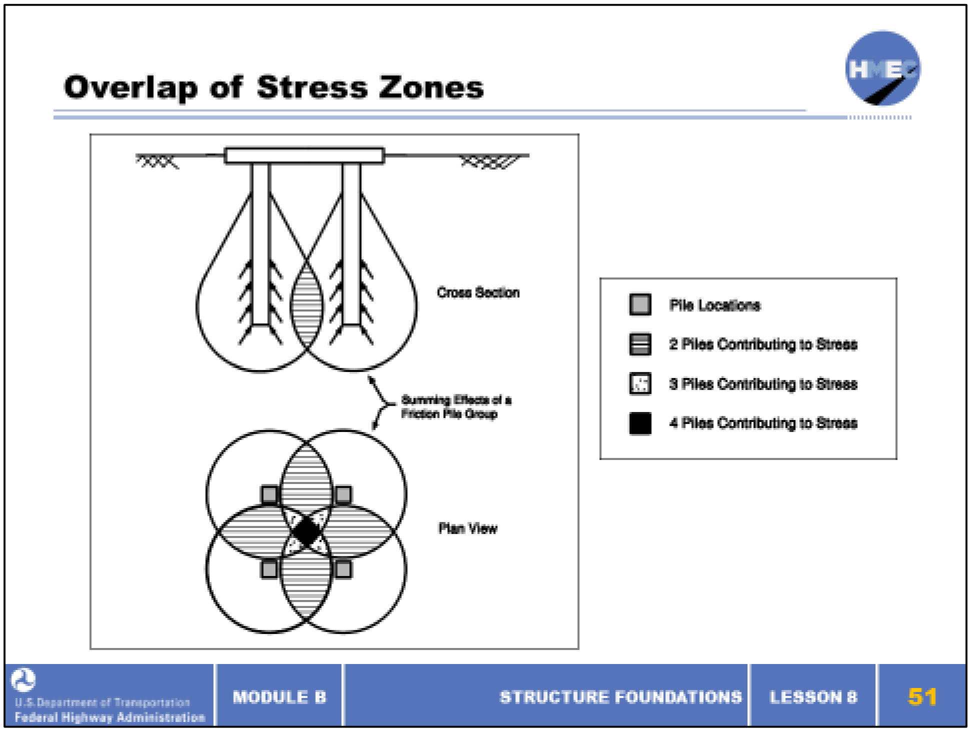
Pile Shape Effects

Shape Characteristics	Pile Types	Placement Effects
Displacement	Steel Pipe (Closed End), Pre-cast Concrete	• Increase lateral ground stress • Densify cohesionless soils, remolds and weakens cohesive soils temporarily • Set-up time may be 6 months in clays for pile groups
Nondisplacement	Steel H, Steel Pipe (Open End)	• Minimal disturbance to soil • Not suited for friction piles in coarse granular soils; piles often have low driving resistances in these deposits making field capacity verification difficult, thereby often resulting in excessive pile lengths
Tapered	Timber, Monotube, Tapertube, Thin-Wall Shell	• Increased densification of soils with less disturbance, high capacity for short length in granular soils

U.S. Department of Transportation Federal Highway Administration | MODULE B | STRUCTURE FOUNDATIONS | LESSON 8 | 50

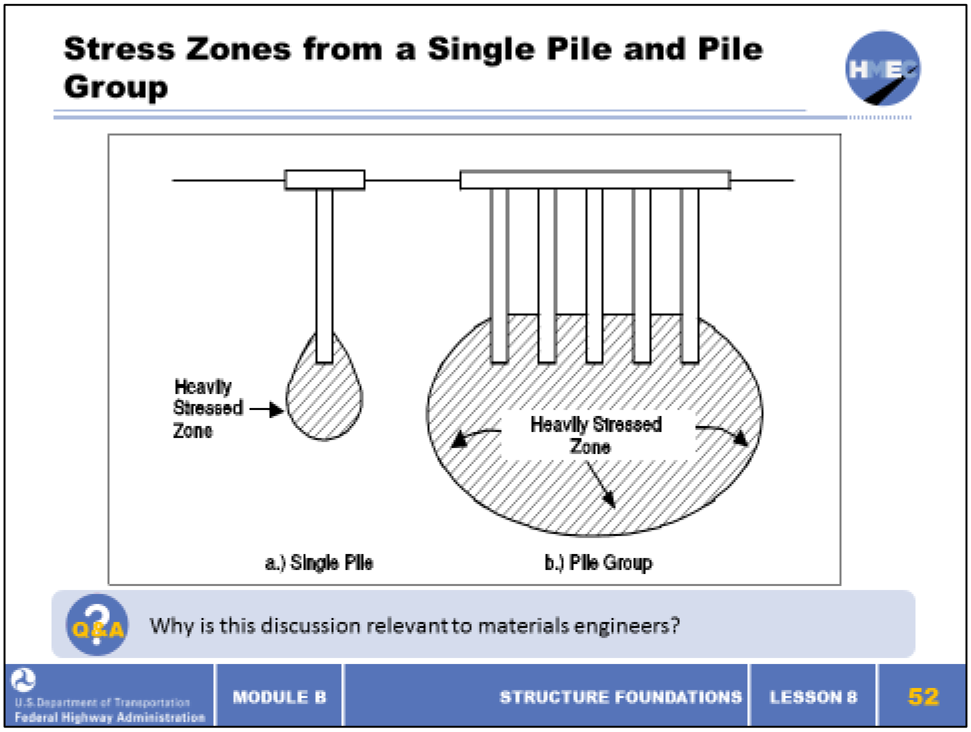
Granular soils are densified during driving displacement piles, but cohesive soils are "disturbed" resulting in temporary strength loss, especially in saturated clays.

Slide 51



The soil adjacent to the pile has the highest stress as it provides shaft resistance for the pile capacity; however, the stress increase in surrounding soil extends out from the pile 2–3 pile diameters. The overlap of stress zones is why our specifications require that piles be spaced a minimum distance from other piles in a footing. Typically the spacing is 2.5–3 pile diameters.

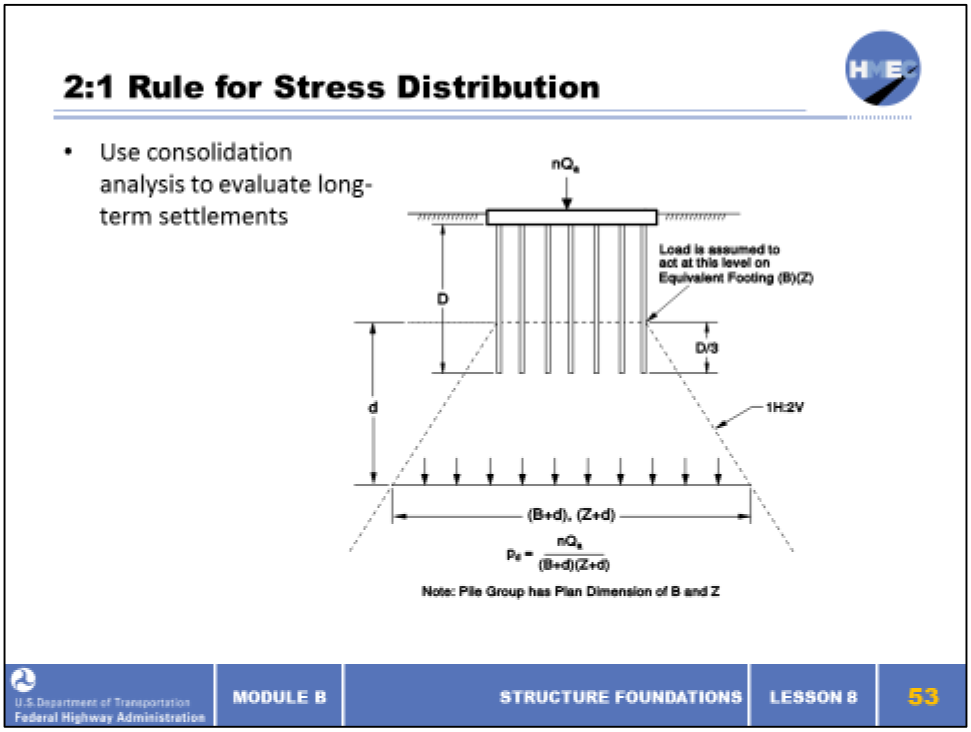
Slide 52



This schematic should be discussed so that the participants understand the DOSI of a pile group.

____
____
____
____
____
____
____
____
____
____
____

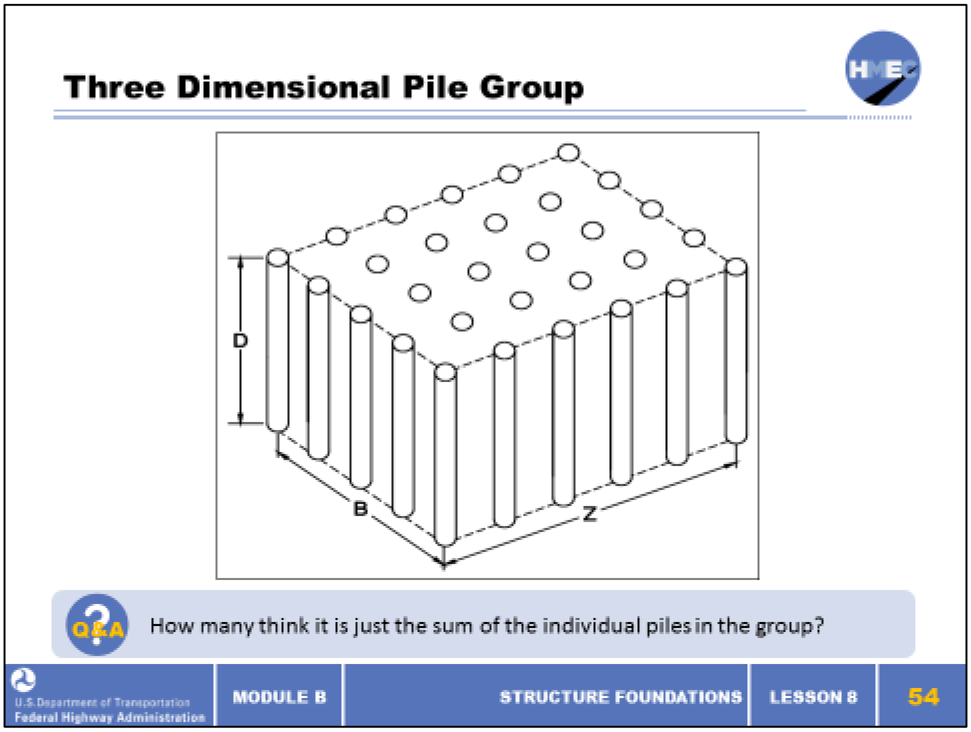
Slide 53



Recall the 2:1 stress distribution method for spread footings—we use a similar procedure to evaluate stress below a pile group. This method of estimating stress distribution below a pile group uses the concept of an equivalent footing at a distance of ⅓ the pile length above the pile tip. The load applied to the pile cap is assumed to act at that location and spreads outward in four directions as shown on the slide. The unit stress in the soil can be found at any depth by dividing the load on the pile cap by the area of the base at that depth. The unit stress calculated can be used to estimate settlement of the pile group, which can be important for pile groups driven into a thick dense layer overlying a soft compressible layer.

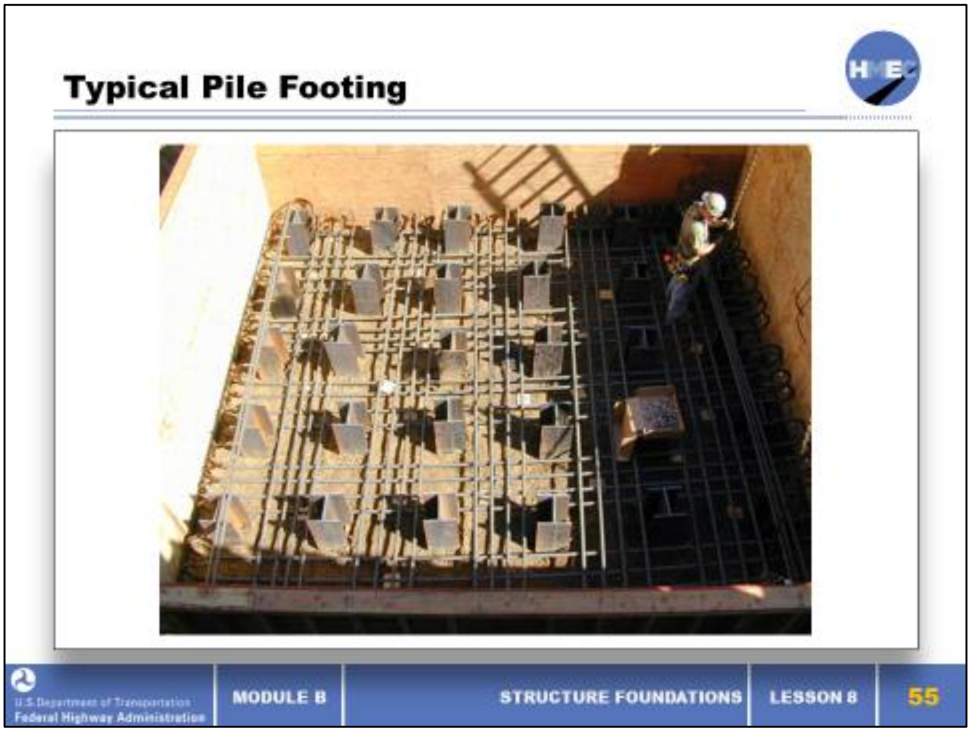
The stress distribution model will vary depending on the soil around and below the piling.

Slide 54



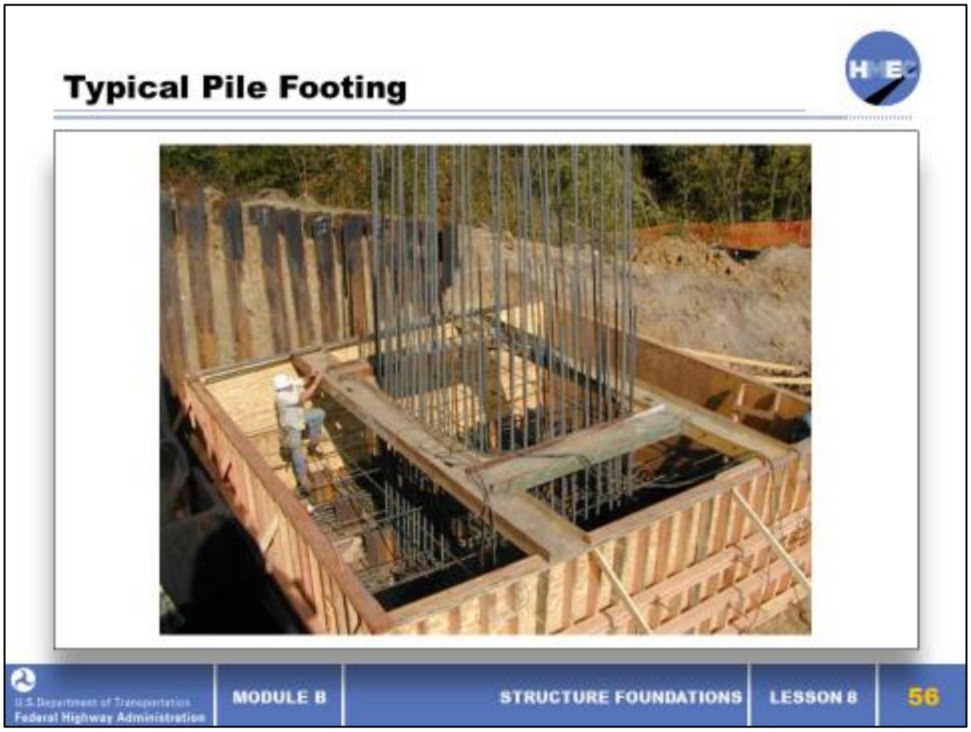
Typically we calculate the capacity of individual piles, but what is the capacity of a large group of piles?

Slide 55



This is a photo of piles driven, a pile cap formed, and a pile reinforcement placed.

Slide 56

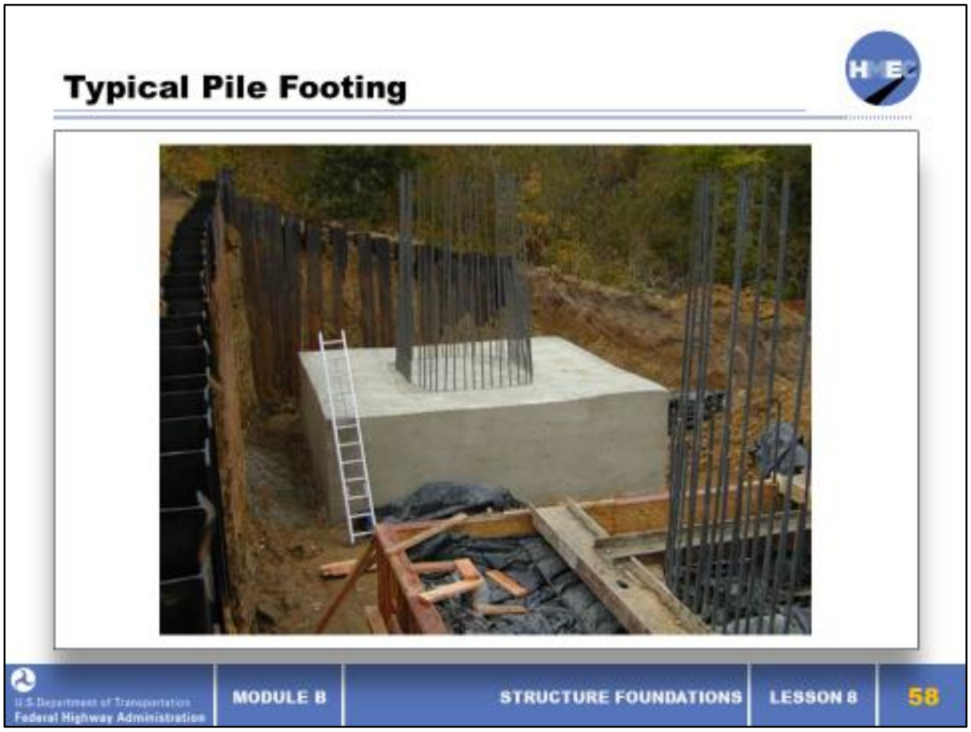


This is a placed column reinforcement.

Slide 57

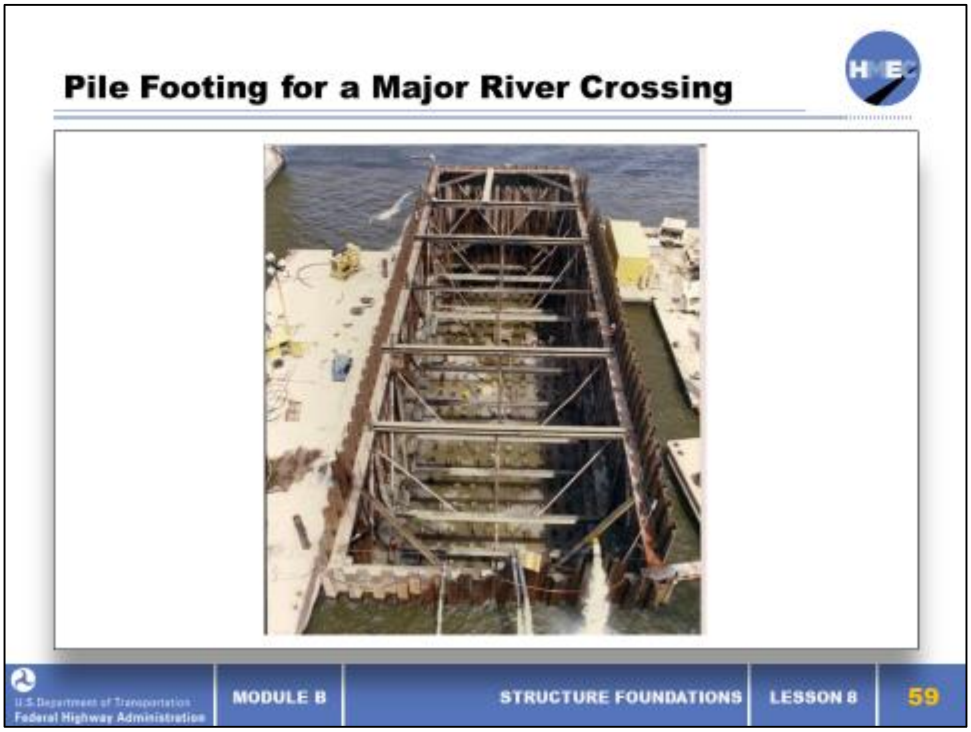
This is a photo of a placement of pile cap concrete.

Slide 58



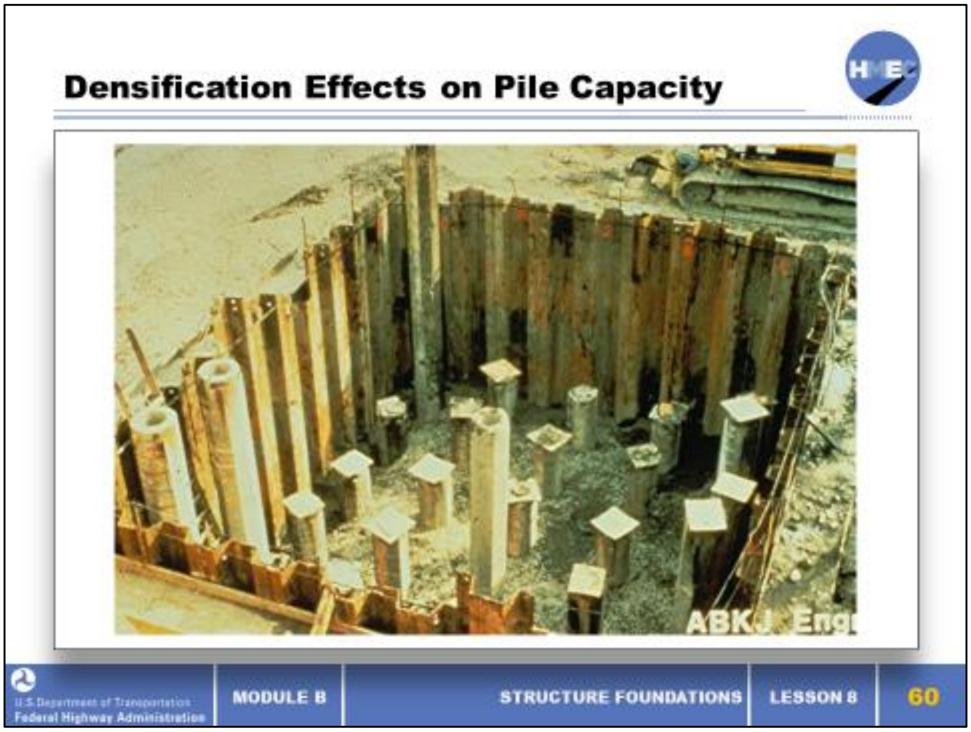
This is a photo of one footing completed with forms stripped, while the other footing is being cured. The contractor used an H-pile retaining wall to stabilize their excavation.

Slide 59



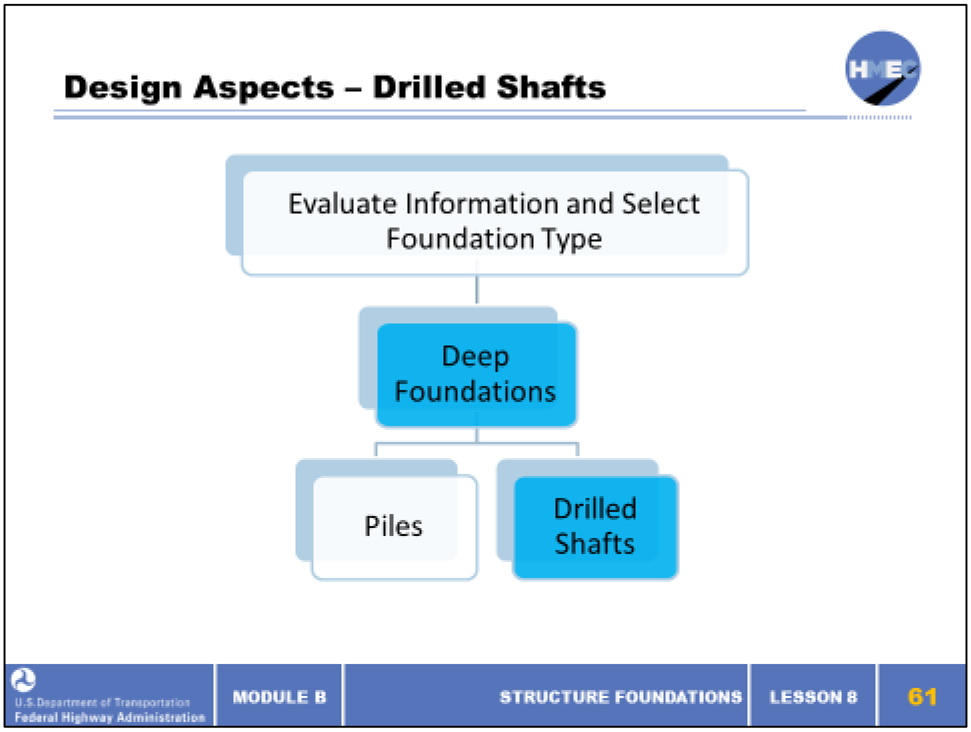
Cofferdam is a braced sheet-pile retaining system to hold back the Mississippi River. We will discuss that type of retaining structure in Lesson 9. Interlocking sheet piling were driven with a vibratory hammer into the granular streambed. Penetration had to ensure stability once the material inside the cofferdam was excavated to the elevation planned for the concrete seal. Piles were driven with an underwater hydraulic hammer. The concrete seal was placed in the bottom of the cofferdam to allow de-watering and placement of the footing and pier stem in the dry.

Slide 60



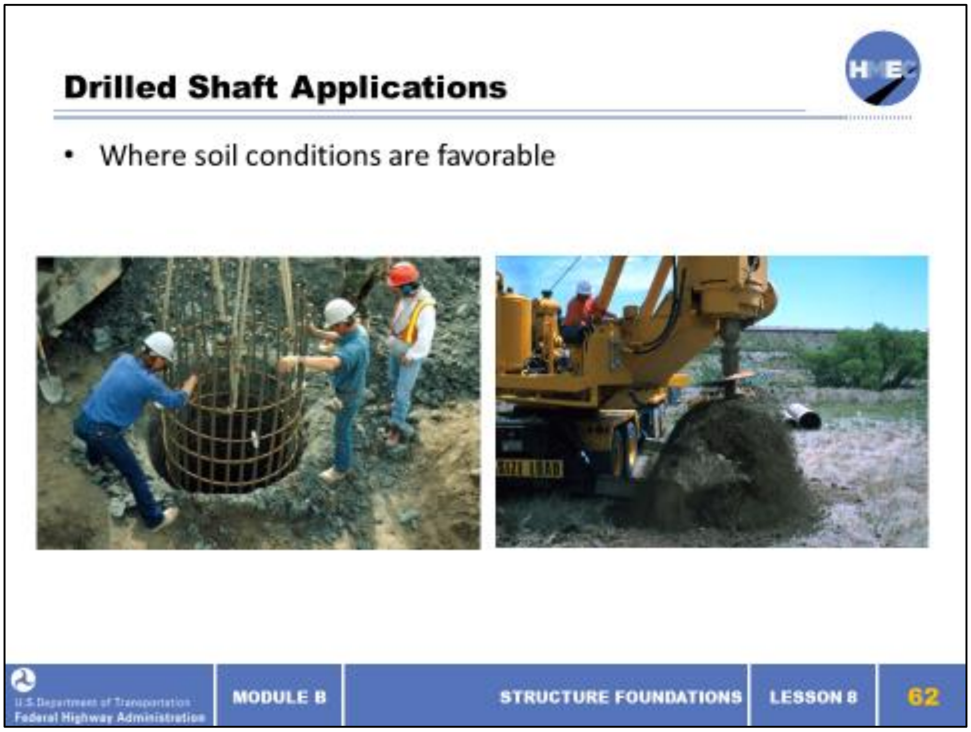
In this case, driving of the initial piles densified the soils within the cofferdam so much that later piles could not be driven to the required tip elevation. The solution would have been to use non-displacement piles and drive them in a specific sequence.

Slide 61



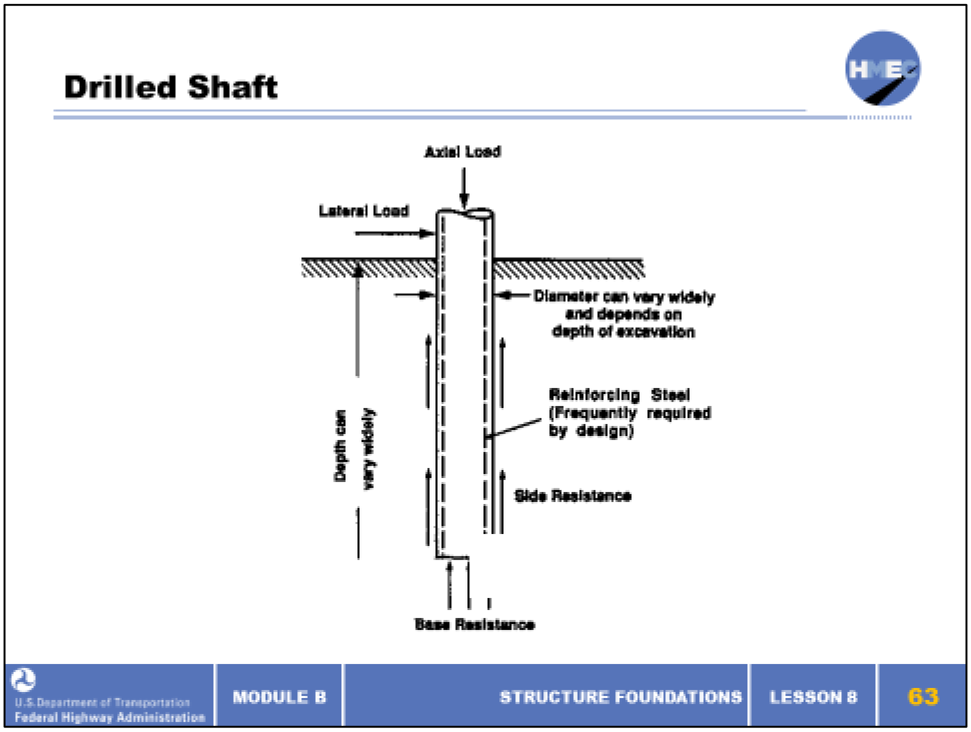
The load transfer mechanism, group effects, settlement, and suitability of soil and rock are similar.

Slide 62



Drilled shafts are economical where cohesive soils or easy drilling conditions exist, or when a strong bearing stratum is present at a depth which is easy to achieve relative to the size of the structure and/or shafts to be employed. It is appropriate to discuss some conditions were drilled shaft construction is difficult, such as conditions with high groundwater, contaminated soil, deep soft soils without a competent bearing stratum (deep friction piles may be more economical), or conditions where site access limitations may favor micro piles.

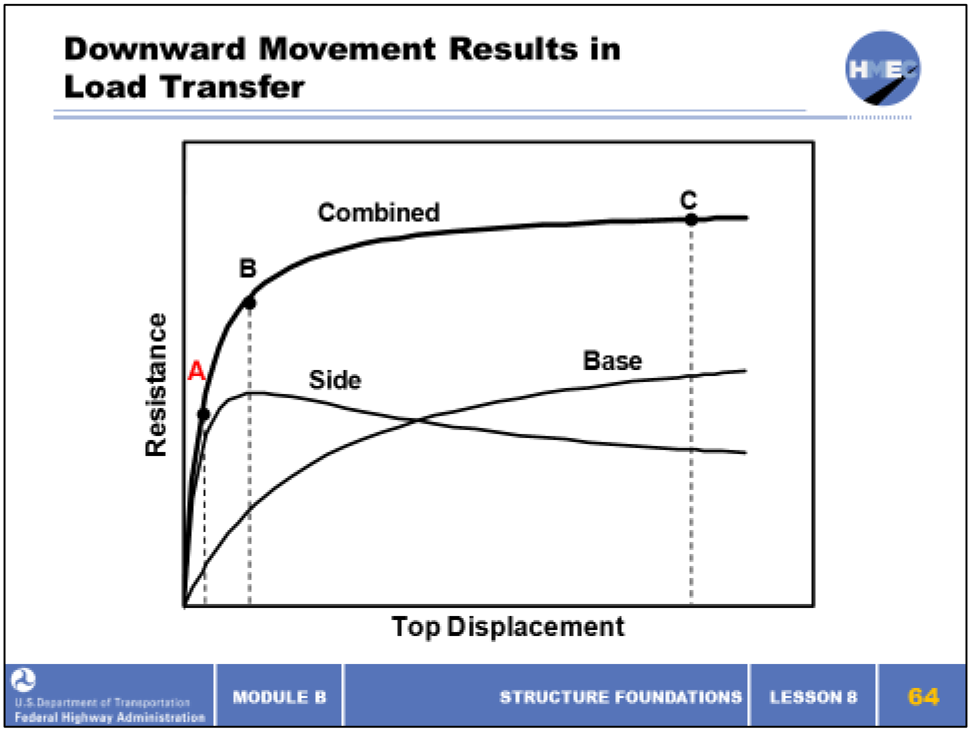
Slide 63



The load transfer mechanism for drilled shafts is similar to driven piles in that the capacity of the shafts is derived from resistance along the shaft and resistance at the base of the shaft. As with the driven pile, movement is required to develop capacity. However, since the drilled shaft is cast in place in a drilled hole, capacity is not developed during driving as it is with a pile. Therefore, movement caused by the structural load is required to develop the drilled shaft capacity.

Note that side resistance develops from shearing stress at the soil-concrete interface and requires only about 0.4–0.6 in. of movement to develop full side resistance. However, because the base of the drilled shaft is large, the movement required to develop full base resistance can be between 4–10% of the base diameter, though there are emerging processes for post-grouting beneath shaft bases to fully develop base resistance without movement. Much of the movement required to develop side resistance occurs during construction as the dead weight of the structure is applied.

Slide 64



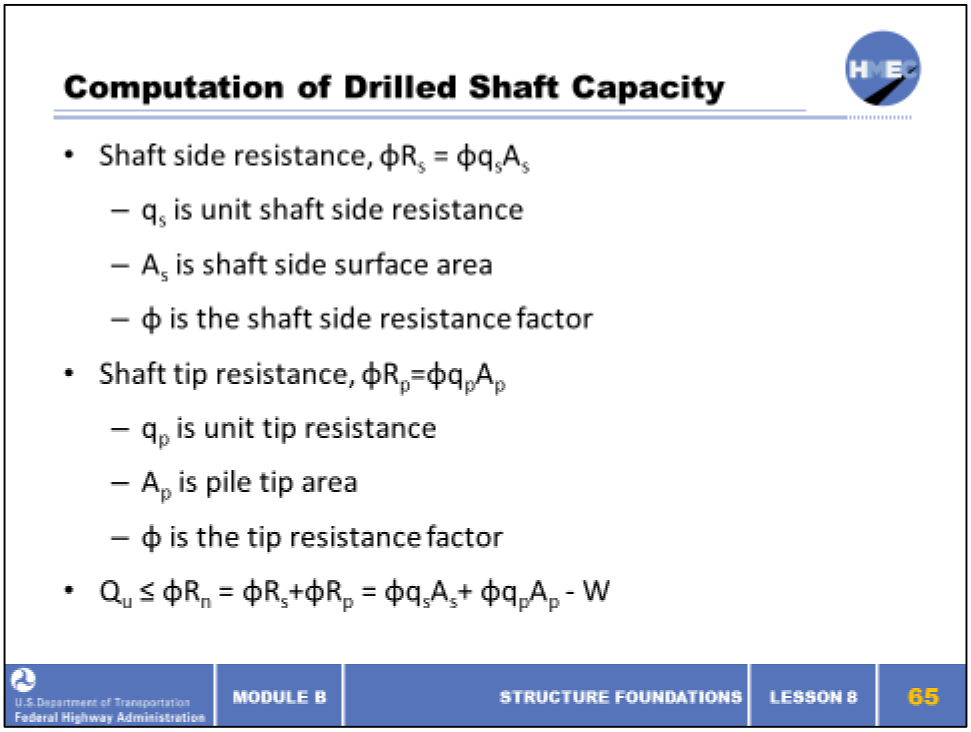
This figure shows the general relationship between axial resistance and downward displacement. Three components of resistance are shown: 1. side resistance R_s, 2. base resistance R_b, and 3. combined (total) resistance.

As axial load on the shaft increases from zero, the shaft displaces downward and side resistance in shear is mobilized (point A). This transfer of load to the surrounding soil or rock results in decreasing load with depth. At this point, load is transferred predominantly in side resistance and load transmitted to the base may be small. With increasing load, the full side resistance is mobilized (point B), typically at a displacement of approximately $\pm$ ½ in.

Further increases in load beyond point B must be resisted by the base, until the maximum base and combined resistances are reached (Point C). The displacement required to mobilize the maximum base resistance varies, but research suggests that maximum resistance is reached at a displacement equivalent to about 4 to 5% of the shaft diameter for bearing in cohesive soil or rock and about 10% of the shaft diameter for bearing in cohesionless soils.

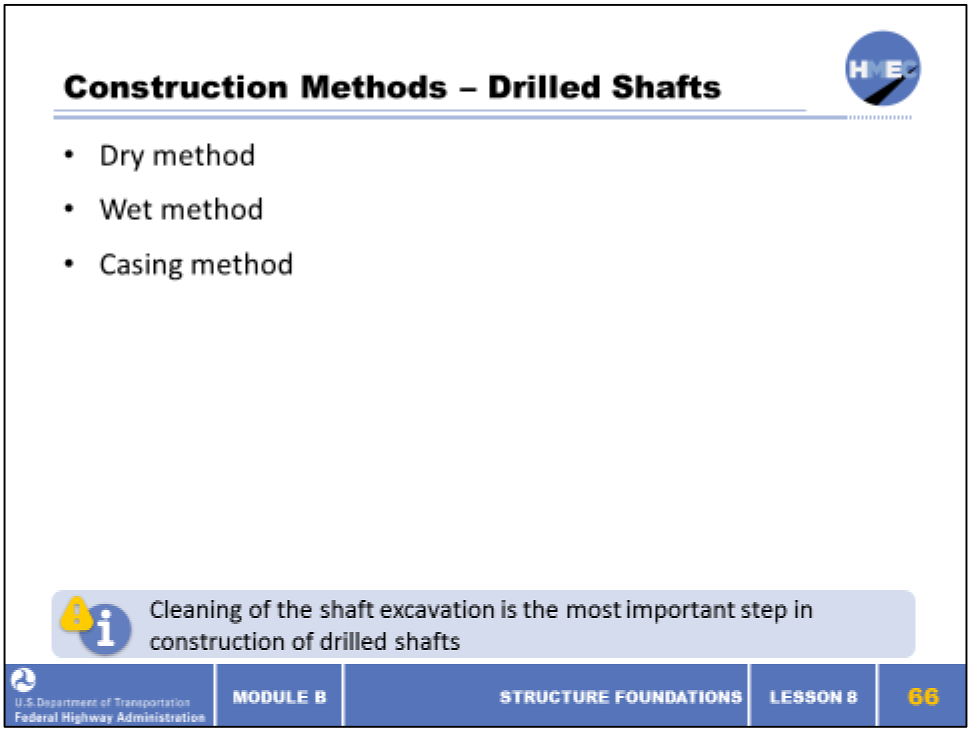
Between points B and C, side resistance may remain constant or change (increase or decrease) depending upon the stress-strain behavior along the interface between the shaft and soil or rock. In some cases, the shaft continues to exhibit increasing resistance with continued downward displacement, thus a well-defined maximum total load is not achieved.

Slide 65



Because drilled shafts have a large tip area and can develop large tip resistance, they are an excellent selection for a foundation system when rock or IGMs are located within 50–80 ft. of the ground surface.

Slide 66



Drilled shaft construction methods are discussed under design aspects, because construction quality and techniques influence the design capacity that can be achieved.

Because drilled shafts require a hole to be drilled into the ground, an appropriate construction method must be used to ensure the hole remains open until after the reinforcement and concrete is placed. The correct method will depend on the soil and groundwater conditions at the site as well as the proximity of adjacent structures.

An advantage of drilled shafts is that a large diameter hole is being drilled and the excavated material can be inspected as the drilling progresses. Close communication between the inspector and the geotechnical specialist regarding the excavation cuttings will ensure that the drilled shaft has penetrated the expected soil layers and is tipped in the base material that was anticipated in design. As a materials engineer involved in subsurface investigation and testing, your role in the accurate identification of soils, their engineering properties, and elevations is critical.

Selection of the appropriate construction method for conditions and construction quality have a tremendous influence on the load-carrying capacity of drilled shafts. Proper cleaning of the shaft base is perhaps the most important consideration in the construction of shafts. To ensure that the proper method is selected and proper/proven construction techniques are used it is critical that the drilled shaft contractor be experienced. Drilled shaft construction specifications should require that the contractor, including the superintendent and crew, have several years of experience constructing drilled shafts similar in size and complexity to those required on your project.

____

____

____

____

____

____

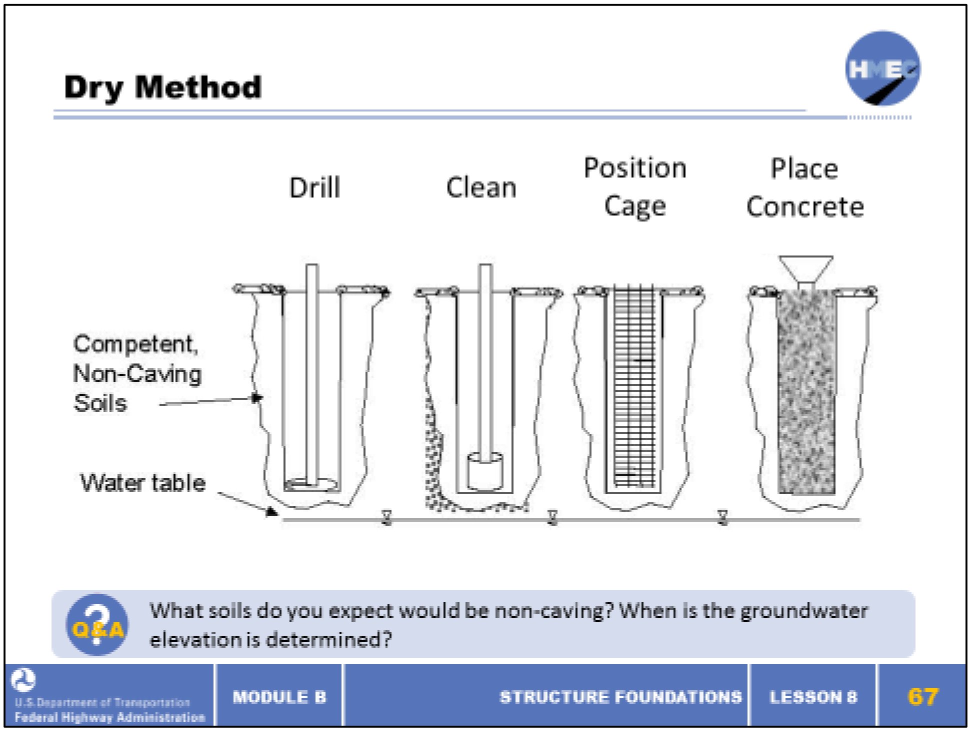
____

____

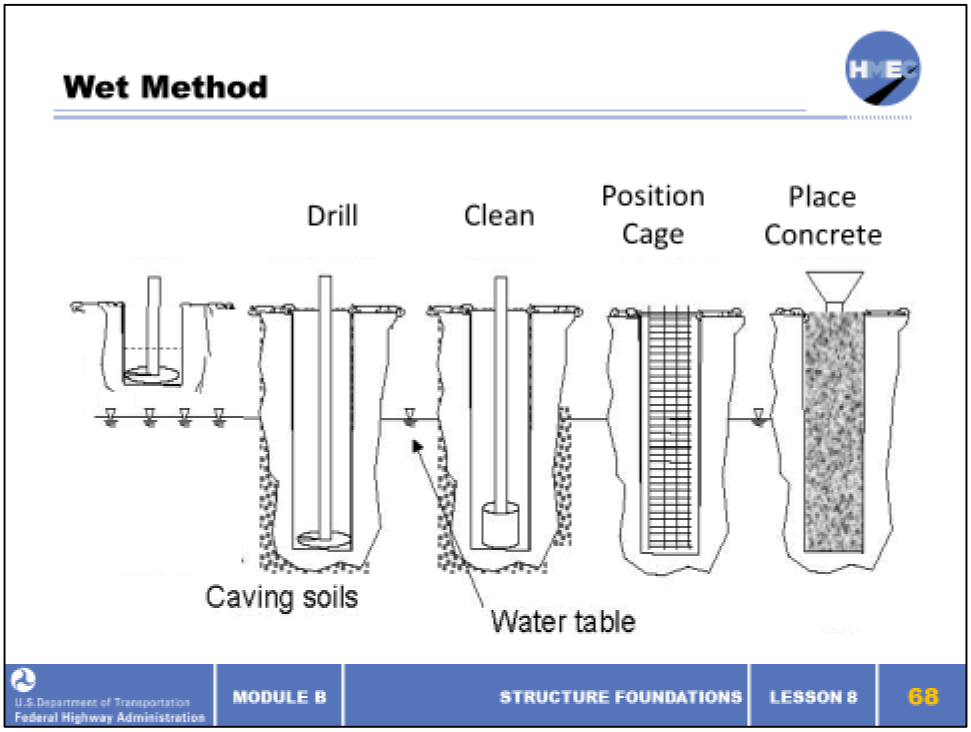
____

____

Slide 67


Dry Method
HMEC
Drill
Clean
Position Cage
Place Concrete
Competent, Non-Caving Soils
Water table
Q&A
What soils do you expect would be non-caving? When is the groundwater elevation is determined?
U.S. Department of Transportation Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
67

Slide 68



A slurry is a mixture of water and mineral or polymer additive to create a slurry that is added to the hole during drilling and helps stabilize a soil that would normally cave into the excavation. The wet method of drilling is more expensive and requires more specialized equipment than the dry method. Having an experienced contractor perform the work is essential to achieving a quality drilled shaft when using the wet method. The most common mineral slurry uses bentonite clay as the mineral additive.

____

____

____

____

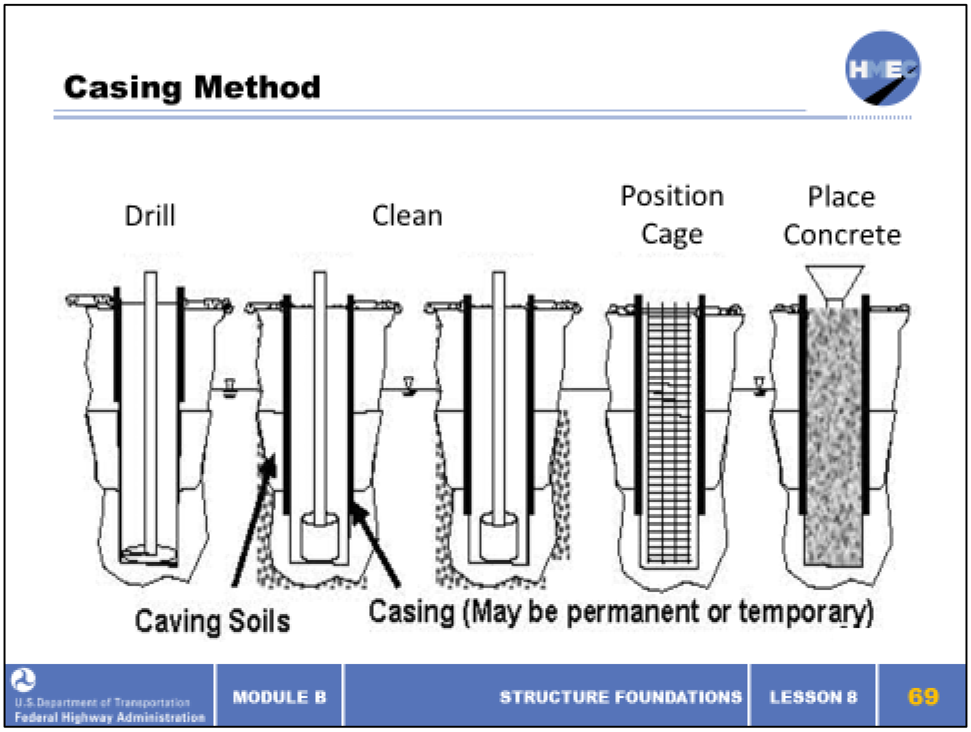
____

____

____

____

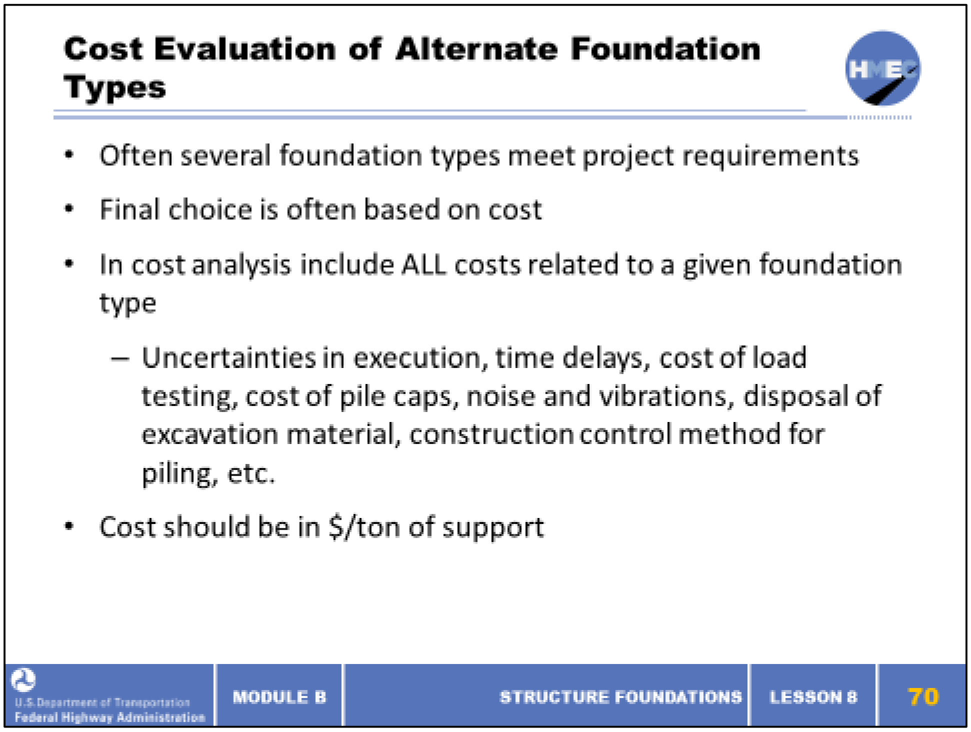
Slide 69



The wet method may not be capable of stabilizing some soil deposits such as loose coarse sands or gravels. A casing can be inserted into the ground through the problematic layer to allow drilling to proceed. The casing once installed can be left in place (permanent) or extracted as the concrete is placed (temporary). An example of the casing method with permanent casing would be to penetrate beyond a void in karst topography.

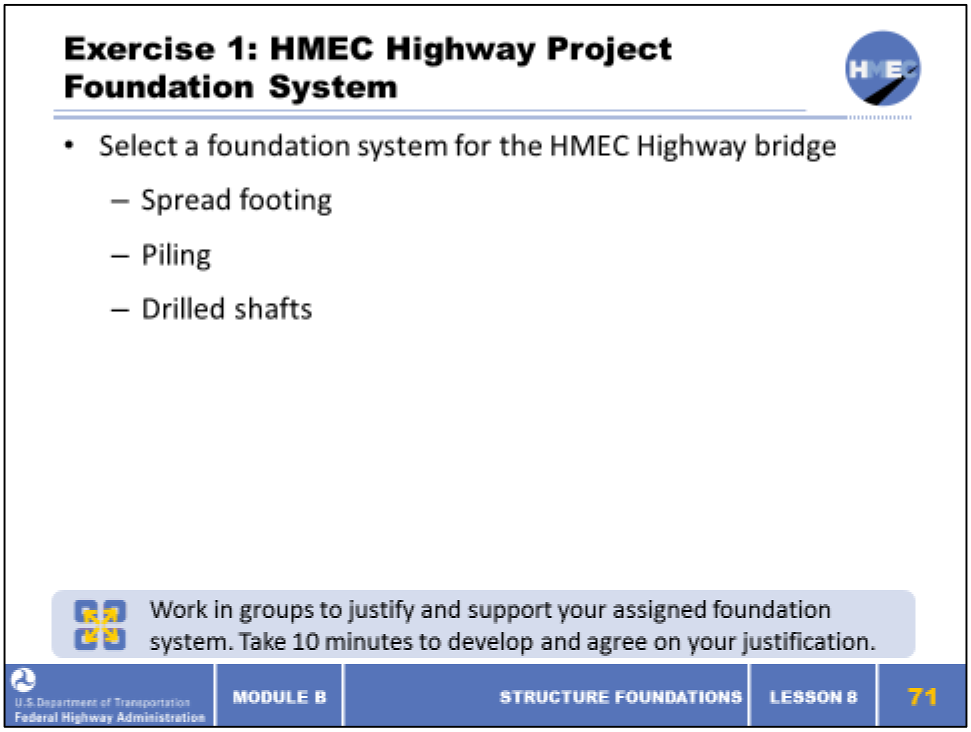
The casing can be placed by vibratory methods or by drilling a short distance below the casing to allow the casing to be pushed into the ground. The casing method is often used in combination with the dry or wet methods.

Slide 70

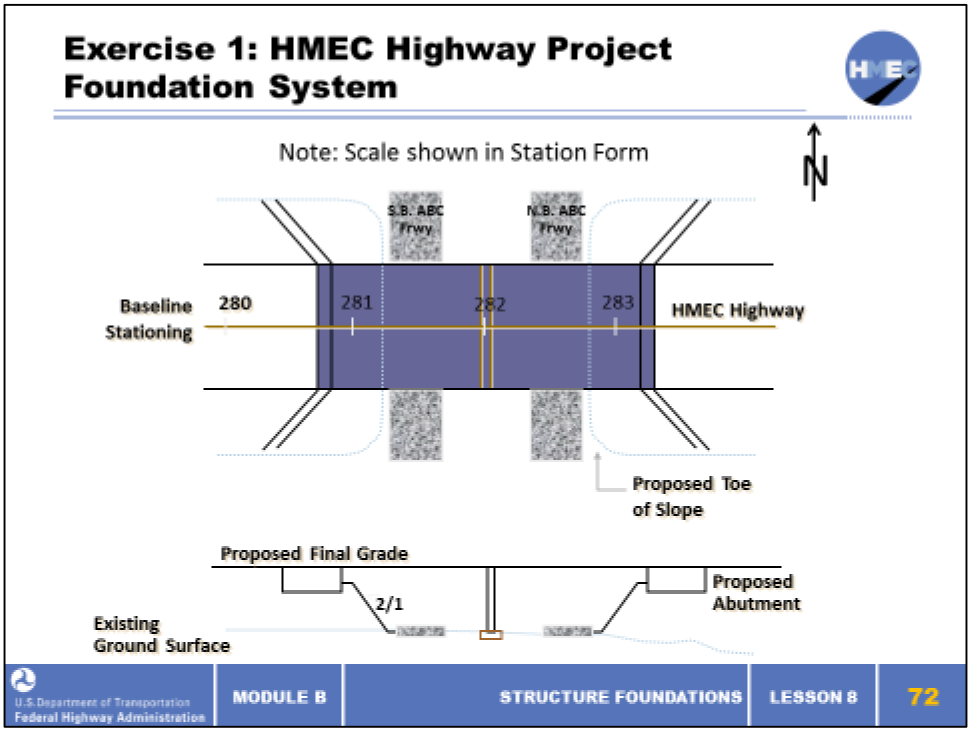


The cost method used must allow comparison of different foundation systems. One method that is often used is to calculate the cost/ton of support. That cost must include all the elements of a particular system. Each system will have some different elements, for instance to use a higher resistance factor for the piling design, a more expensive construction control method might be specified.

Slide 71

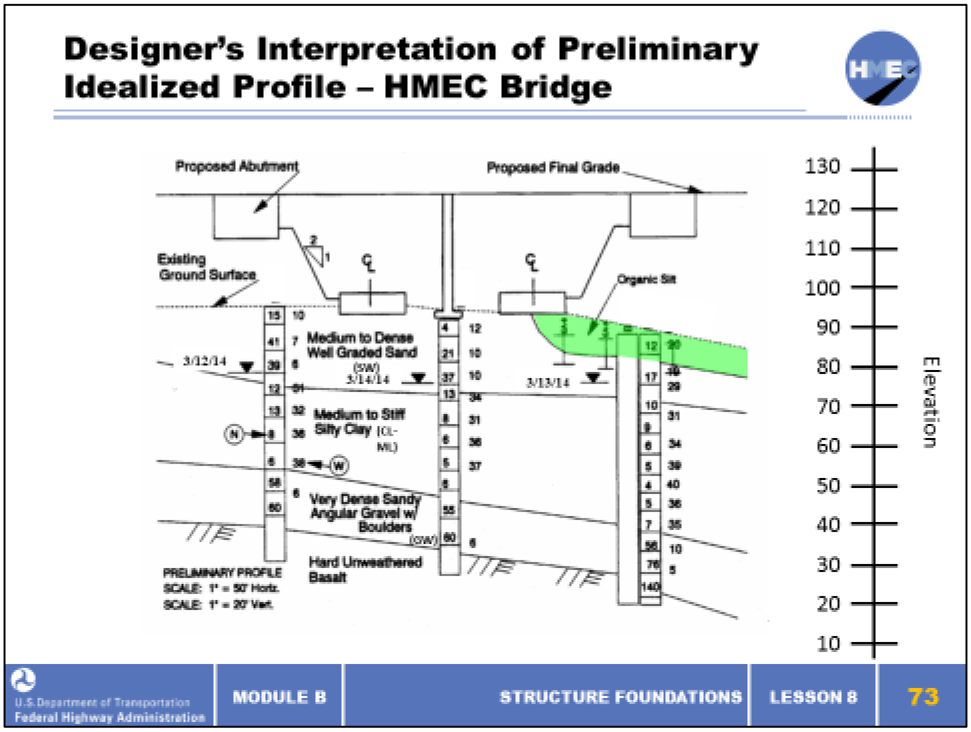

Exercise 1: HMEC Highway Project Foundation System
HMEC
• Select a foundation system for the HMEC Highway bridge
– Spread footing
– Piling
– Drilled shafts
Work in groups to justify and support your assigned foundation system. Take 10 minutes to develop and agree on your justification.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
71

Slide 72



The HMEC Bridge is shown here.

Slide 73



Profile for HMEC Highway Bridge.

____

____

____

____

____

____

____

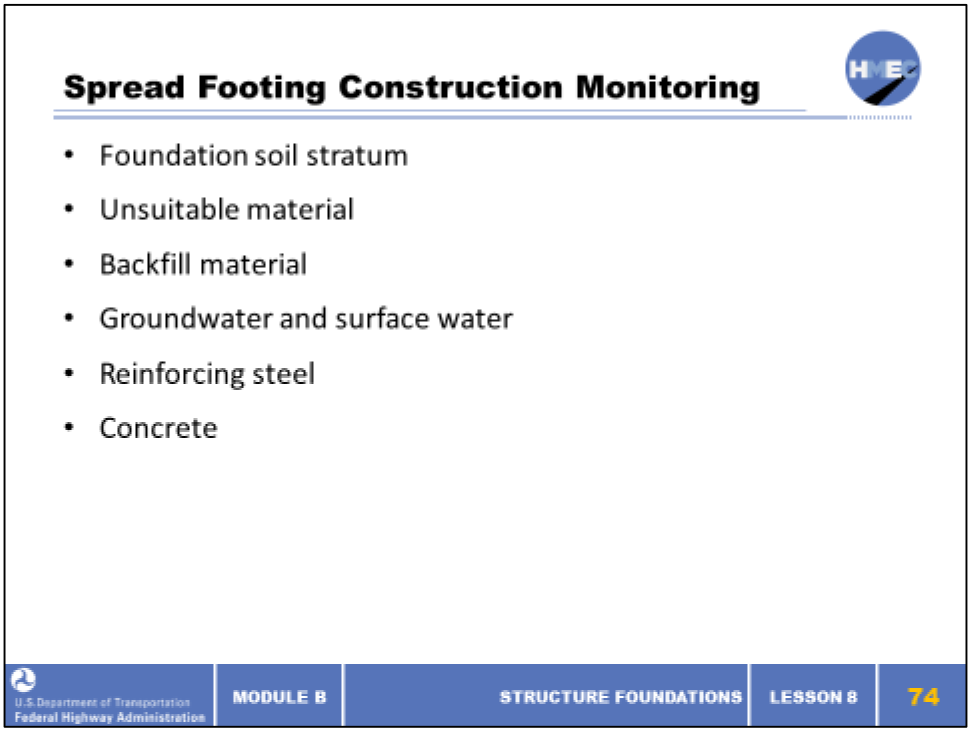
____

____

____

____

Slide 74



8.11 Construction Inspection: Construction inspection requirements for shallow foundations are similar to those for other concrete structures. In some cases, agencies may have inspector checklists for construction of shallow foundations. Table 8-16 provides a summary of construction inspection check points for shallow foundations. Throughout construction, the inspector should check submittals for completeness before transmitting them to the engineer.

8.11.1 Structural Fill Materials: Fill requirements should be strictly adhered to because the fill must perform within expected limits with respect to strength and, more importantly, within tolerance for differential settlement. Sometimes the area for construction of the fill is small, such as behind abutment and wing walls. In such situations, the use of hand compactors or smaller compaction equipment may be necessary.

When the construction of structural fills that will support shallow foundations is being monitored, particular attention should be paid to the following items:

The material should be tested for gradation and durability at sufficient frequency to ensure that the material being placed meets the specification.

The specified level of compaction must be obtained in the fill. Testing, if applicable, should be performed in accordance with standard procedures and at the recommended intervals or number of tests per lift.

If a surcharge fill is required for pre-loading, it should be verified that the unit weight of the surcharge fill meets the value assumed in the design.

Slide 75

It is critical to confirm that the foundation material is as expected in design, whether it is soil, weathered rock or sound rock. If the foundation material is designed for weathered rock, it is advisable to have the geotechnical engineer on site for the first footing excavation to verify that the inspector's determination of what foundation material is required. I was involved in a project where the inspector had the contractor excavate over 5 ft. of rock beyond the planned footing elevation "looking for good material," until the geotechnical engineer was finally called to the site and the footing elevation was set at the planned elevation.

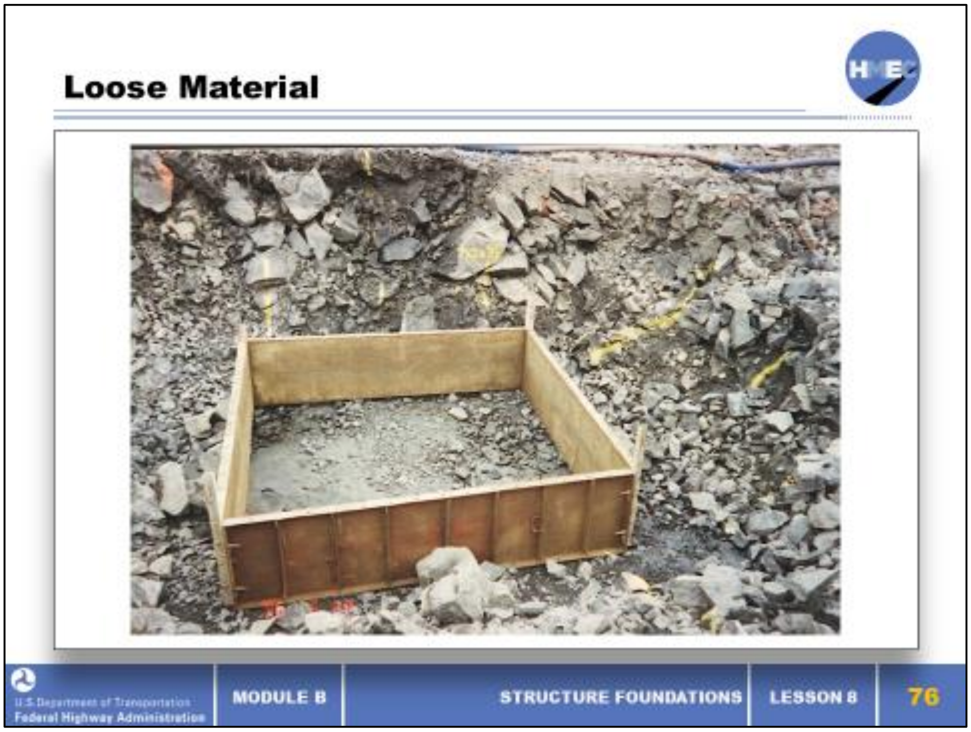
When the spread footing is founded on soil, compaction testing and proof rolling is required. Groundwater elevations should be monitored and the geotechnical specialist contacted if it not as expected. Ground water or surface water can have an adverse impact on the foundation performance.

____

____

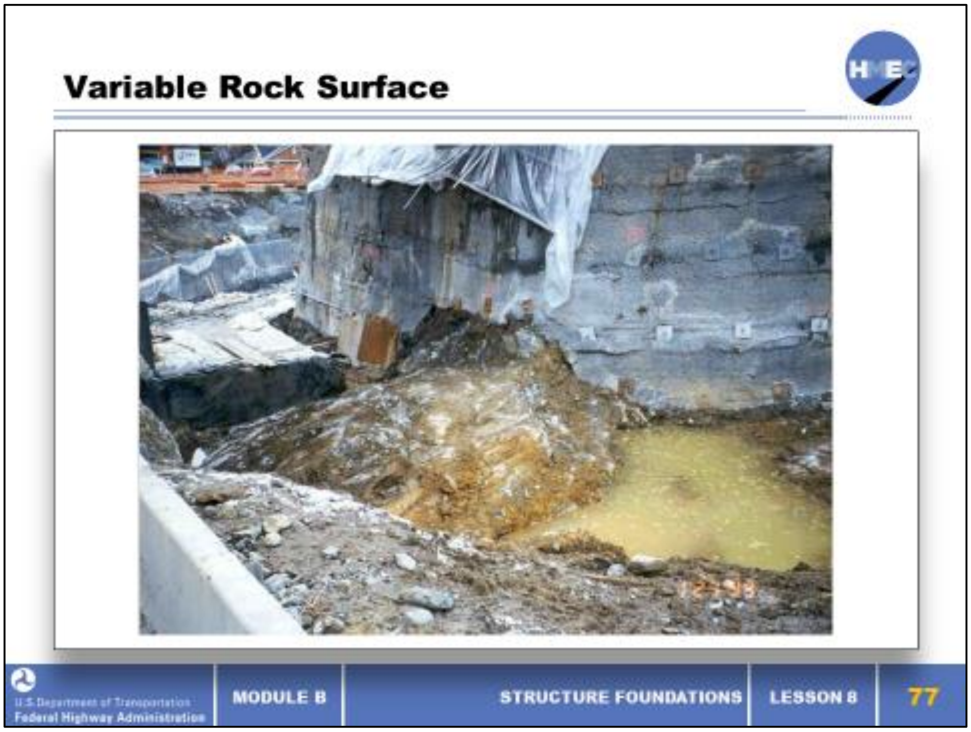
____

Slide 76



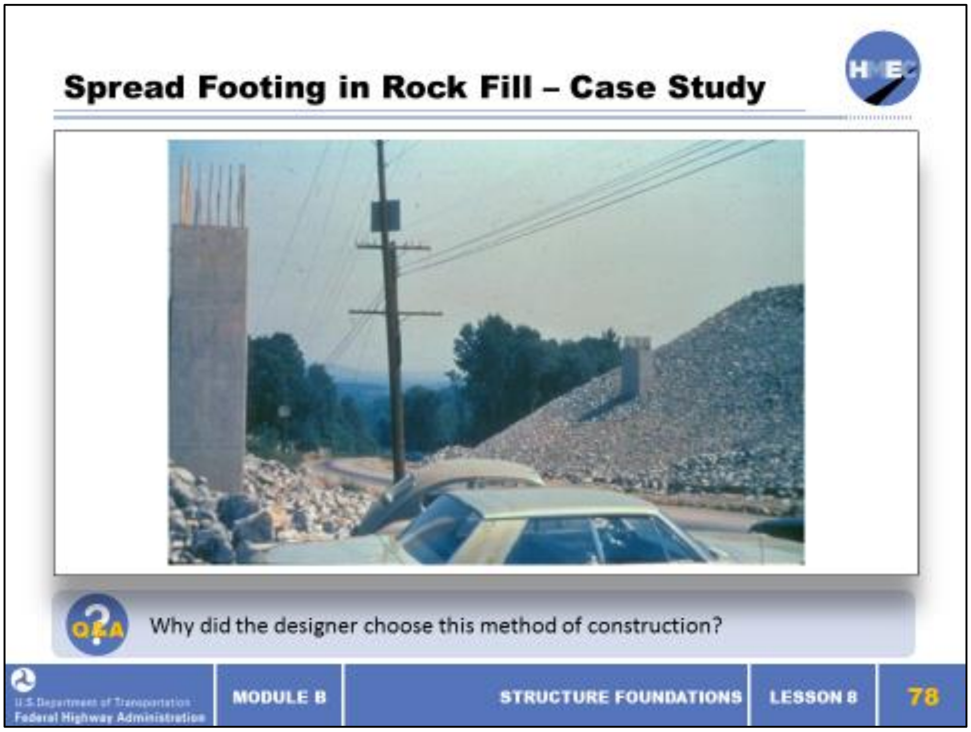
When the footing is cast directly on rock, the rock surface should be free of loose material, cleaned and either leveled, stepped, or roughened as necessary. Any visible seams are cleaned out of loose material and filled with lean concrete, mortar, or grout before the footing is cast.

Slide 77



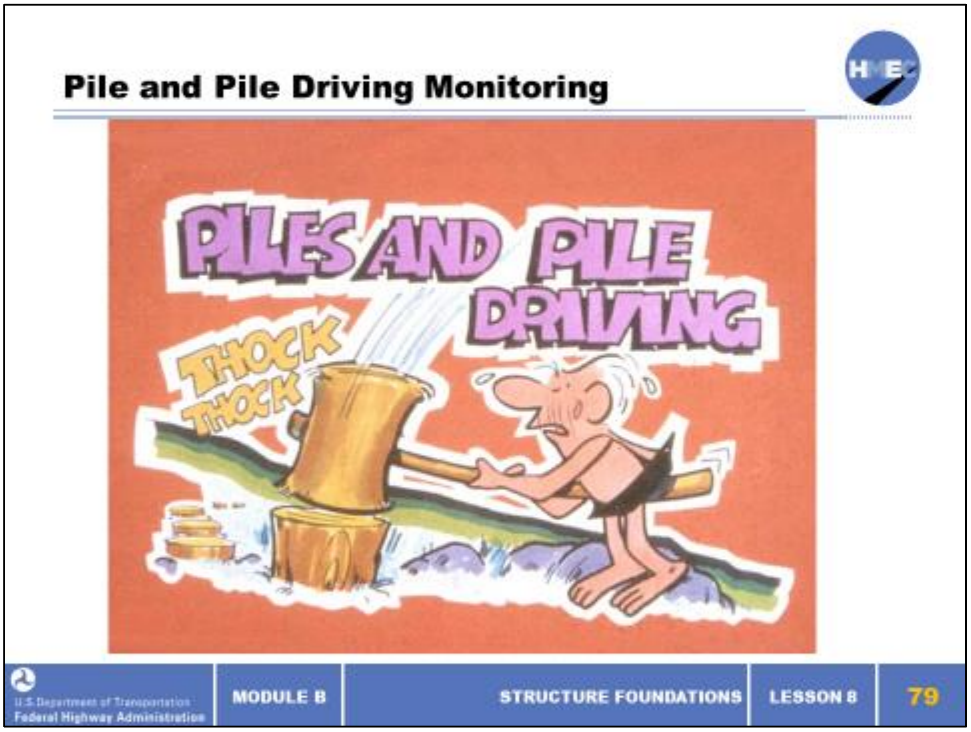
If boulders or ledges of rock are found at the bottom of the excavation or are protruding into the excavation's base, they are usually removed with care to a minimum depth of six inches below the bottom of the footing, and the over-excavation space is backfilled with a compacted coarse aggregate, select granular fill, or lean concrete depending on the loading requirement. Blasting can disturb the rock to remain in place and must be performed carefully.

Slide 78



The piers columns shown were constructed in the manner shown (allowance for extension) because a previous spread footing at a similar site settled differentially and caused problems with the beam seat. The 24-ft. wide spread footing was cast within the 1:1 end slope of the rock fill and the 24 ft. difference in the embankment height over opposite ends of the footing caused differential settlement and tilting of the column. The designer's solution was to construct the pier columns in stages.

Slide 79



If you have ever raised tomatoes, think about driving tomato stakes into the ground. If you have a heavy stake, a steel fence post for instance, you can drive it easily with an 8 lb. maul, but try using a 16 oz. framing hammer. If your stake is a slender wooden dowel, what happens if you hit it with an 8 lb. maul? It’s not good, but the 16 oz. hammer works fine. Have you noticed that as you drive a flexible stake into the ground, it vibrates and flexes with each blow? The more flexible the stake is, the harder it is to drive. For any stake, if the ground is soft it is easier to drive the stake.

Those same concepts relate to hammer size and energy, pile impedance, and soil resistance.

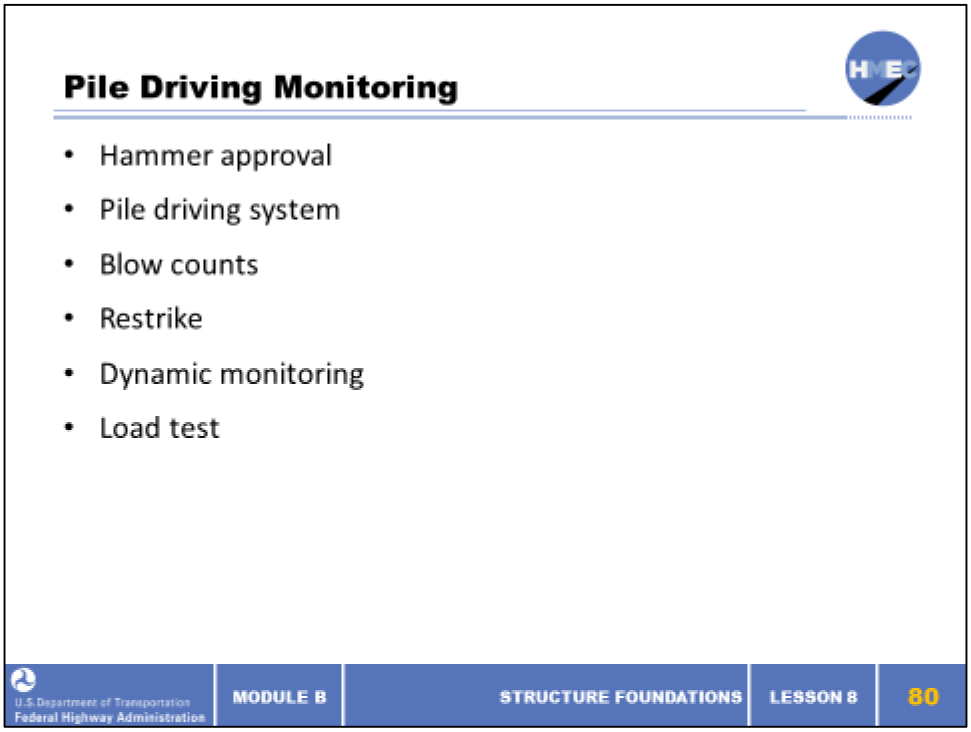
Construction monitoring of the pile cap will be similar to a spread footing so we will look only at monitoring piles and pile driving.

____

____

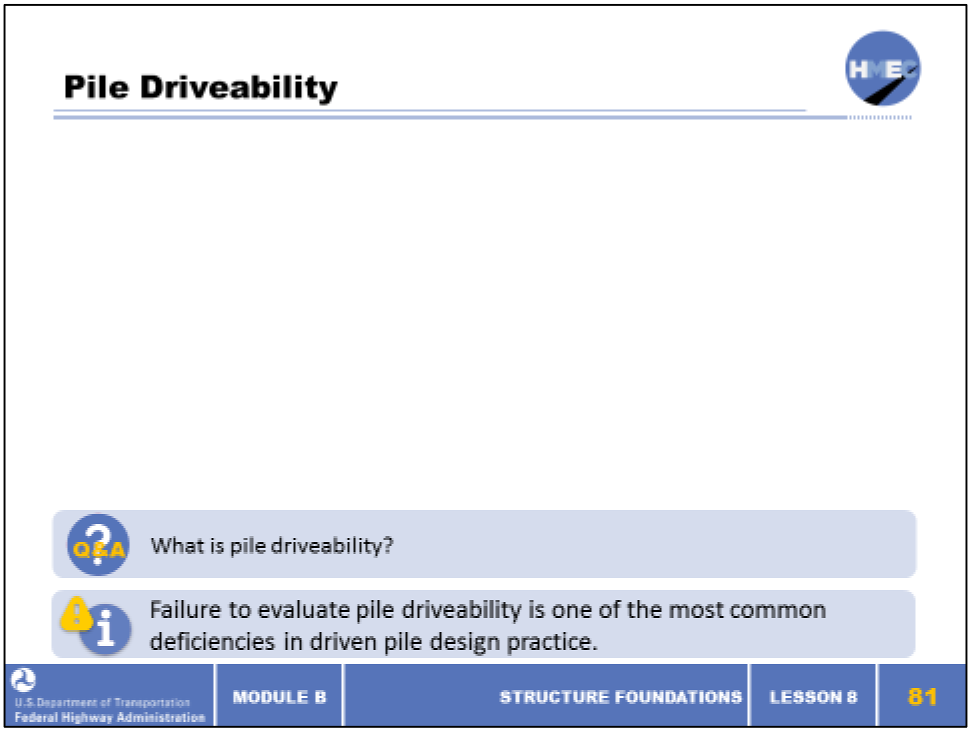
____

Slide 80

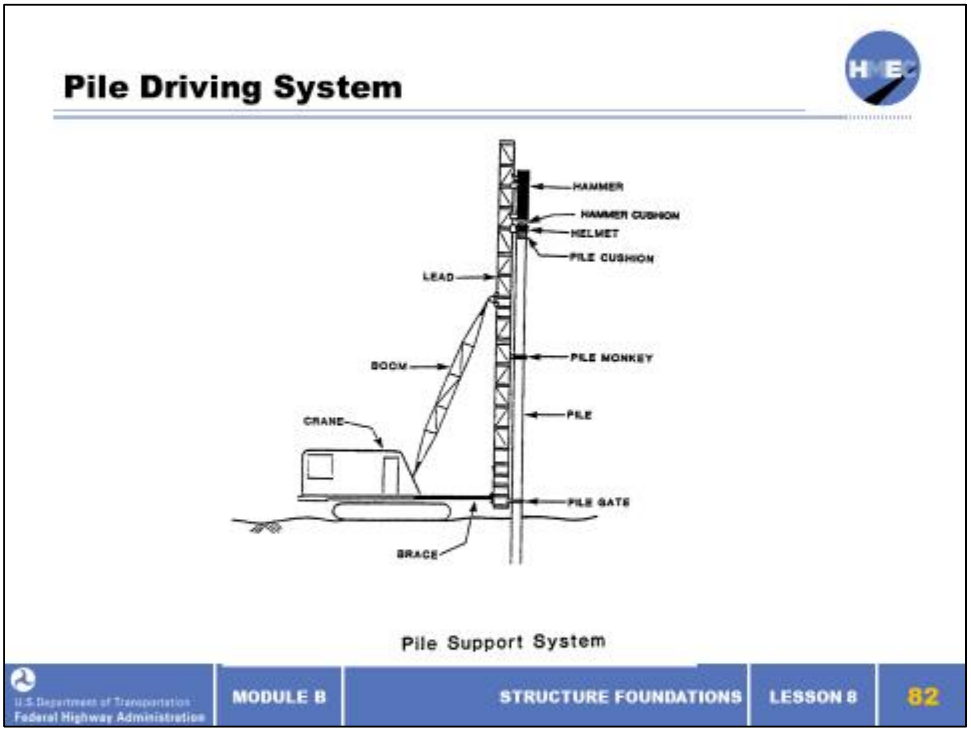


The evaluation of pile driving actually begins in design. The designer must perform an evaluation of the pile foundation system, including an assumed pile driving hammer to evaluate the driveability of the selected piling.

Slide 81

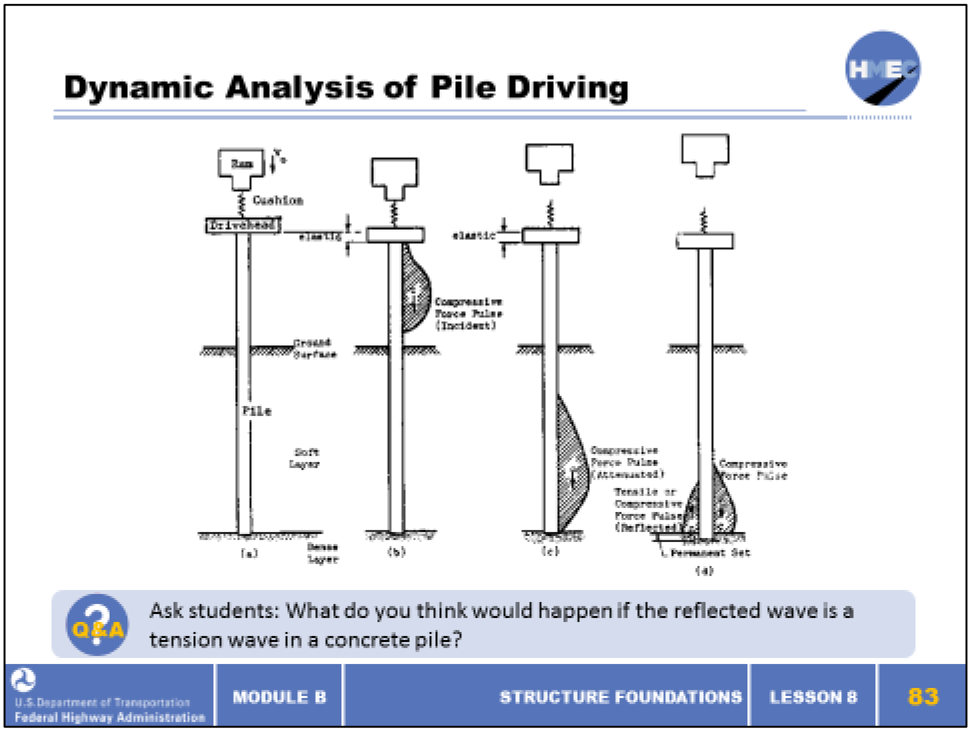

Pile Driveability
HMEC
Q&A
What is pile driveability?
Failure to evaluate pile driveability is one of the most common deficiencies in driven pile design practice.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
81

Slide 82



A typical pile driving system consists of several components, all of which have a key role in the installation of the pile.

Slide 83



When the hammer impacts the pile head a force wave is generated that travels down the pile, the amplitude of the wave decays due to system dampening (soil resistance and pile impedance). When the force wave reaches the toe of the pile it acts to pull the pile into the ground. A wave is reflected back up the pile and an amount of penetration or set remains.

The primary components of the system that are considered in the analysis include hammer data from the manufacturer, hammer cushion, pile cushion, pile properties (material, size shape and length), soil properties (resistance based on our soil tests), soil quake, and dampening properties.

____

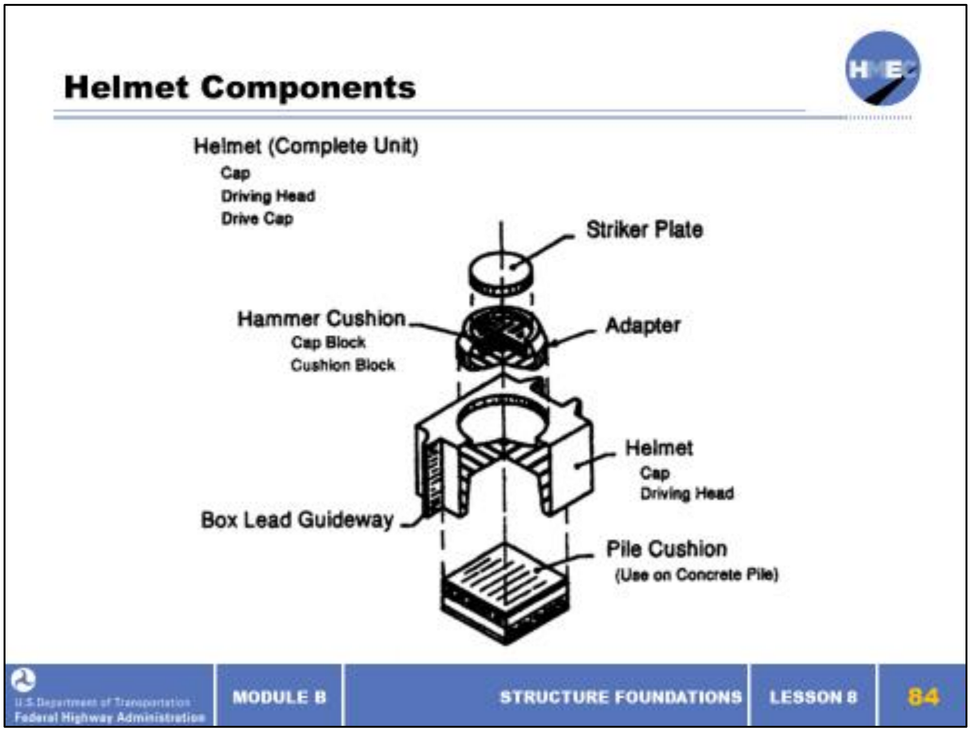
____

____

____

____

Slide 84



The hammer cushion needs to be the material recommended by the manufacturer. The pile cushion is typically multiple layers of ¾ in. plywood. The cushions shown, the condition of the helmet, and the fit of the helmet over the pile all influence the energy transmitted by the hammer to the pile. They are key influences on driveability.

Note that the diagram of the helmet is for nomenclature only. Various sizes and shapes are available to drive H, pipe, concrete (shown), and timber piles. A system of inserts or adapters is used inside the helmet to change from size to size and shape to shape.

____

____

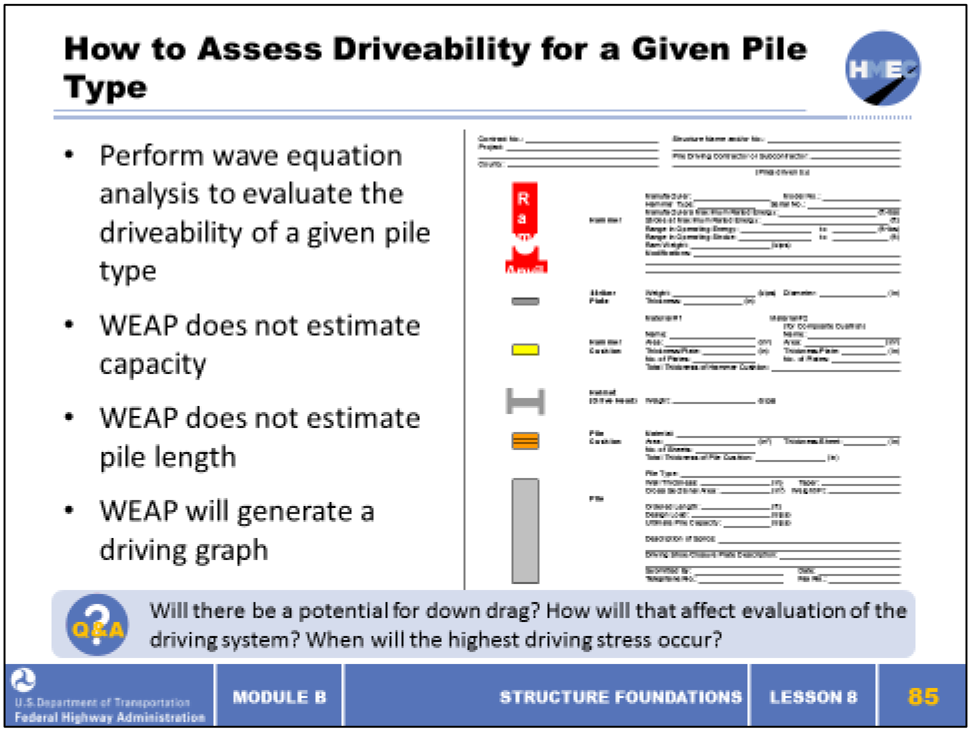
____

____

____

____

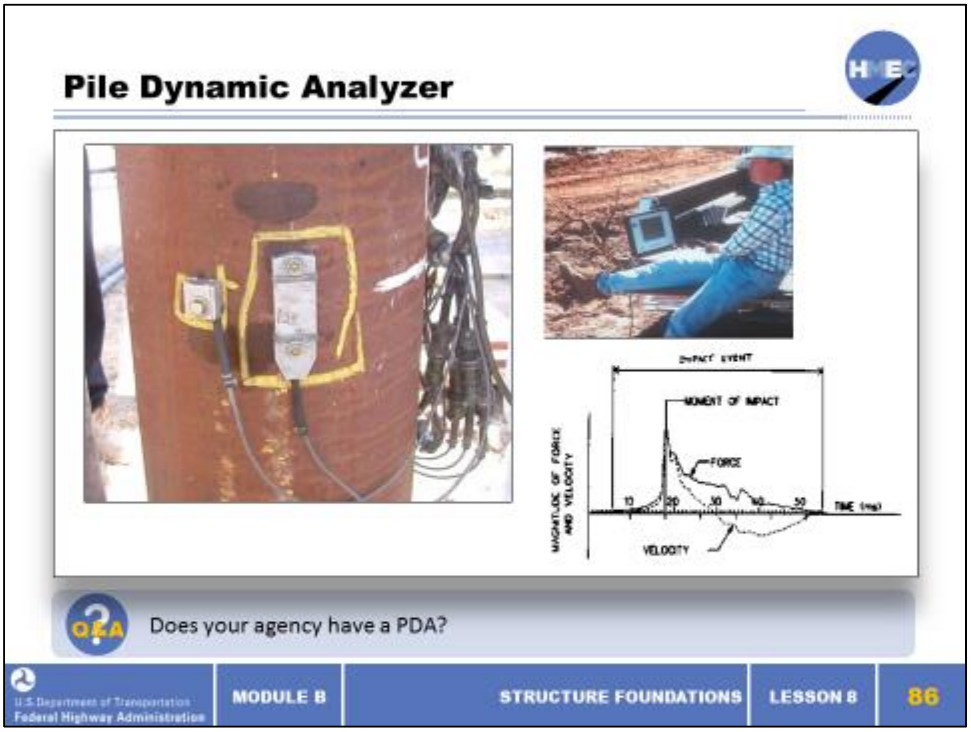
Slide 85



The WEAP needs to be performed during design to check driveability of the proposed design, and again prior to construction to evaluate the contractor's proposed pile driving system. Generally the agency or their designer performs the first analysis and sometimes the second analysis based on a submittal from the contractor. Alternatively, the contractor's engineer may perform the second analysis, and submit those results to the agency.

The WEAP does not estimate pile capacity or pile length—those are both input into the program. The WEAP will generate a driving graph, a plot of blows/foot at a given hammer drop and a given pile length and pile capacity. This can be used by the inspector to determine if the design capacity has been achieved.

Slide 86



Dynamic test methods use measurements of strain and acceleration taken near the pile head as a pile is driven or re-struck with a pile driving hammer. These dynamic measurements can be used to evaluate the performance of the pile driving system, calculate pile installation stresses, assess pile integrity, and estimate static pile capacity. Dynamic test results can be further evaluated by using signal matching techniques to determine the relative distribution of soil resistance along the pile, as well as representative dynamic (quake and dampening) soil properties for use in wave equation analyses.

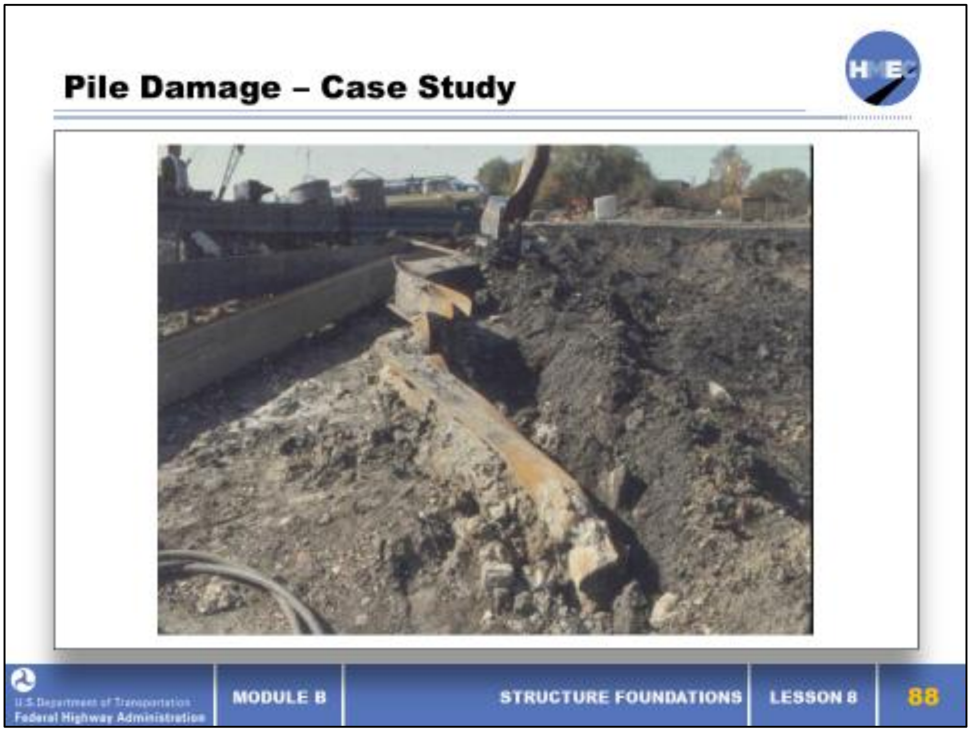
A re-strike is performed after initial pile driving. After the pile has been allowed to set for a predetermined time, the pile driving hammer is used to re-strike the pile a few blows. The PDA monitors the readouts from the instrumentation and can assess any increase or decrease in pile capacity. Depending on the soils set-up (increase in capacity) or relaxation (loss of capacity) may have occurred.

Note that the restrike can also be performed without use of the PDA and the set-up or relaxation evaluated simply by a dynamic formula.

Slide 87

The load test can be conducted during design or during construction. The results can be used to determine pile capacity and resistance factors for LRFD. Getting the most benefit from a load test requires soil borings and lab tests at the load test site, in addition to full instrumentation of the test pile.

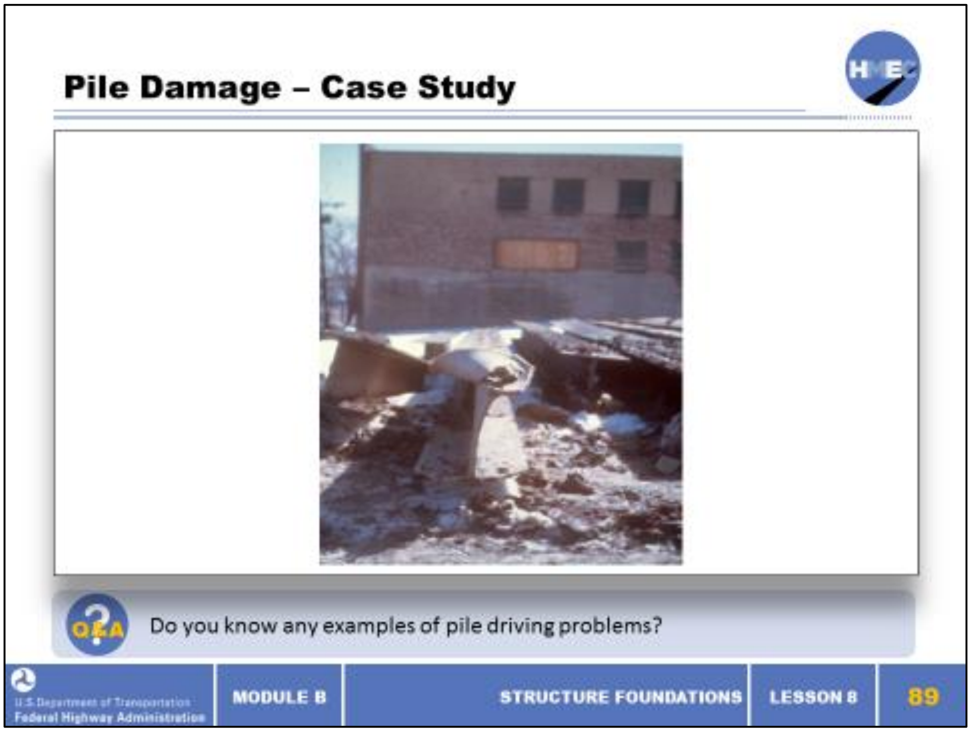
Slide 88



The hammer used by the contractor was a large diesel hammer that was required for larger piles in the river piers. The pile driving was monitored using a dynamic formula. The piles were driven into rock and just as capacity was reached, it would suddenly drop off. This scenario continued for several iterations: drive to capacity, then capacity drop off. The pile was pulled and it looked much like an accordion. The drop off in capacity occurred each time the large stress in the pile caused the pile section to collapse. The large energy from the hammer was not attenuated significantly through the weak soft overburden, therefore all the energy was being transmitted to the pile at the rock surface. The pile could not penetrate the rock and large stresses built up in the pile and failed the pile.

A case history of pile damage. The worst problem is when damage occurs after the pile is below ground. If the damage is detected, the pile is usually pulled and a new pile driven. If the damage is not detected, the problem may not become evident until structural loads are applied to the pile. The bottom line is that highway agencies need to consider pile overstress caused by the driving operation.

Slide 89



: Piling was driven into limestone with solution channels. The pile was driven past the planned tip elevation and did not attain the required bearing. High bearing was expected since the pile was driven into sound limestone. Other piles in the footing had attained the required bearing at the planned tip elevation. The pile was pulled and it was found that the flanges had turned and were flat along the web of the H-pile. The pile had been driven into a vertical joint in the limestone, which flattened the flanges and formed a “knife” edge that continued down the joint without an increase in bearing.

____

____

____

____

____

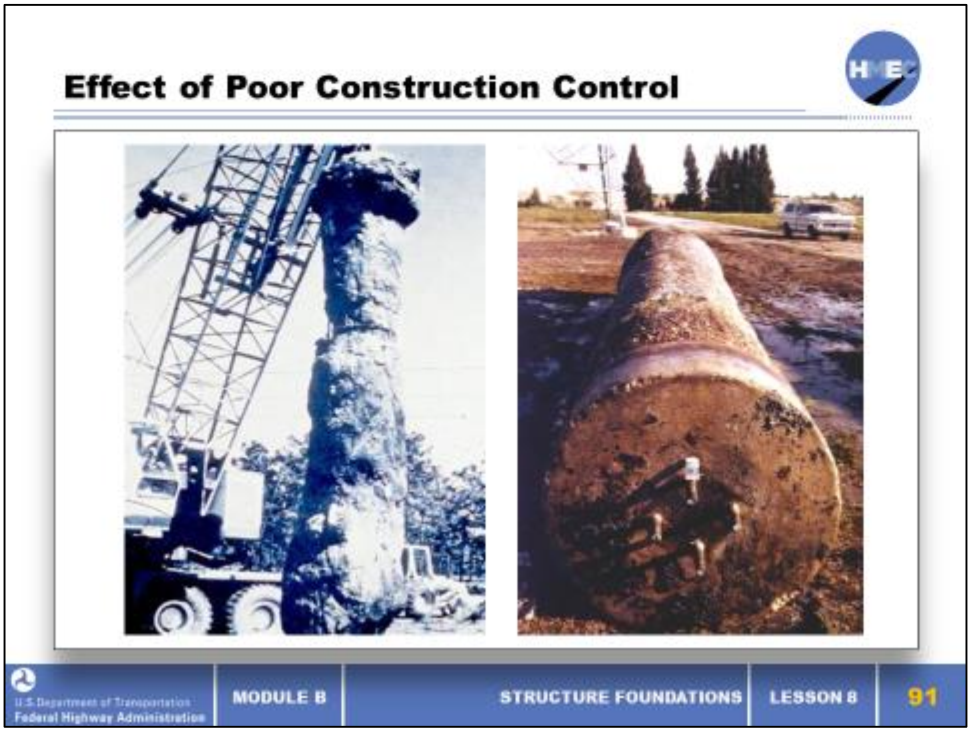
____

____

Slide 90

Drilled shafts are manufactured on site and quality assurance and integrity testing are important to ensure a quality product.

Slide 91



The shaft on the left was constructed in an uncontrolled bentonite slurry whose unit weight was about 100 pcf (<75 is recommended) and sand content was above 20% (< or = 4% is recommended). The slurry was not agitated for several hours. (a limit of 4 hours is recommended) Note the exposed rebar at the bottom (trapped sediment) and the exposed rebar about one-third of the way down from the top. This coincided with the level of the water table. Apparently, as the sediments being pushed up by the concrete column lost buoyancy at this level the concrete column broke through, trapping the sediments there.

The shaft on the right was constructed using bentonite slurry whose properties fell within the ranges recommended here. No defects and no malformations. We should expect to produce shafts like this if specifications are written properly, inspection is competent and contractors are qualified.

Potential defects for drilled shaft might include necking, bulbing, soft-bottom, voids or soil intrusions, poor quality concrete, debonding, lack of concrete cover over reinforcement, and honey-combing. All of which can be seen in the shaft on the left.

____

____

____

____

____

____

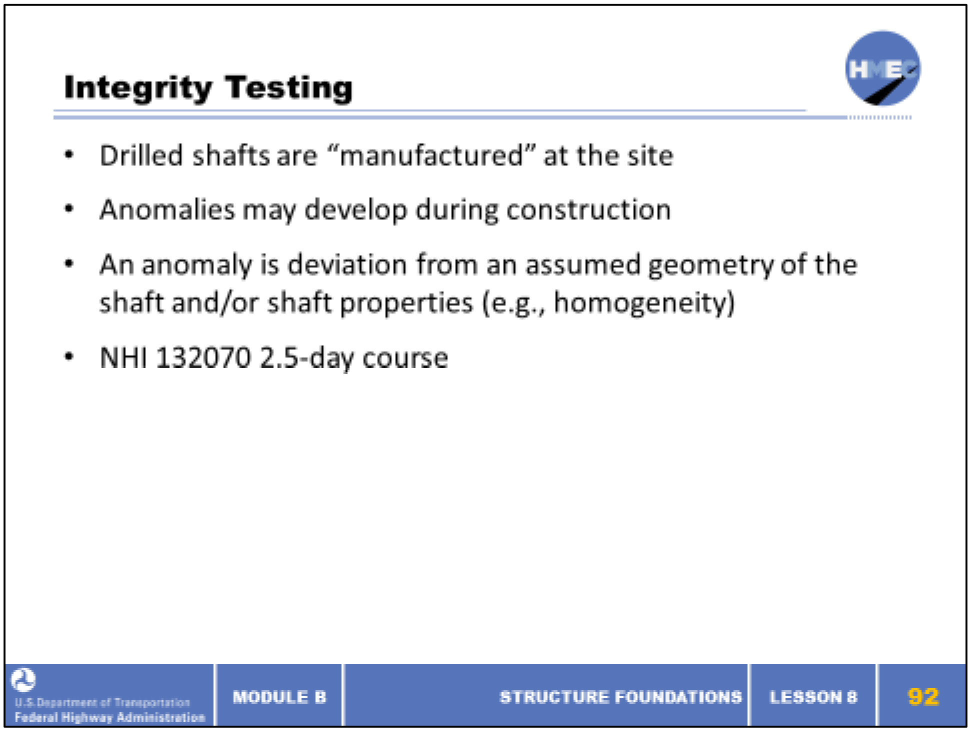
____

____

____

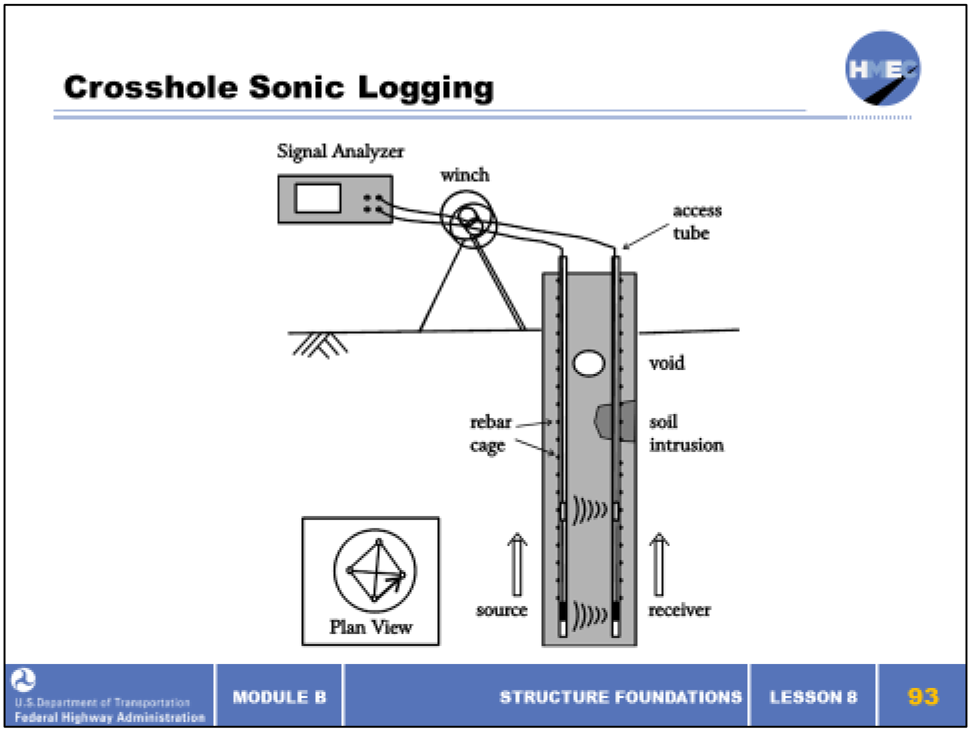
____

Slide 92



Integrity testing complements good inspection by identifying anomalies that may be present in the shaft. Inspection and testing are considered together to determine if further investigation is required. The NHI course 132070, Drilled Shaft Inspector's Qualification, is a two and a half-day course that provides additional details

Slide 93



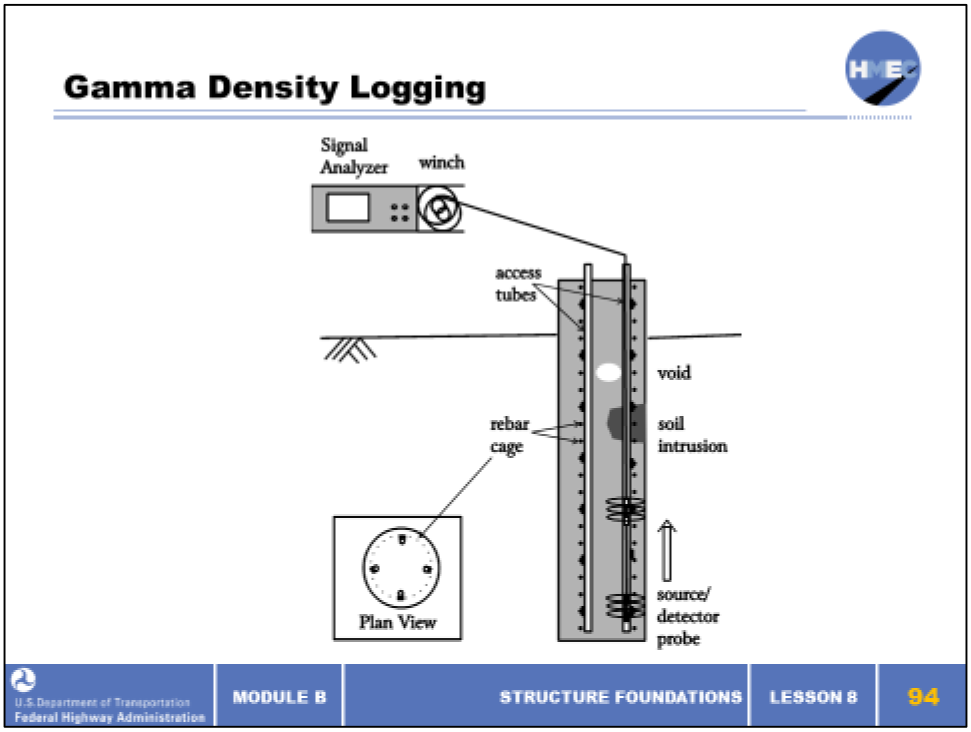
The correct response to that question is to ensure quality construction techniques by having a well-qualified inspector on site. However, provisions for post integrity inspection should also be included in the design. Cross-hole sonic logging (CSL) is a common method used to assess the quality of the shaft after construction. CSL tubes are placed around the perimeter of the shaft, typically inside the reinforcement cage with about one tube for each foot of diameter. After the concrete has hydrated for 2–3 days, ultrasonic transmitters and receivers are lowered down various pairs of tubes and the travel time and amplitude of the signals are recorded for each pair of tubes.

Note that the CSL testing can only “see” on the lines between tubes and therefore inside the reinforcing cage. Note diagram in the lower left.

Steel tubes are typically used because of problems with PVC tubes debonding.

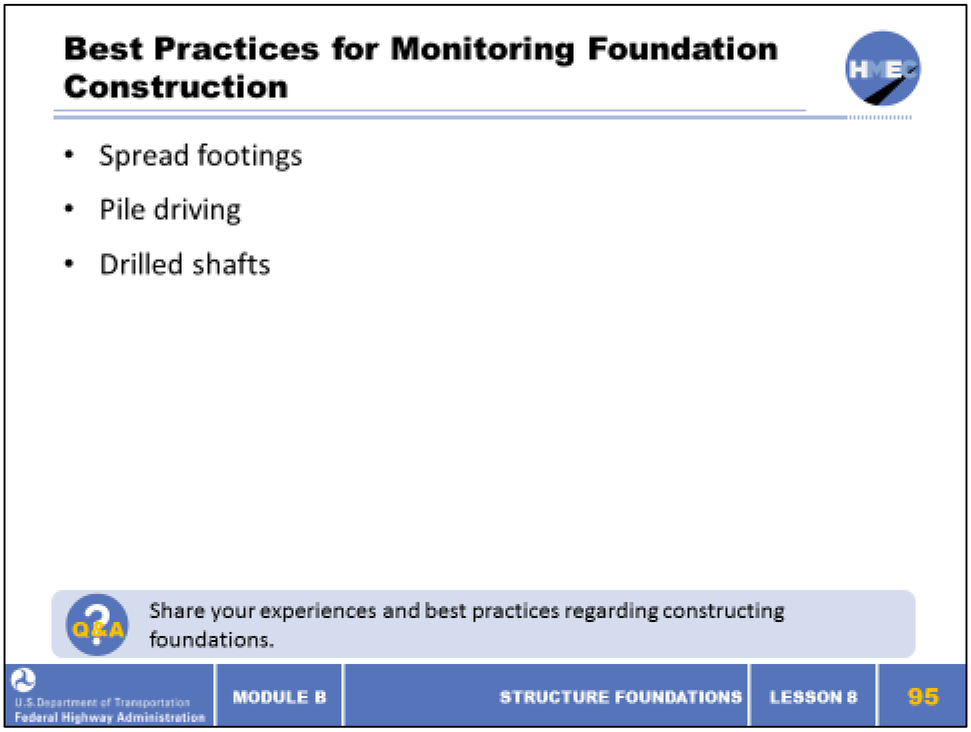
A drop of > 20% in the velocity is an indication of an anomaly and may require further investigation. Such a reading coupled with an inspector’s note of an issue during concreting or a problem indicated by the concrete volume plot adds credence that the anomaly may well be a defect.

Slide 94



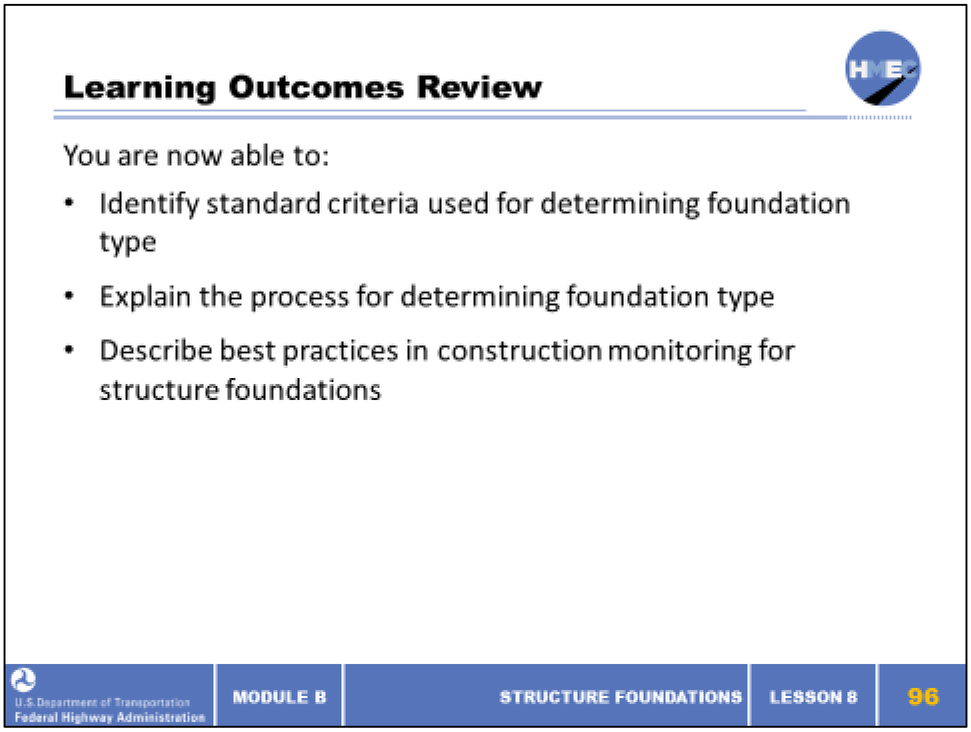
This tests uses gamma radiation to detect potential defects within about a 5–6 in. radius around the tube. Since debonding does not adversely affect gamma-gamma, PVC tubes are typically used. The gamma-gamma test provides results outside the reinforcing cage.

Slide 95



Share your experiences to help us identify best practices when constructing foundations.

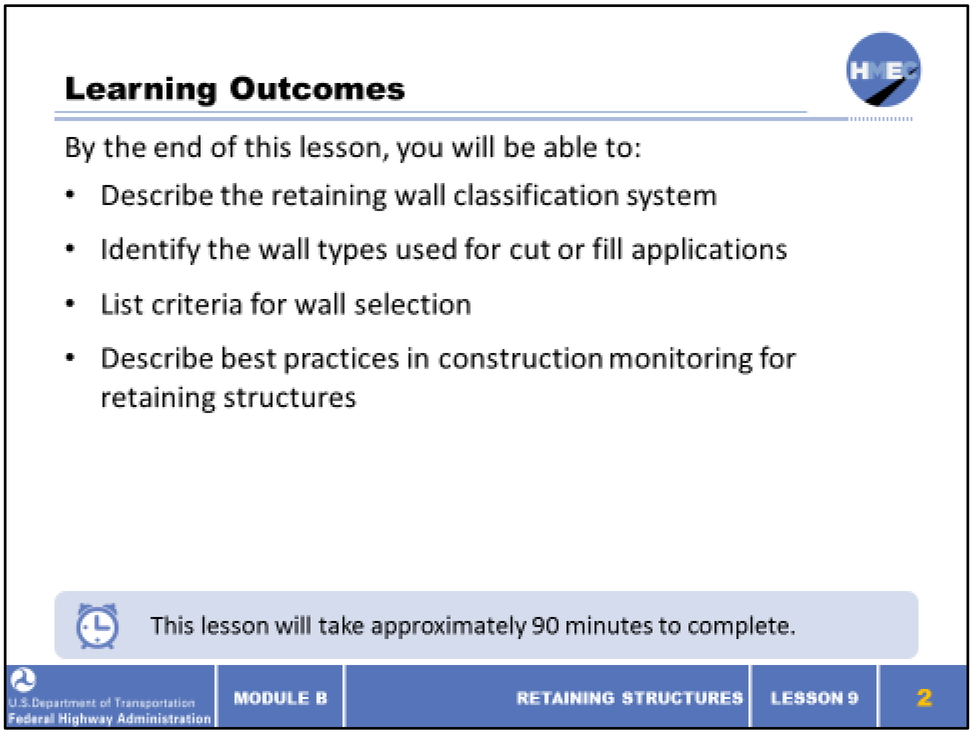
Slide 96


Learning Outcomes Review
HMEC
You are now able to:
• Identify standard criteria used for determining foundation type
• Explain the process for determining foundation type
• Describe best practices in construction monitoring for structure foundations
U.S. Department of Transportation
Federal Highway Administration
MODULE B
STRUCTURE FOUNDATIONS
LESSON 8
96

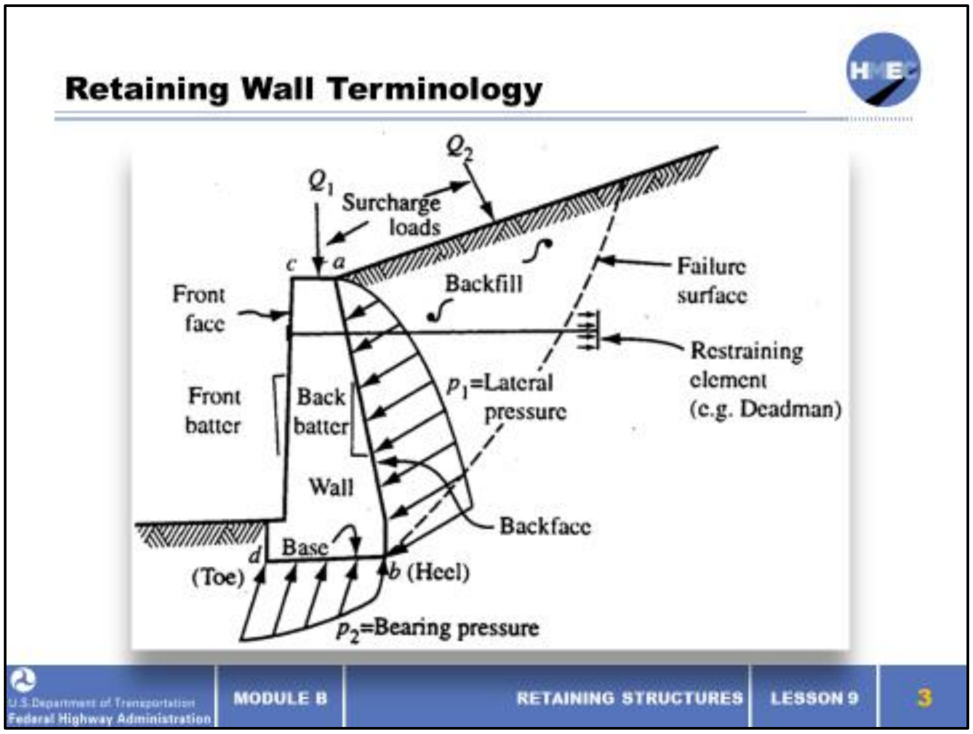
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 9: Retaining Structures
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2

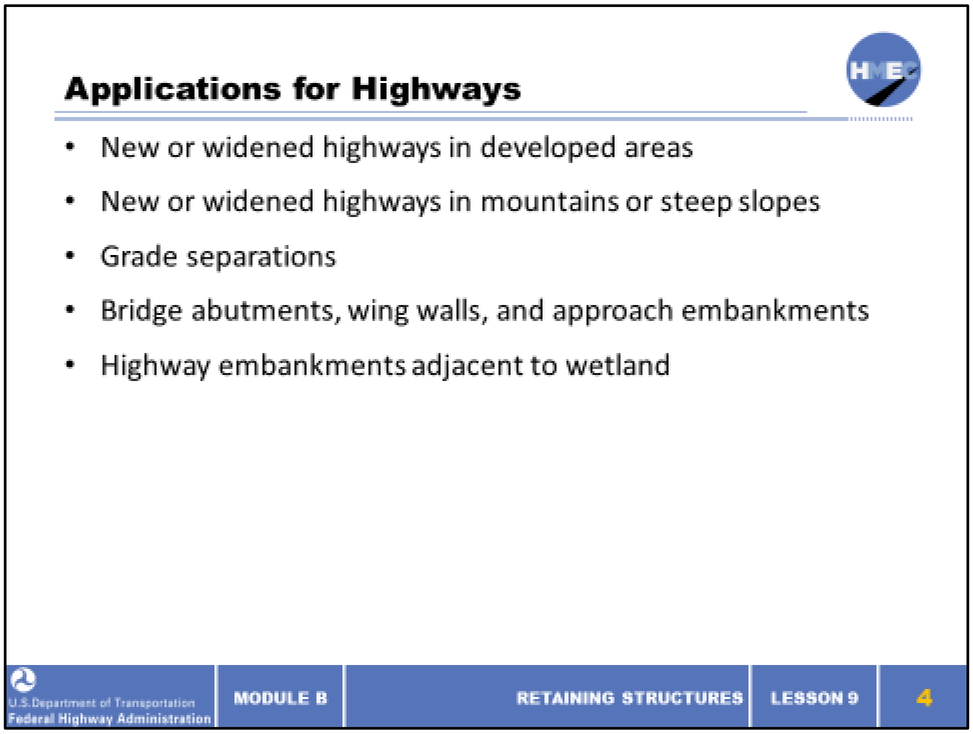

Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
Describe the retaining wall classification system
Identify the wall types used for cut or fill applications
List criteria for wall selection
Describe best practices in construction monitoring for retaining structures
This lesson will take approximately 90 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
RETAINING STRUCTURES
LESSON 9
2

Slide 3



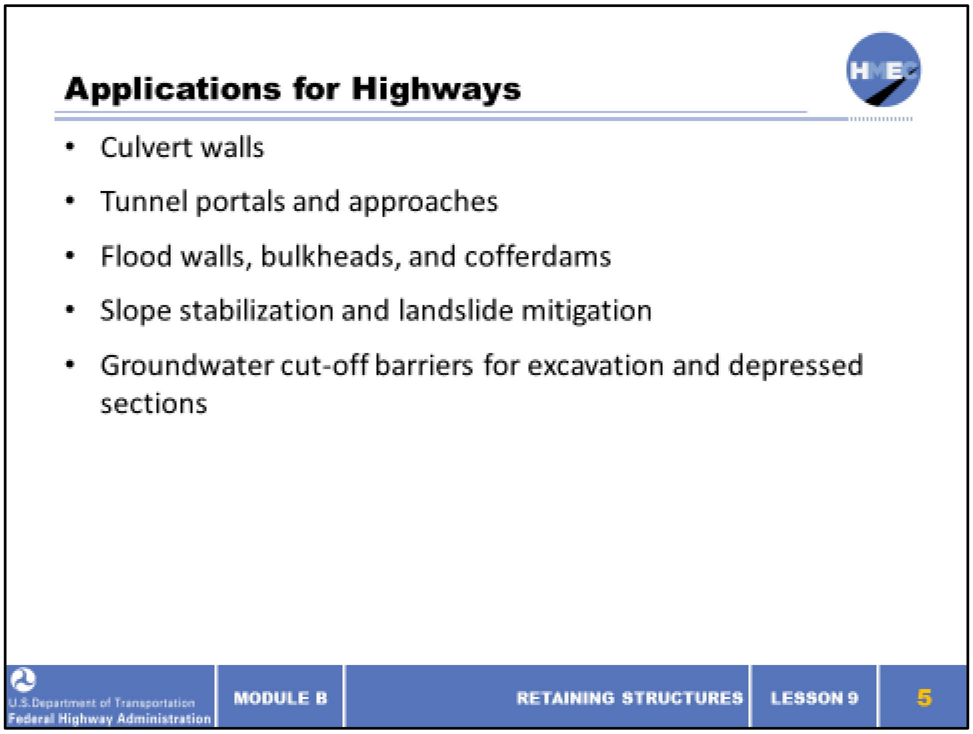
Earth retaining structures or systems are used to hold back earth and maintain a difference in the elevation of the ground surface, as shown on the screen. They take on many forms as we will discuss during this lesson. First, let's look at common retaining wall terminology

Slide 4



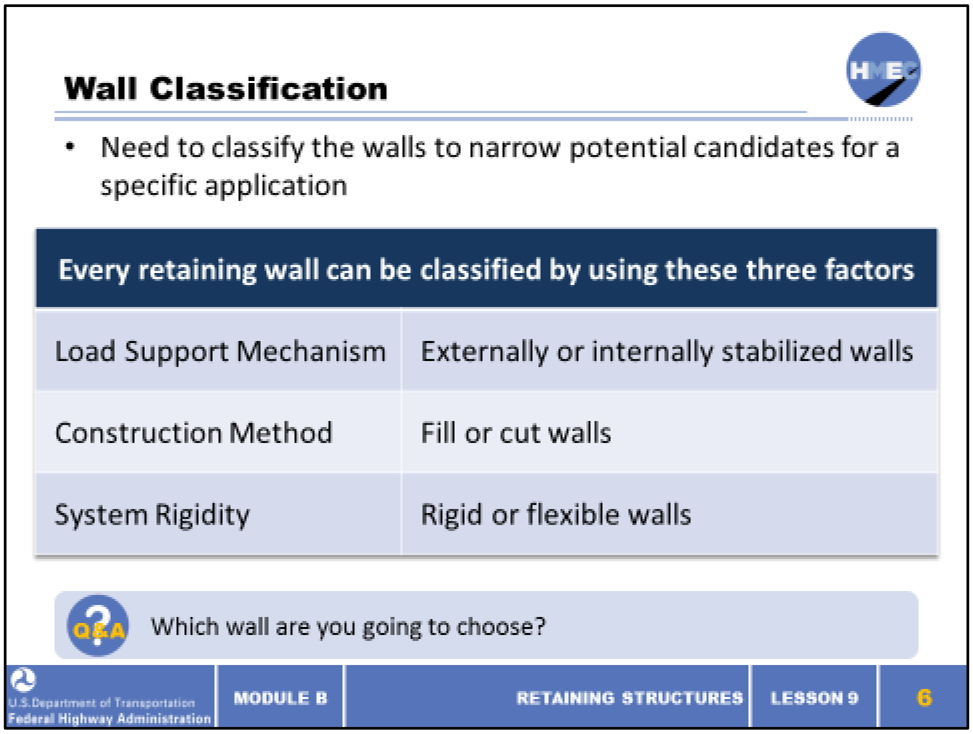
Retaining walls are typically required when there is not adequate right-of-way to build a slope. A slope is less expensive initially and less expensive if repair becomes necessary, therefore, it is the preferred option if appropriate. However, in developed areas or in mountains, retaining walls may allow construction when a slope cannot be built. At grade separations, retaining walls can allow a shorter, less expensive bridge. Bridge abutments and wing walls, which must support earth fills, are also designed as retaining walls, supporting the approach fill at the end of the bridge. Retaining walls are also essential when a slope cannot be used because of adjacent wetlands, rivers, or lakes.

Slide 5



Culvert walls and culvert wing walls are a common use of retaining walls. Retaining walls are required at tunnel entrances to support the slope at the sides of the portals. Flood walls, bulkheads, and cofferdams not only support-retained soil, but also must hold back water.

Slide 6



Three factors can be used to classify any wall, and that classification can help us select an appropriate wall for our site. Walls are either externally or internally supported, and we will show you some graphics in a few slides to help you understand what that means. Walls are constructed as either cut or fill walls. It might be useful to think of the construction method as top down or bottom up, since some walls built with fill (such as an MSE wall) might be used in a cut section. Walls are also classified as either rigid or flexible, which is an important factor in design.

____

____

____

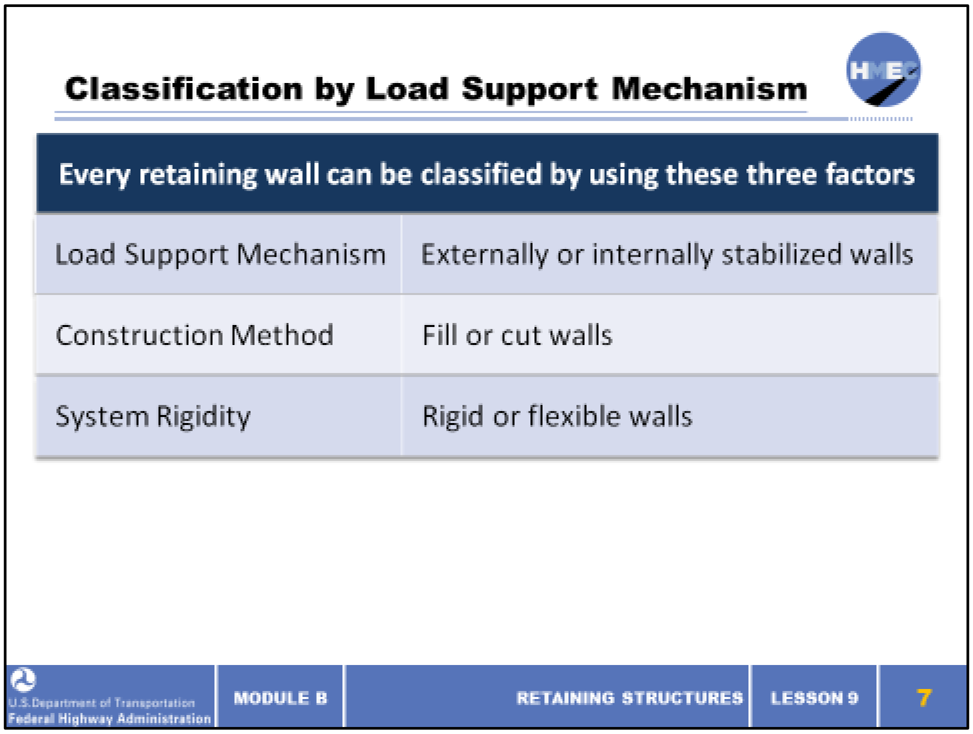
____

____

____

____

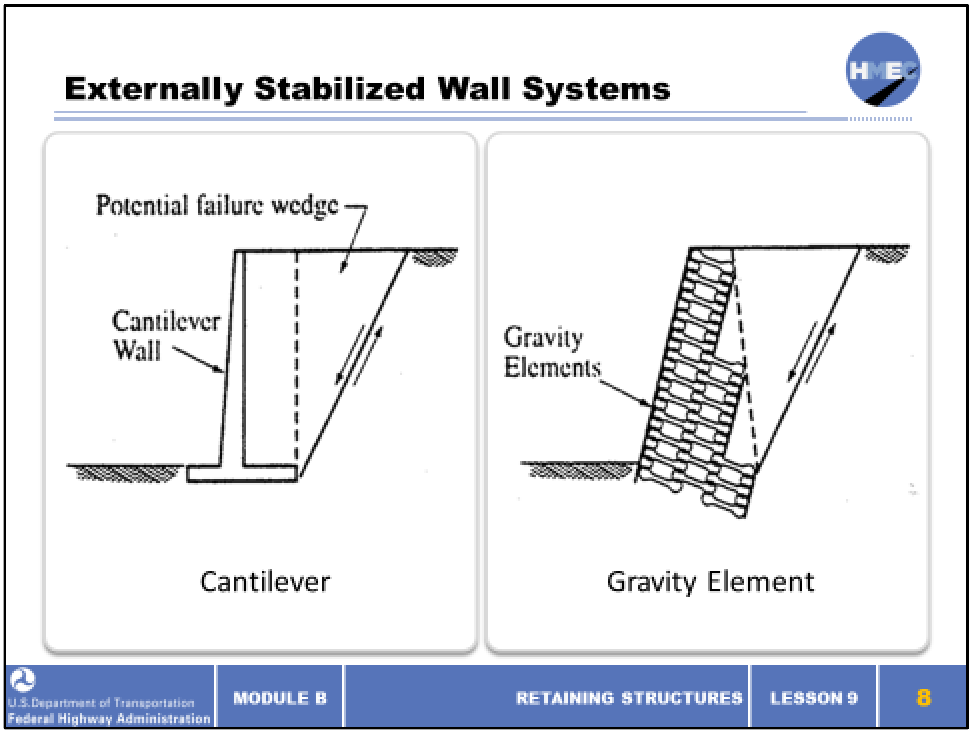
Slide 7



An externally stabilized system uses an external structural wall against which stabilizing forces are mobilized.

An internally stabilized system involves reinforcements installed within the backfill or retained soil mass and extending beyond the potential failure plane.

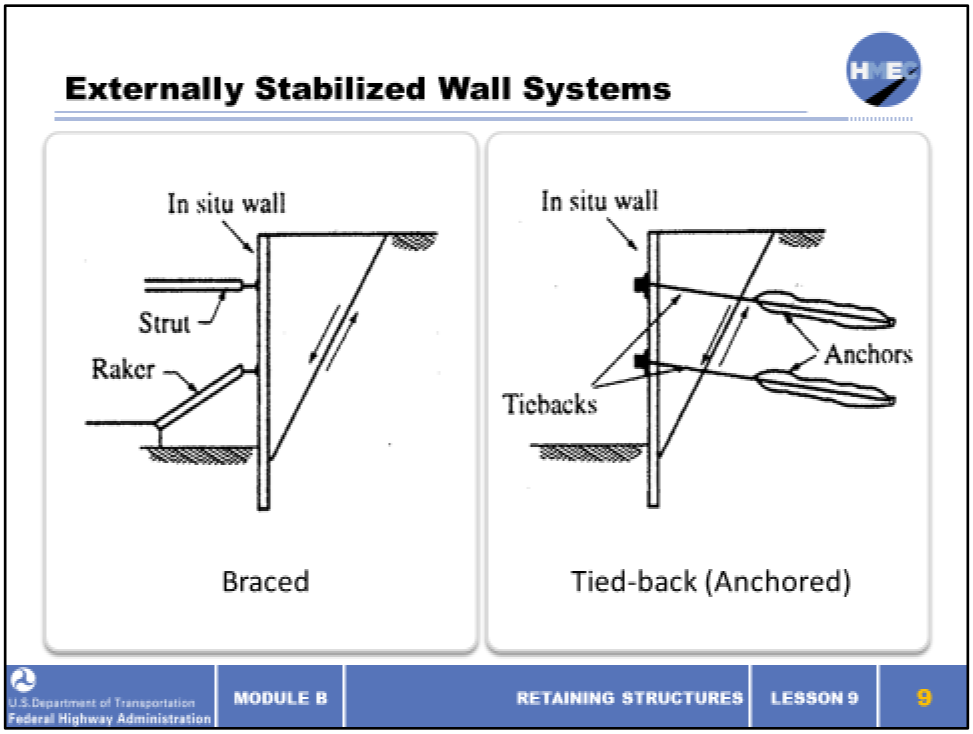
Slide 8



Virtually all traditional types of walls may be regarded as externally stabilized systems. Gravity walls, in the form of cantilever structures or gravity elements (e.g., bins, cribs and gabions), support the soil and, through their weight and stiffness, resist sliding, overturning, and shear.

The gravity element may consist of reinforced concrete or bins, cribs, or wire baskets filled with soil or rock.

Slide 9



Bracing systems, such as cross-lot struts and rakers, provide temporary support for in-situ structural and chemically stabilized walls. Ground anchors provide support through their pullout capacity in stable soils outside of the zone of potential failure. It may seem that the anchored wall is internally stabilized, since anchors provide resistance from the soil. However, remember that an externally stabilized system uses an external structural wall against which stabilizing forces are mobilized. The anchored wall shown has that structural wall (labeled in-situ wall in the sketch), which the tiebacks are using to mobilize the stabilizing forces.

____

____

____

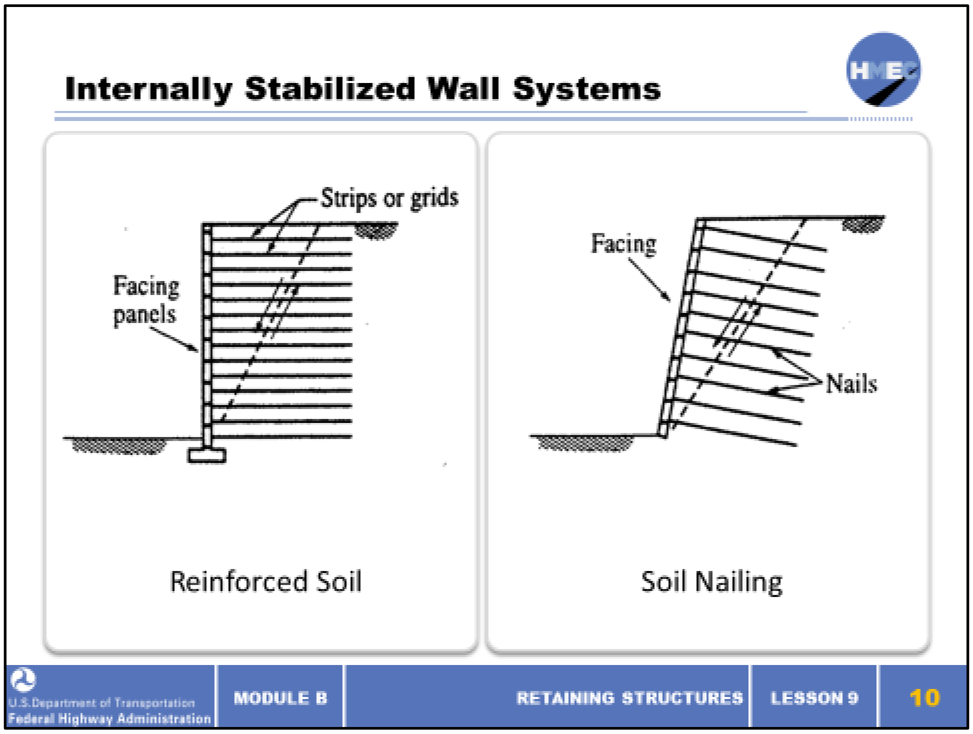
____

____

____

____

Slide 10



MSE walls and soil nail walls are two common internally stabilized wall systems. Although MSE walls have a facing, it is not structural but simply serves to prevent surface raveling. The facing for soil nail walls is structural.

____

____

____

____

____

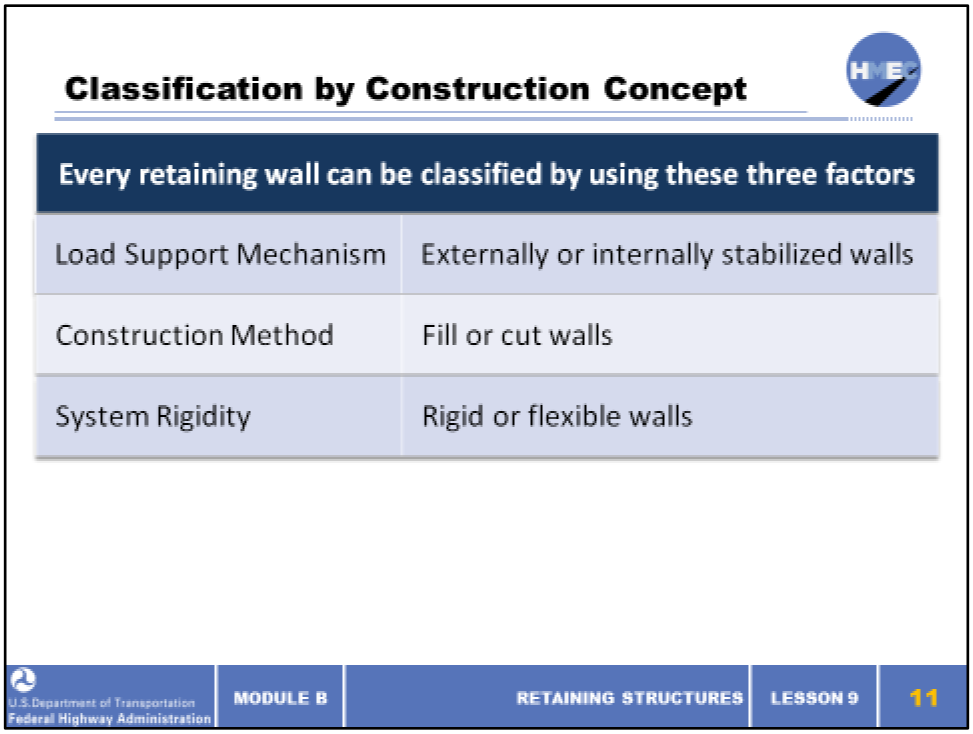
____

____

____

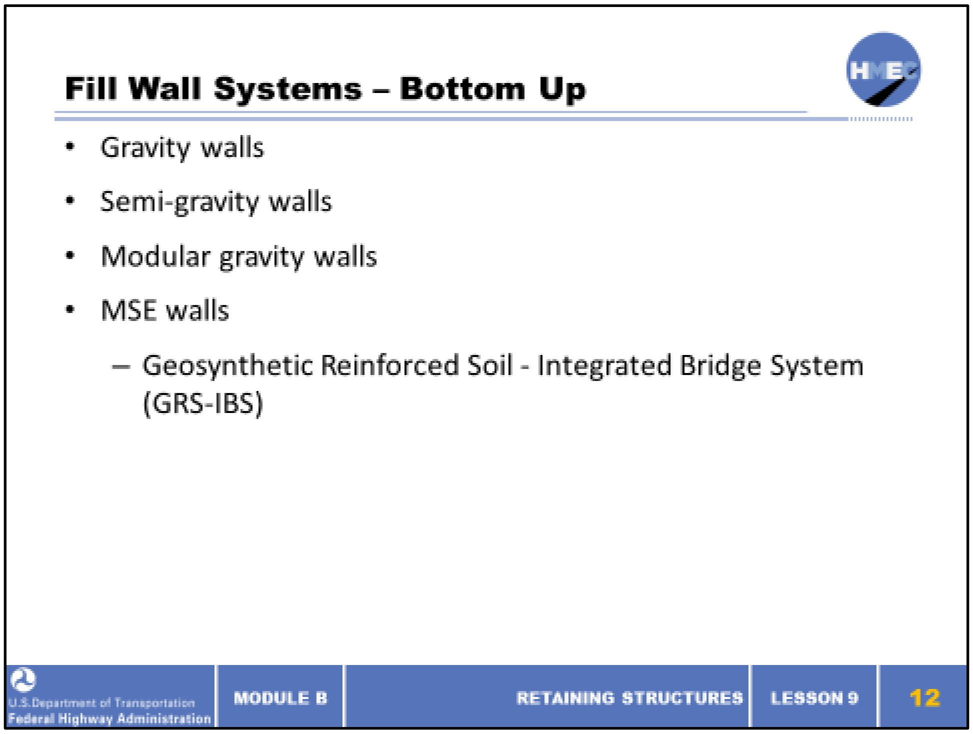
____

Slide 11



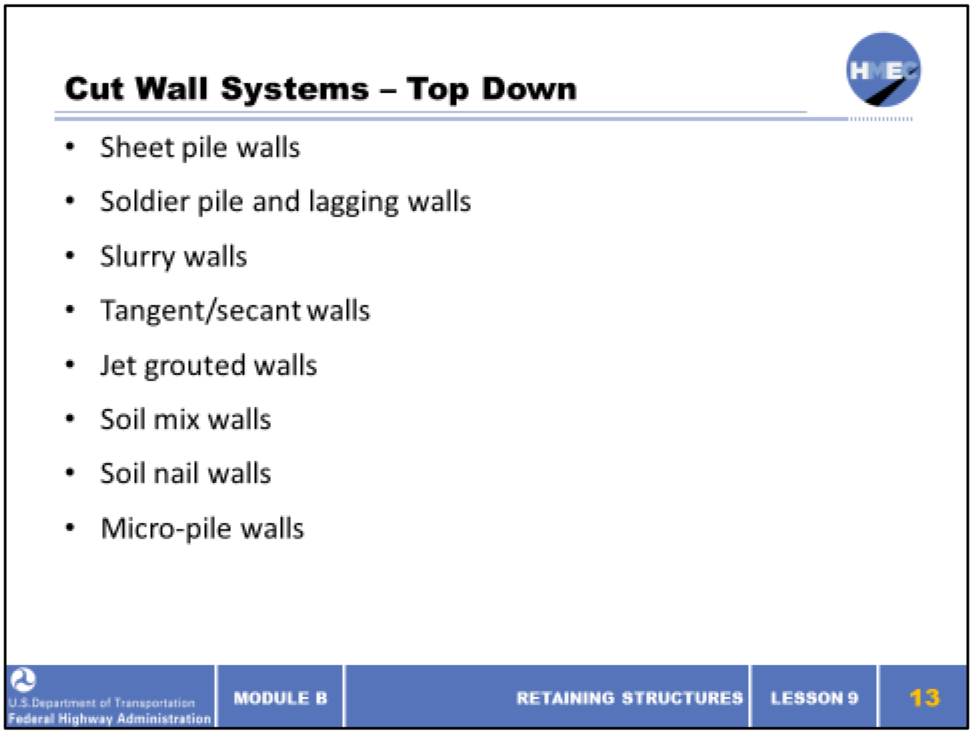
Bottom-up walls require that the area encompassing the footprint of the wall is available for construction of the wall and placement of backfill in order to begin construction of the wall. Top-down walls may be constructed in stages, with the excavation progressing as the wall is constructed.

Slide 12



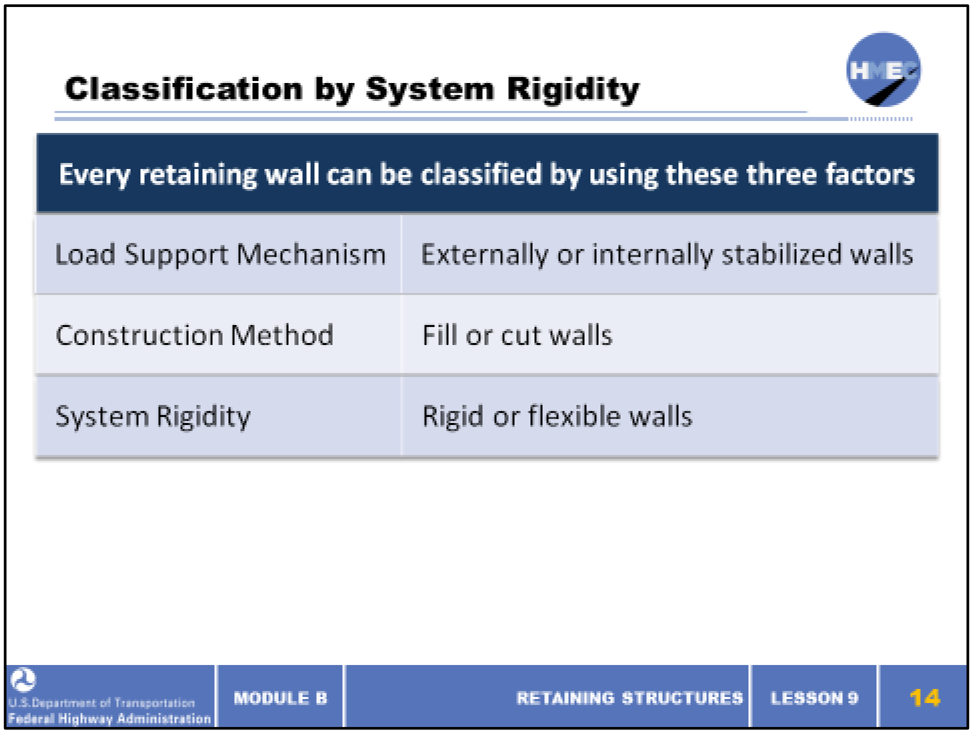
You can see each of these wall types require the construction to proceed from the bottom up.

Slide 13



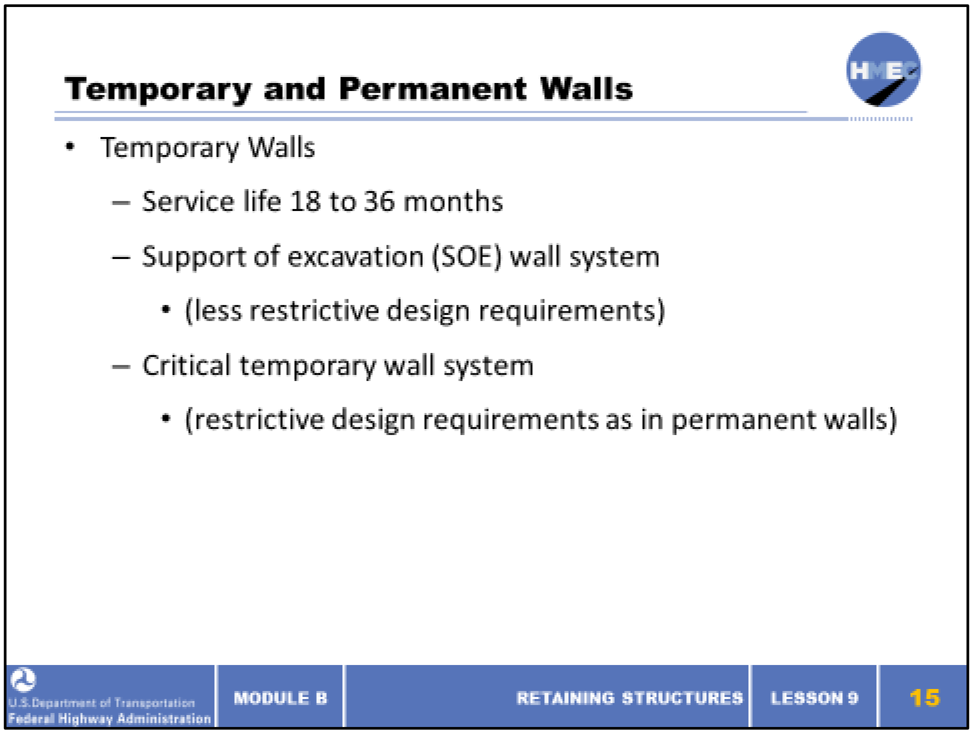
Sheet pile walls are often used by contractors to stabilize their excavations. Tangent/secant walls typically use drilled shafts. Soil nail walls are constructed in vertical stages.

Slide 14



Rigid wall systems rotate or translate as a unit when subjected to lateral earth pressure; only gravity walls are considered rigid. Flexible wall systems allow bending deformations, which redistribute the soil pressures acting on the wall. All systems except gravity walls can be considered flexible.

Slide 15



A temporary wall that needs to provide service for only 18–36 months does not need to meet the same durability requirements as a permanent wall designed for 75–100 years. The temporary wall may be designed to a lower standard, unless it is a critical wall as determined by the owner. An example of a critical temporary wall would be when lateral movement of the wall is restricted to prevent damage to an existing structure.

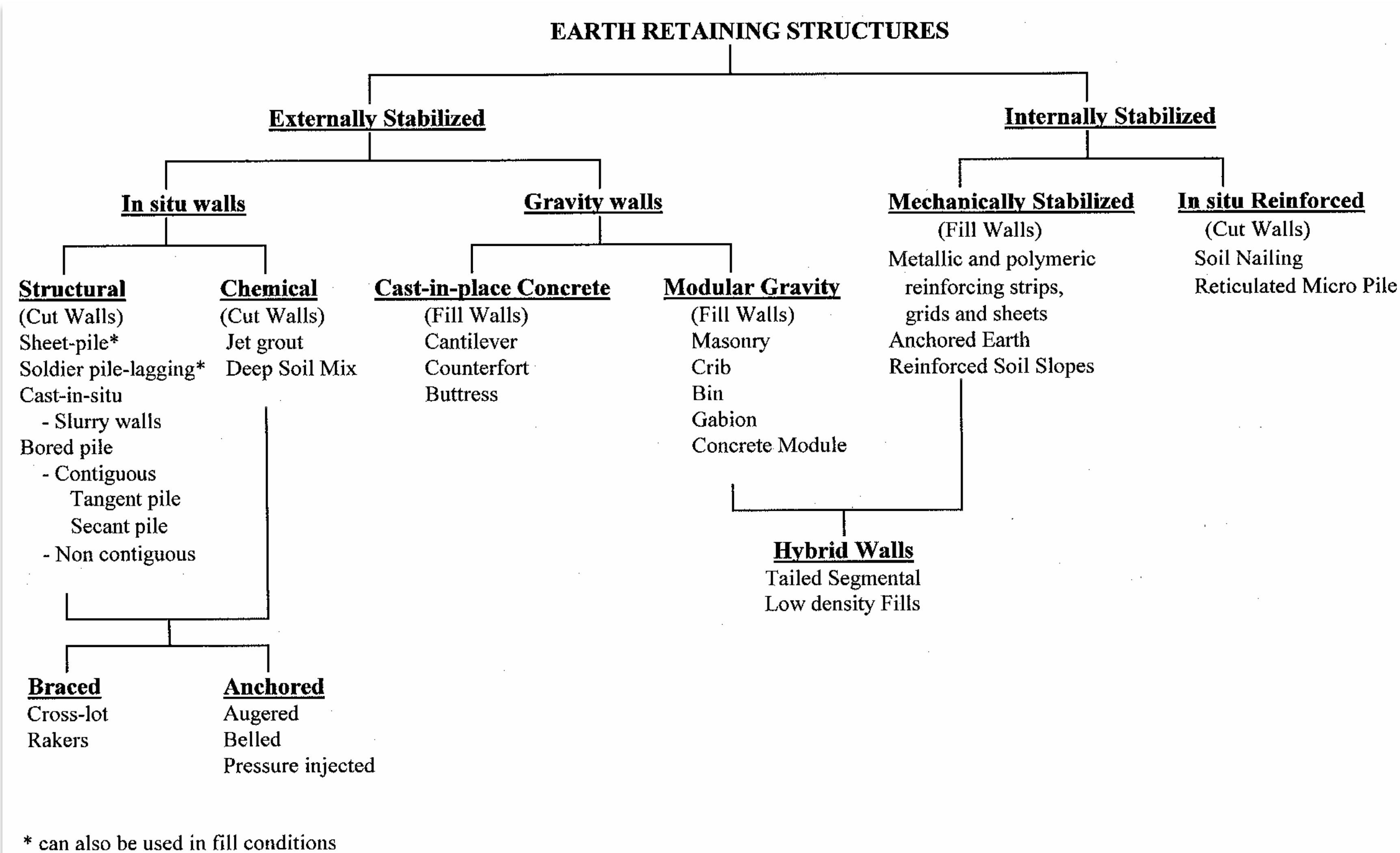
Slide 16

Temporary and Permanent Walls

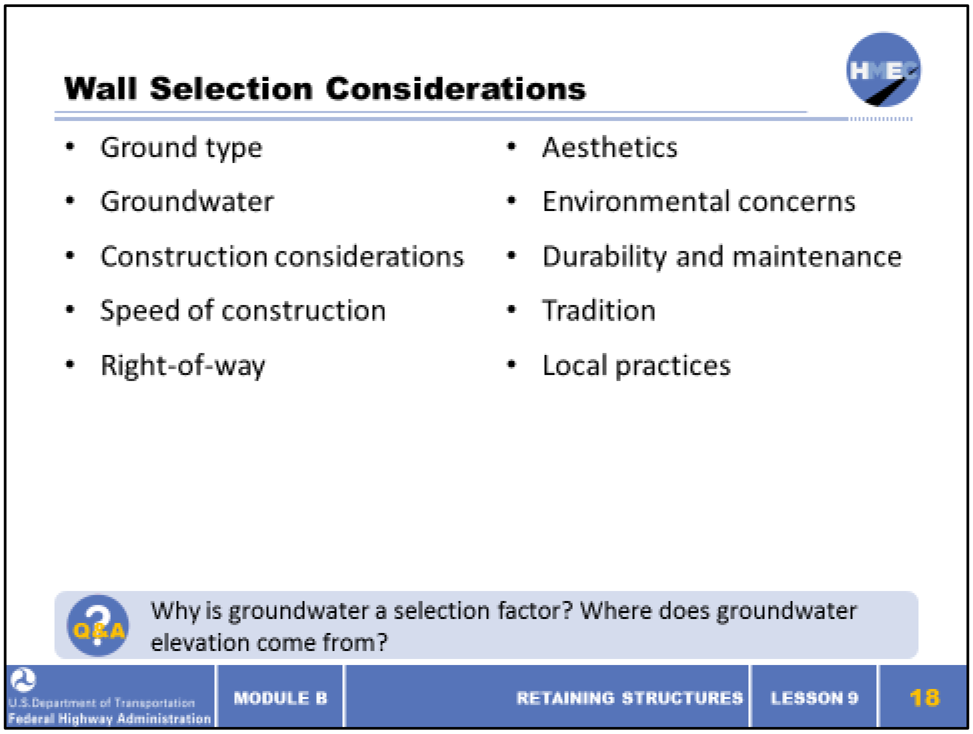
- Permanent Walls
  - Service life 75 to 100 years
  - More restrictive requirements on:
    - Material durability
    - Design factors of safety/allowable stresses
    - Performance
    - Overall appearance

U.S. Department of Transportation Federal Highway Administration | MODULE B | RETAINING STRUCTURES | LESSON 9 | 16

Slide 17


EARTH RETAINING STRUCTURES
Externally Stabilized
Internally Stabilized
In situ walls
Gravity walls
Mechanically Stabilized
(Fill Walls)
Metallic and polymeric
reinforcing strips,
grids and sheets
Anchored Earth
Reinforced Soil Slopes
In situ Reinforced
(Cut Walls)
Soil Nailing
Reticulated Micro Pile
Structural
(Cut Walls)
Sheet-pile*
Soldier pile-lagging*
Cast-in-situ
- Slurry walls
Bored pile
- Contiguous
Tangent pile
Secant pile
- Non contiguous
Chemical
(Cut Walls)
Jet grout
Deep Soil Mix
Cast-in-place Concrete
(Fill Walls)
Cantilever
Counterfort
Buttress
Modular Gravity
(Fill Walls)
Masonry
Crib
Bin
Gabion
Concrete Module
Hybrid Walls
Tailed Segmental
Low density Fills
Braced
Cross-lot
Rakers
Anchored
Augered
Belled
Pressure injected
* can also be used in fill conditions

Slide 18



Groundwater affects every wall type and its elevation or flow must be identified and accounted for with any wall system. Construction issues must be considered: Is it a cut or fill wall? Does the location allow easy access for large equipment? How critical is the speed of construction? Some walls take longer. Does the right of way allow access and construction? If not, certain wall systems may not be appropriate. Aesthetics is critical in some locations and may eliminate some wall types or required a CIP facing. Disposal of excavated material or encroachment into sensitive areas may be environmental factors that must be considered. Will the wall be built in a corrosive environment? That may eliminate some wall systems. How important is the agency tradition of using a certain wall type? Their positive or negative experience with wall systems may control selection. Local contracting expertise or contracting practice may eliminate some walls systems from consideration.

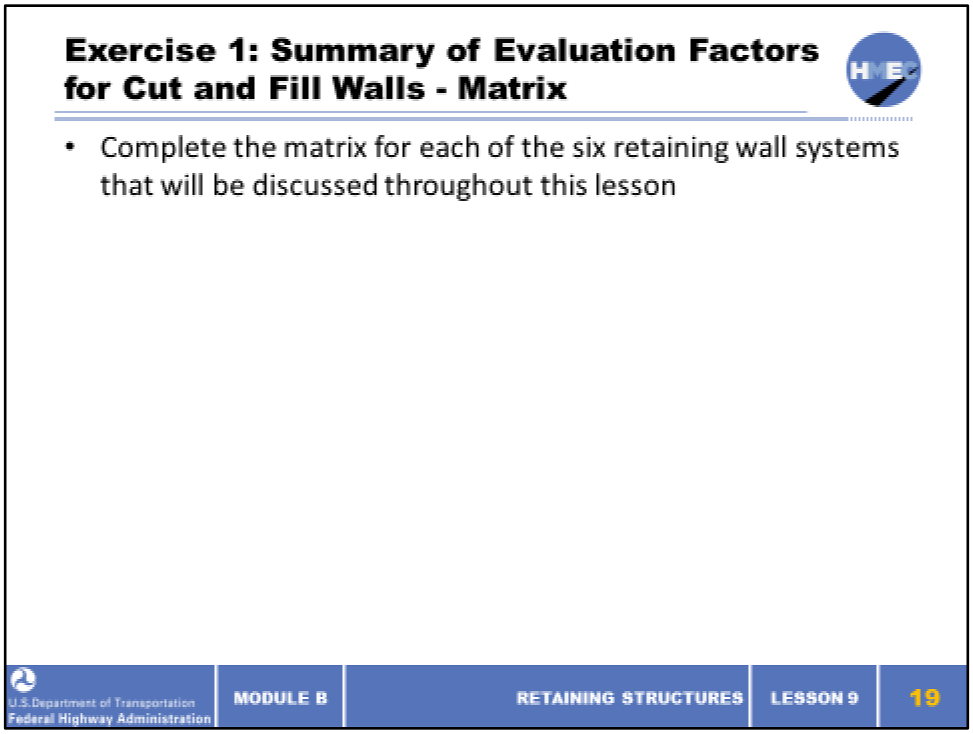
____

____

____

____

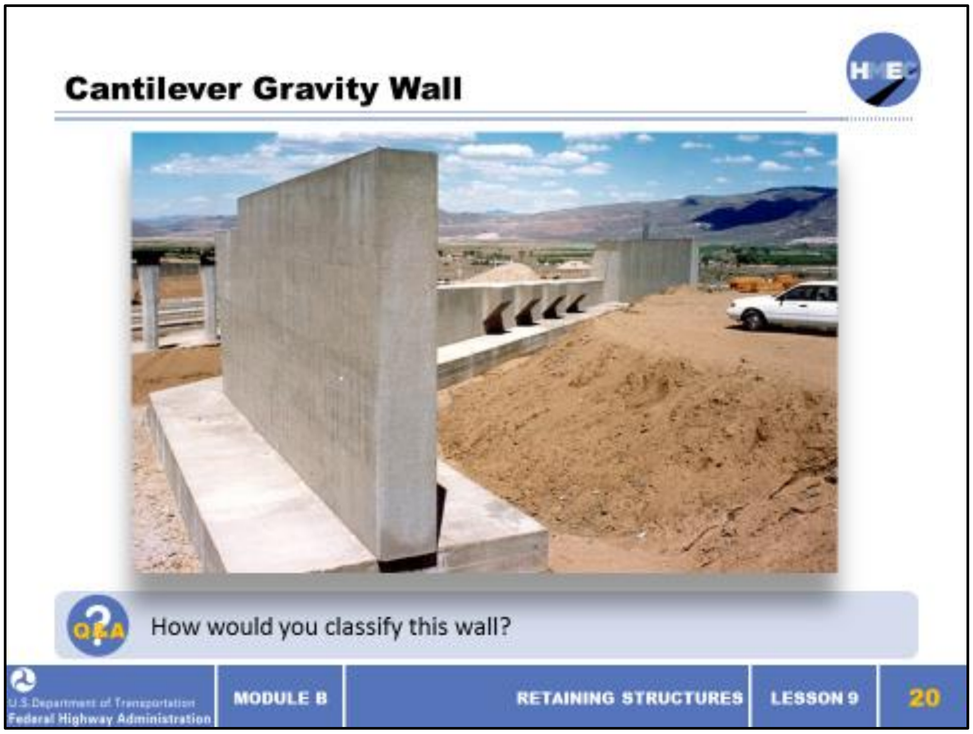
Slide 19



Locate the HMEC Module B Lesson 9 Handout 2: Exercise 1 Summary of Evaluation Factors for Cut and Fill Walls (paper handout) in your handout binder. Complete this matrix as we discuss the six types of retaining walls (three cut and three fill) throughout this lesson.

There will be other types of retaining walls discussed, but that information is not necessary to record on the matrix.

Slide 20



This is a CIP cantilever gravity wall used as an abutment wall for a bridge. These can be supported by a spread footing, but are often supported on piling.

____

____

____

____

____

____

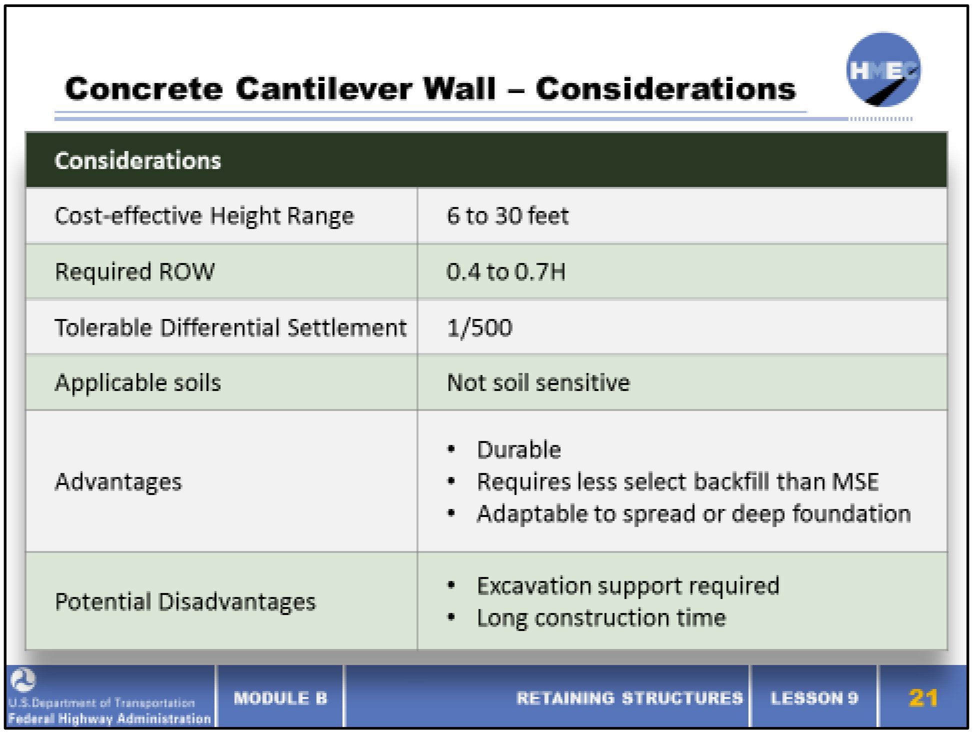
____

____

____

____

Slide 21



Cost-effective height range is the range of height where this wall type may be cost effective versus other types. The required ROW is the space required to construct the wall. The tolerable differential settlement is the amount that this wall type can withstand and not cause problems with cracking and/or maintenance issues; 1/500 is restrictive when compared to other walls, as we will discuss later. The wall is not soil sensitive since it can be founded on spread or deep foundations. Deep foundations would add to the cost.

____

____

____

____

____

____

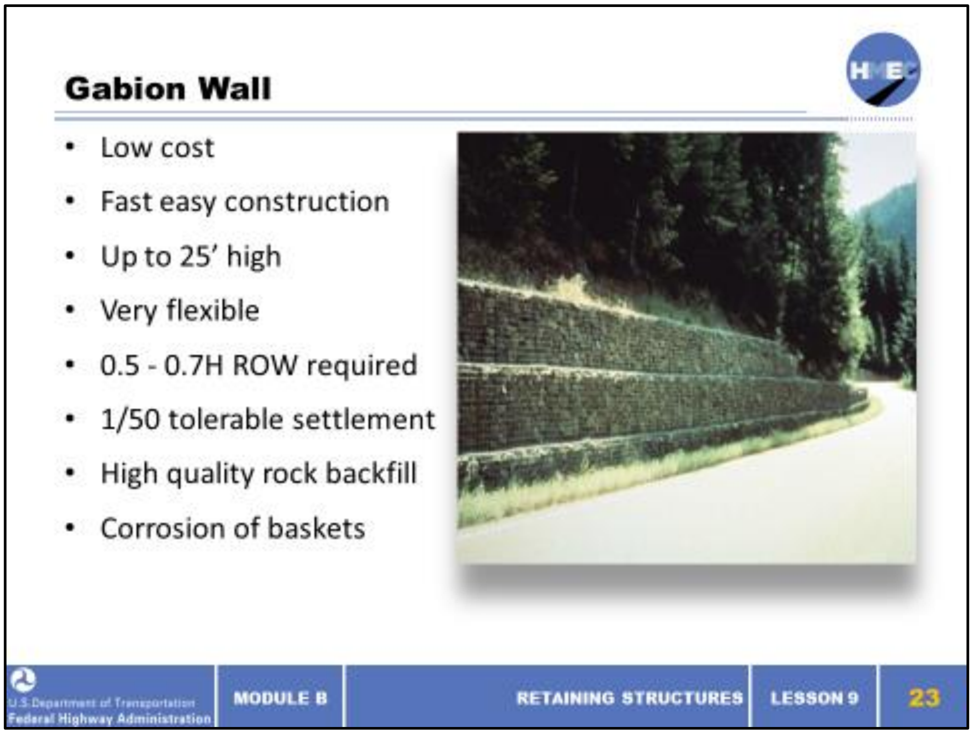
____

Slide 22

This wall system is a cantilever gravity wall with buttresses for added stability. Note that a buttress wall and a counterfort wall are similar. The buttress wall has the "vertical brackets" on the exposed side versus buried in the backfill/retained soil side of the wall as they are for a counterfort wall.

Selection consideration for a buttress or counterfort cantilever wall are similar to the concrete cantilever wall, except that the cost effective height for this type of wall is about 35 ft.

Slide 23



Gabion wall systems are often used in remote areas because the baskets are easily transported and if local rock is available, it can be used to fill the baskets. The filled baskets provide the weight (gravity) to resist the lateral earth pressures from the retained fill. Gabions are one example of a modular gravity wall. Other examples are crib walls, concrete module walls, and steel bin walls.

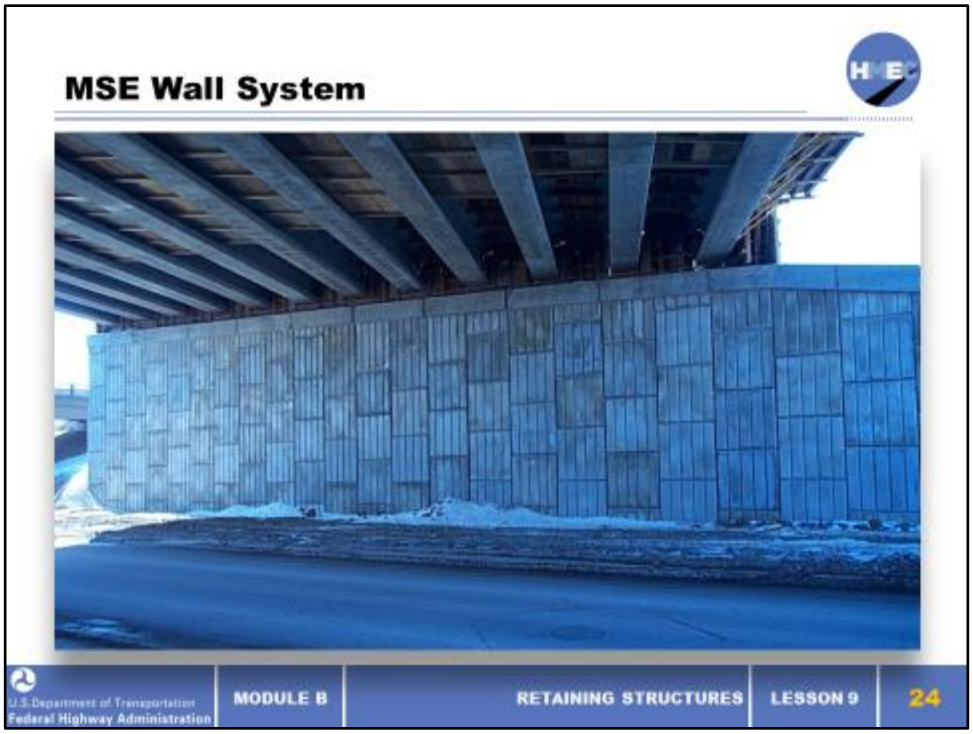
Modular gravity walls comprise two major components: modular structural elements and the fill material placed within these elements. The structural elements, which may be proprietary, may consist of steel modules or bins, prefabricated concrete modules, timber units, wire baskets, or other configurations and materials. The fill material used within the modular units is recommended to be free-draining granular soils, gravel, or rock fragments. Modular gravity walls rely on their own weight and the weight of the fill material within and above the wall elements to resist the applied loads and moments.

____

____

____

Slide 24



The MSE wall system is a bottom-up construction wall system. It is internally reinforced and very flexible. This wall was built to allow a shorter bridge to be used in a roadway overpass. The bridge abutment is founded and piling set about 7 ft. behind the MSE wall face. MSE walls are also often used to support roadway fills when there are ROW restrictions.

MSE walls can be built with several different facing materials. This wall used precast concrete panels, however, modular blocks, welded wire mesh, and geosynthetic wrapped faces are also common

____

____

____

____

____

____

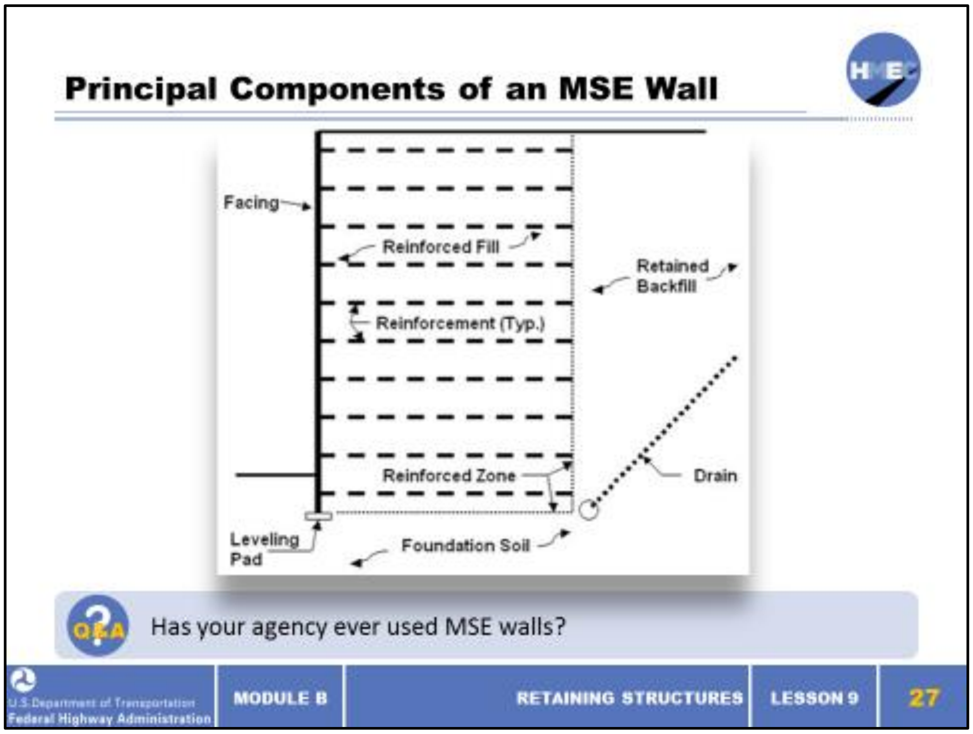
Slide 25

This wall was built to temporarily support an embankment during stage construction of adjacent lanes of an interstate. The relatively inexpensive facing was buried in the fill when the second stage for the remaining lanes was constructed.

Slide 26

A temporary detour bridge was required to maintain traffic during construction. An MSE wall with geogrid reinforcement and facing was used to support the bridge abutments.

Slide 27



A MSE wall is designed by reinforcing a mass of select backfill that then acts a gravity mass to resist lateral earth pressures from the retained soil. The reinforced soil mass is constructed by placing reinforcing elements in layers. The reinforcement can be a variety of materials from steel strips and grids to polymer geogrids or geotextiles. The key point is that the reinforced mass must be inspected to ensure a quality product, just like a reinforced concrete wall would need to be inspected. Note that this wall system is very flexible.

____

____

____

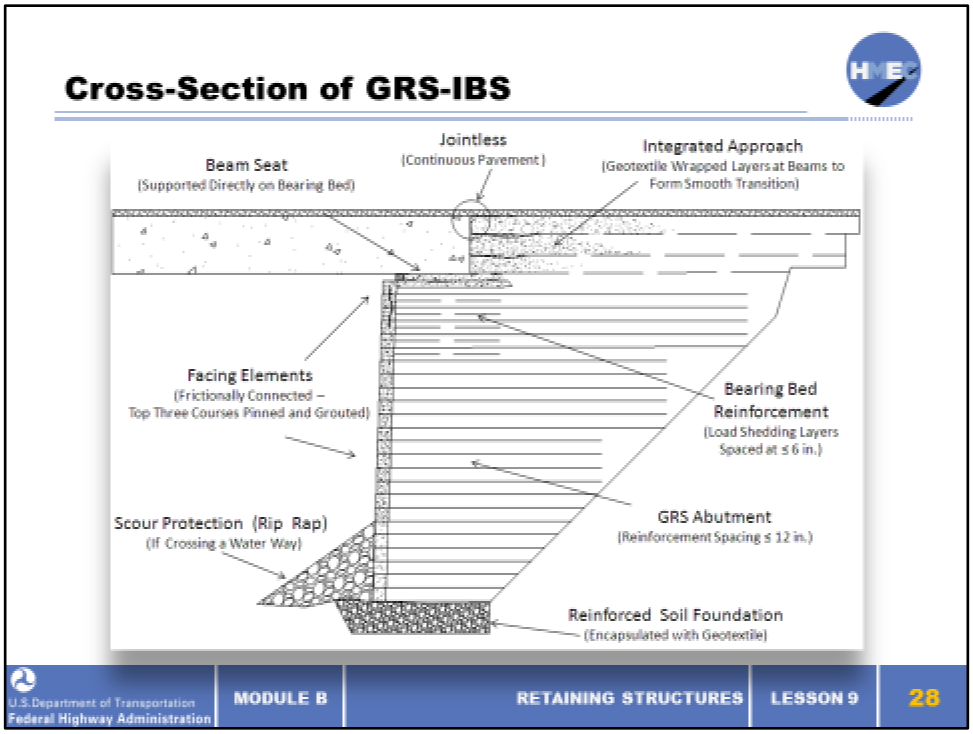
____

____

____

____

Slide 28



This shows a typical cross-section a Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS). This wall has a block facing, is founded on a reinforced soil foundation of geotextile wrapped gravel, has very closely spacing geotextile or geogrid soil reinforcement, and supports a bridge bearing directly on GRS. The soil reinforcement is typically spaced at 8 in. vertically throughout most of the reinforced mass and 4 in. vertically beneath the bridge footing.

Slide 29

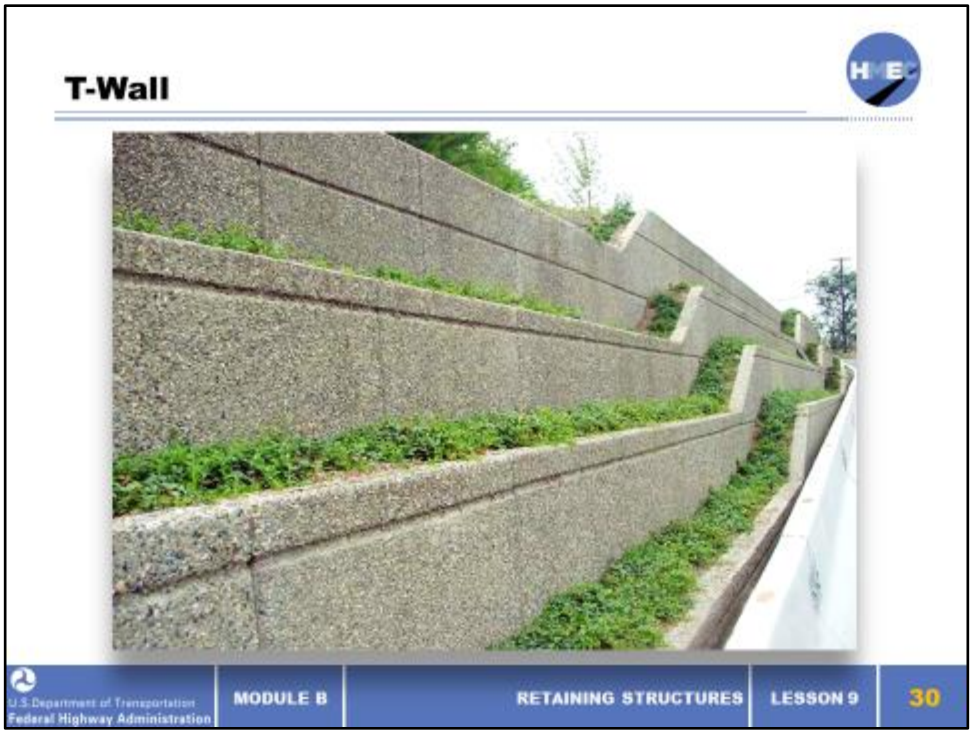
MSE Wall – Considerations

Considerations	
Cost-effective Height Range	10–100 ft., depending on facing used
Required ROW	0.7–1.0 H
Tolerable Differential Settlement	1/60 to 1/100, depending on facing
Applicable soils	Select backfill required
Advantages	• Does not require skilled labor or specialized equipment • Flexible choice of facings
Potential Disadvantages	• Select backfill required • Some facing and reinforcements are subject to corrosion • Construction damage of reinforcement can occur

U.S. Department of Transportation Federal Highway Administration | MODULE B | RETAINING STRUCTURES | LESSON 9 | 29

The cost-effective height for MSE walls is up to 100 ft., larger than any other wall type. The required space to construct the wall is typically 70–100% of the height, but may be larger if weak soils underlay the foundation. These walls are very flexible and deform without failure when settlement occurs. The facing material is the limiting factor for tolerable differential settlement. The precast concrete panels will not tolerate as much settlement as modular blocks or weld wire fabric. The walls require select backfill in the reinforced soil zone to provide the internal strength required for the wall design and to provide free drainage for any water that enters the backfill. No large construction equipment is required, however, the construction techniques are critical to the long-term performance of the wall, and installation crews must be properly trained. Some facing and reinforcement are subject to corrosion in aggressive environments, which must be considered in design. Damage to the reinforcement by construction equipment must be avoided. This is especially critical when geosynthetics are used as reinforcement.

Slide 30



It is considered a gravity structure that combines the weight of externally stabilized retaining walls (e.g. concrete module wall) with the frictional resistance of internally stabilized systems (e.g., MSE walls). The concrete modules are quite large, hence the gravity aspect of stability. The unit consists of the face and a "T" section that extends back into the backfill providing the internal frictional component of the wall system

____

____

____

____

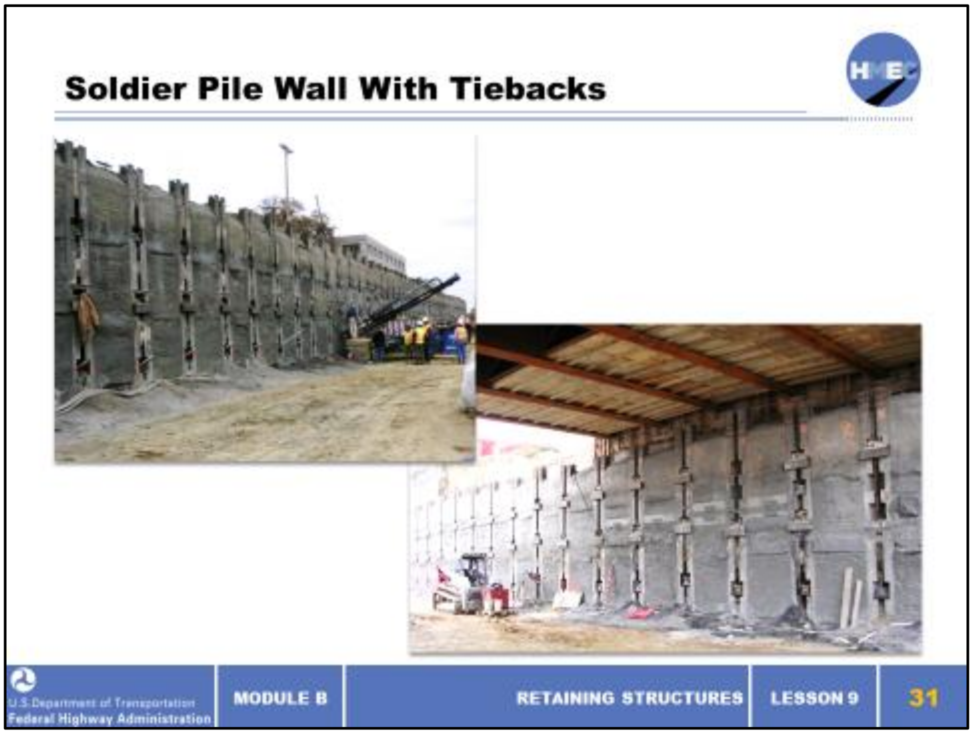
____

____

____

____

Slide 31



This soldier pile wall required tiebacks, however, many do not as it depends on the lateral earth pressures. This wall was constructed in stages and did not require lagging immediately, therefore, reinforcement and shotcrete were placed after each stage was excavated. A permanent CIP concrete facing was cast in front of this permanent retaining wall system for aesthetics.

____

____

____

____

____

____

____

____

Slide 32

Soldier Pile Wall – Considerations

Considerations	
Cost-effective Height Range	Up to 16 ft., and up to 70 ft. with tiebacks
Required ROW	None, unless tiebacks are used
Lateral Movement	Medium
Applicable soils	Need good foundation soil to embed piles into
Advantages	• Rapid construction • Piles can be driven or drilled
Potential Disadvantages	• Difficult to maintain vertical tolerances in hard ground • Potential for ground loss at excavated face

U.S. Department of Transportation Federal Highway Administration | MODULE B | RETAINING STRUCTURES | LESSON 9 | 32

Soldier pile walls are cost effective up to 16 ft. high, and much higher (up to 70 ft.) when used with tiebacks. They require no ROW, unless tiebacks are used, in which case, an easement will be required. This wall system will experience lateral movement, which must be considered if there are nearby structures. The use of tiebacks will reduce the amount of lateral movement. Good foundation soil or rock is required to embed the piles into order to achieve the bending resistance required for design. The method of construction is fast and holes can be predrilled to set the piles, allowing reduced vibrations and penetration of hard rock.

The soil between the piles must be stable enough to stand unsupported after excavation until lagging is placed, or ground loss may occur.

Water tightness is poor, they are typically free draining. However, it depends on the facing used

Slide 33

Sheet pile walls are often used as temporary walls by the contractor to stabilize their excavations. However, they are also used as permanent walls. In the example shown here, the wall forms the abutment for a bridge on a local road.

Sheet pile walls are limited to 12–15 ft. in height. They may be subject to fairly large lateral movement at the upper range of those heights. They are also subject to corrosion.

____

____

____

____

____

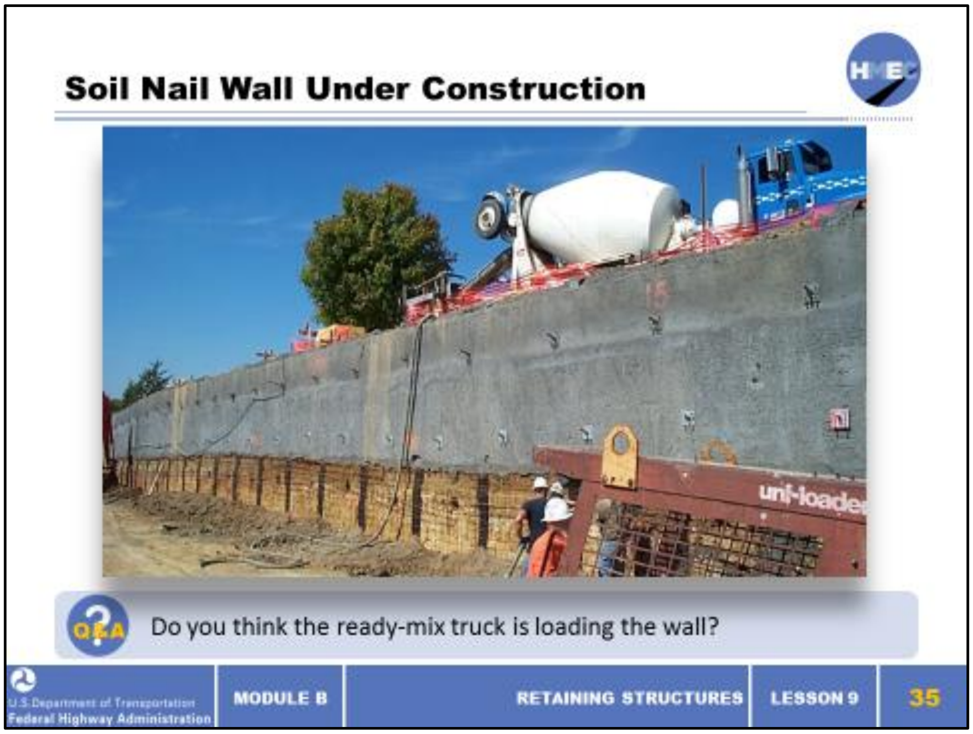
____

____

Slide 34

Soil nail walls are one of the in-situ reinforced walls that are constructed to support temporary and permanent excavations. Typically soil nail walls have a CIP concrete facing or sculpted shotcrete as their final face for aesthetic reasons. This soil nail application was used during widening of an interstate though a downtown district. The restricted ROW and close proximity of houses made the two-tier soil nail system an excellent choice.

Slide 35



The stages are typically 3–5 ft., depending on the stability of the soil being excavated. After the lift is excavated, holes are drilled at a downward slope beyond the anticipated failure surface for installation and grouting of the soil nails. Drainage (the black strips) is placed along with wire mesh and the lift is shotcreted. The process is repeated until final grade is reached.

____

____

____

____

____

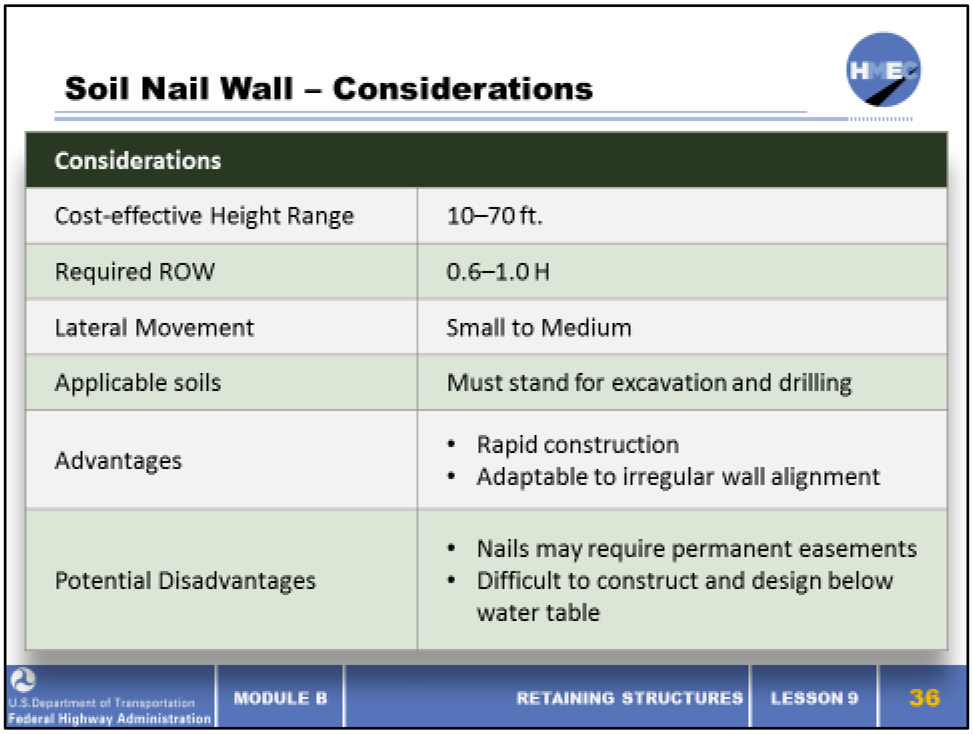
____

____

____

____

Slide 36



Soil nail walls can be cost effective up to 70 ft. high and can be installed as tiered walls if more height is required. Since this wall system consists of installing steel rods in the soil behind the wall face, ROW or an easement is required. Lateral movement is required to develop the stabilizing effect of the soil nails, but it is relatively small. There are some soils that are not conducive to using soil nails. The soil must be stable enough to stand unsupported in a 3–5-ft. lift to allow installation of the nails and shotcreting of the face. Soil nail construction proceeds rapidly.

Water tightness is poor. However, it depends on the facing used.

Slide 37

This tangent wall system was constructed using drilled shafts. The first shaft in the left photograph has crosshole sonic logging (CSL) tubes showing. CSL was used for integrity testing on a percentage of the drilled shafts.

A tangent or secant pile wall consists of a line of drilled shafts (also referred to as bored piles). If the bored piles are contiguous, or tangent, to each other, the wall is called a "tangent pile" wall.

Slide 38

Tangent Pile Wall – Considerations

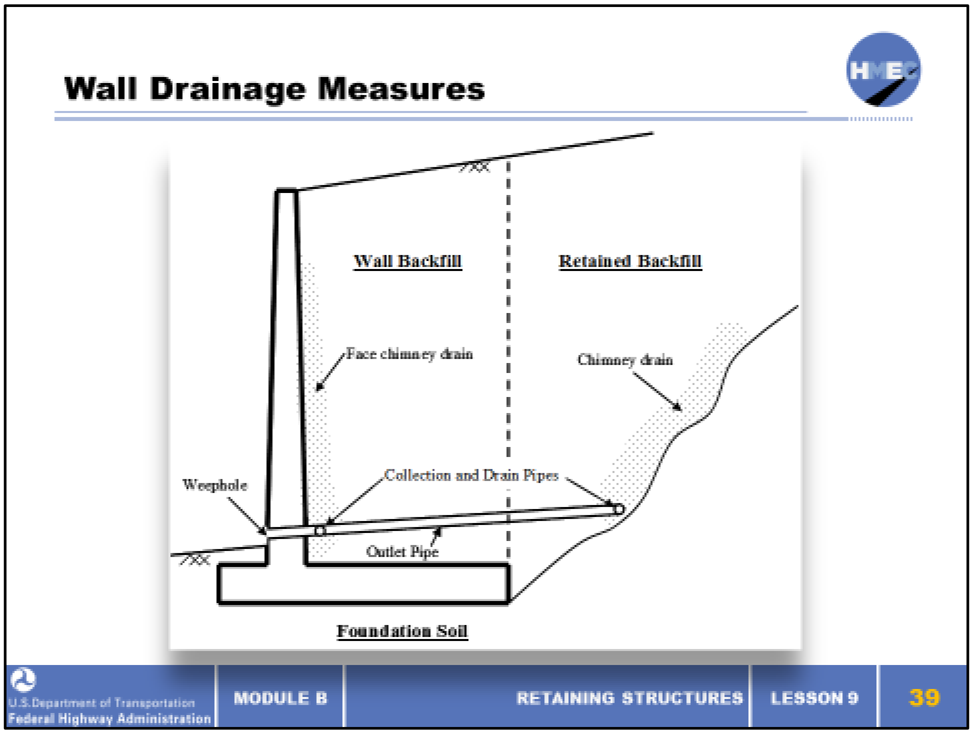
Considerations	
Cost-effective Height Range	10-30', 20-80' with anchors
Required ROW	None, unless anchors are used
Lateral Movement	Small
Applicable soils	Need good foundation soil to embed piles into
Advantages	• Adaptable to irregular layout • Can control wall stiffness
Potential Disadvantages	• Difficult to maintain vertical tolerances in hard ground • Requires specialized equipment • Can be significant spoil for disposal

U.S. Department of Transportation Federal Highway Administration | MODULE B | RETAINING STRUCTURES | LESSON 9 | 38

The diameter of the drilled shafts will vary depending on the exposed height of the wall and the lateral earth pressure that must be resisted. This wall system is cost effective up to 30 ft., when anchors (tiebacks) are used, up to 80 ft. No ROW is required unless tiebacks are used. The lateral movement depends on the diameter of the shaft, but is typically small, which is important if buildings are in close proximity. A good foundation soil or rock is required to provide support for the drilled shafts. There will be excavation cuttings (spoil) that will need to be disposed. Specialized drilled shaft equipment is required for construction.

Water tightness is poor. However, overlapping the shafts to form a secant wall improves the water tightness.

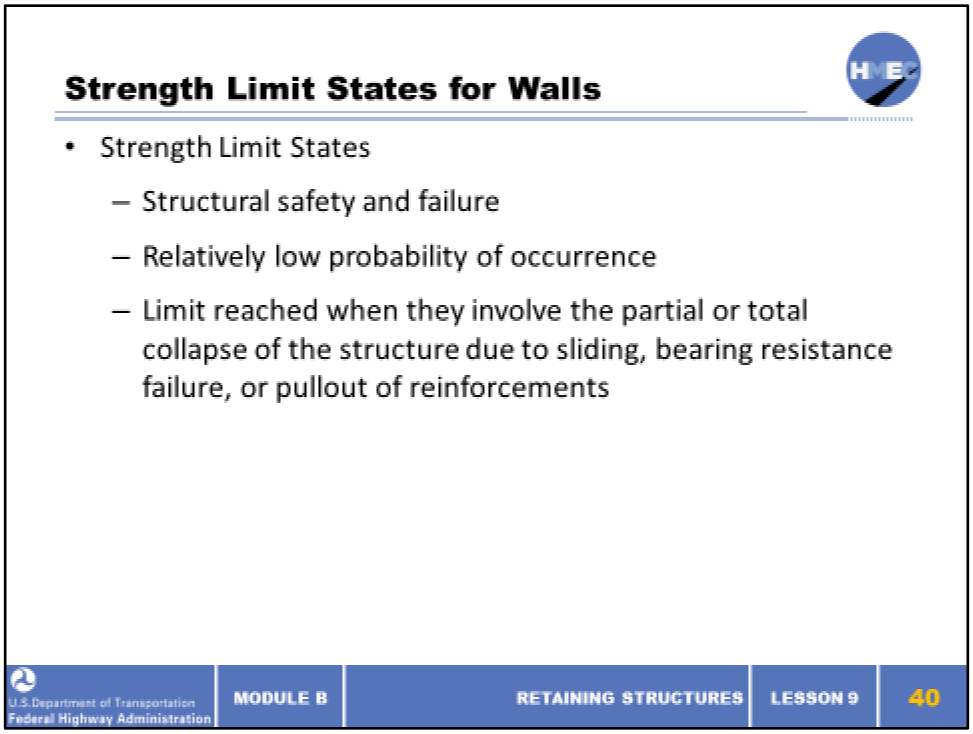
Slide 39



Other wall systems have a more free-draining face (MSE, gabions etc.), however, drainage must still be collected behind the wall and allowed to drain from the backfill.

Both groundwater and surface water drainage must be considered and accounted for during design and construction. Explain that retaining wall select backfill is typically assumed to be free draining. Many specifications allow about 15% fines in the select backfill.

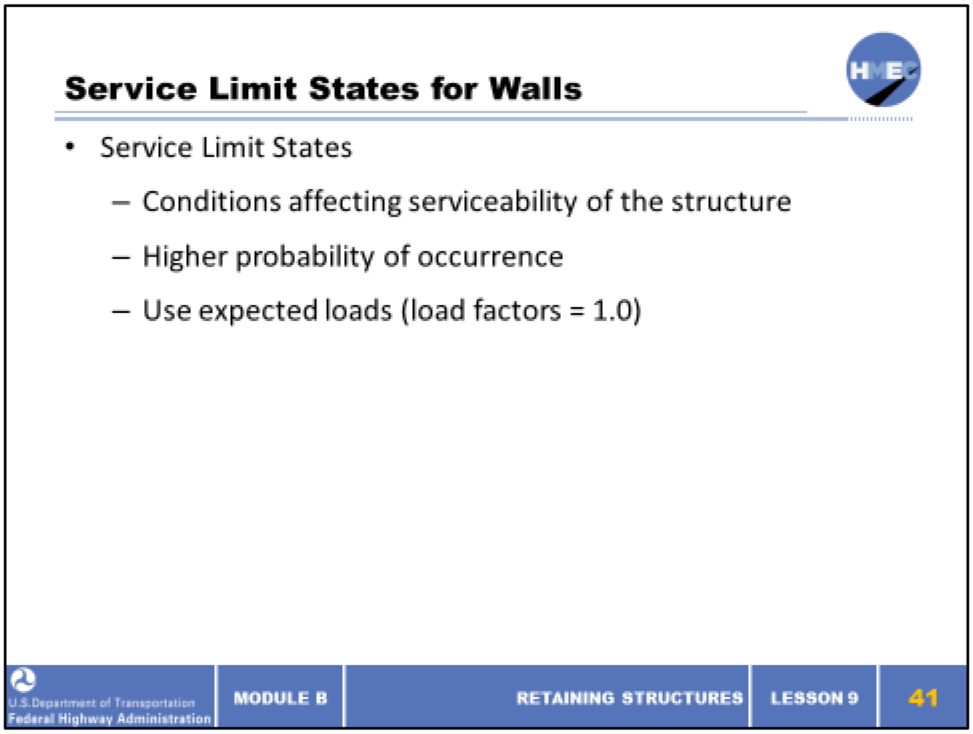
Slide 40



There are both structural and geotechnical strength limit states for retaining walls. Evaluation of geotechnical strength limit states is used to ensure a low probability against failure due to sliding, bearing failure, eccentricity, or pullout of reinforcement.

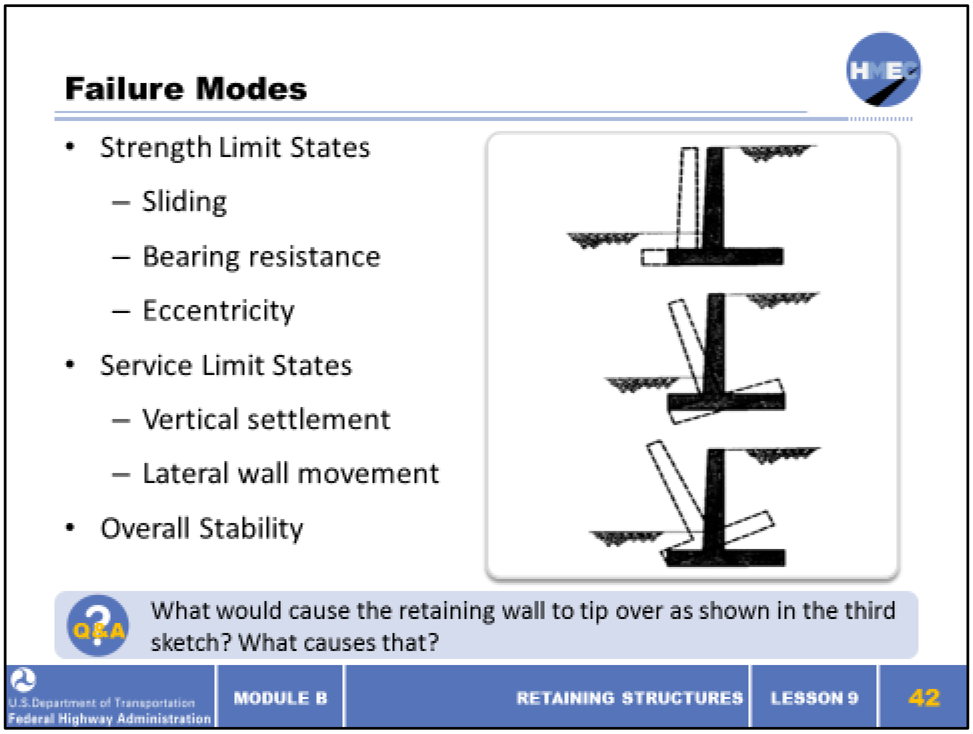
If the retaining wall system is properly designed, the probability of a strength limit state failure is low.

Slide 41



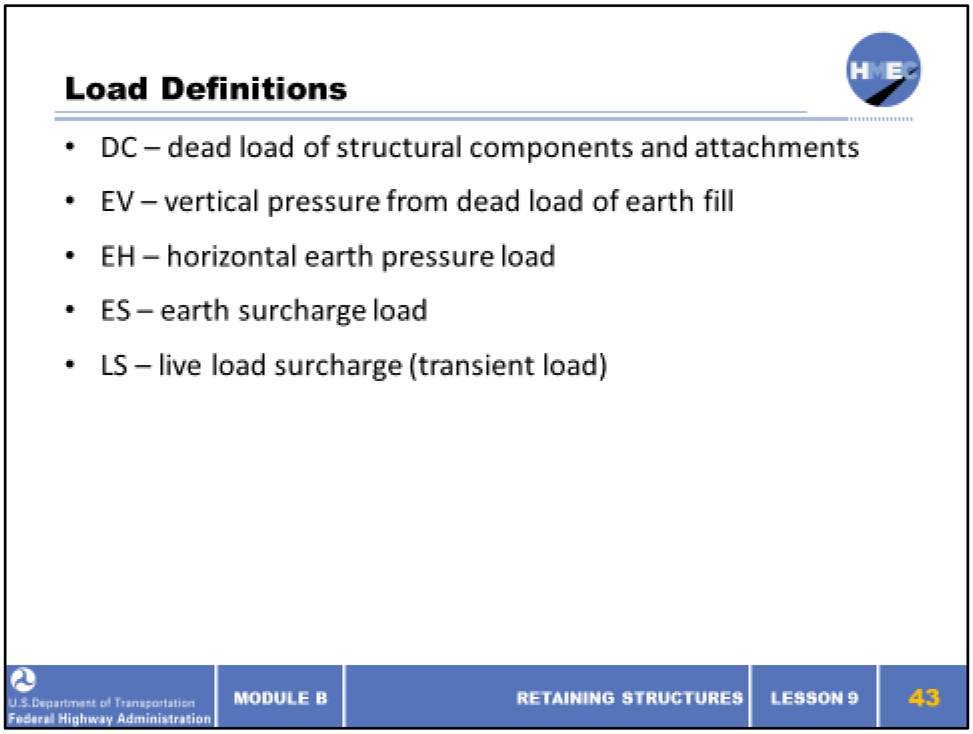
Service limit state analysis is performed with load and resistance factors equal to 1.0 to evaluate deformations.

Slide 42



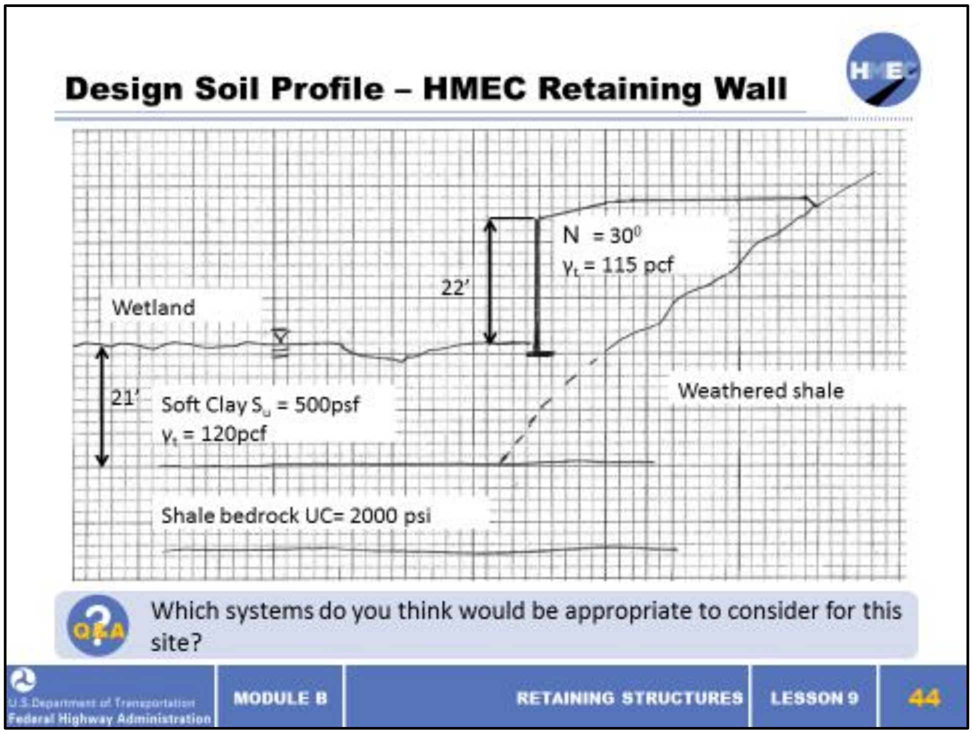
This slide depicts the possible failure modes that must be analyzed during design.

Slide 43

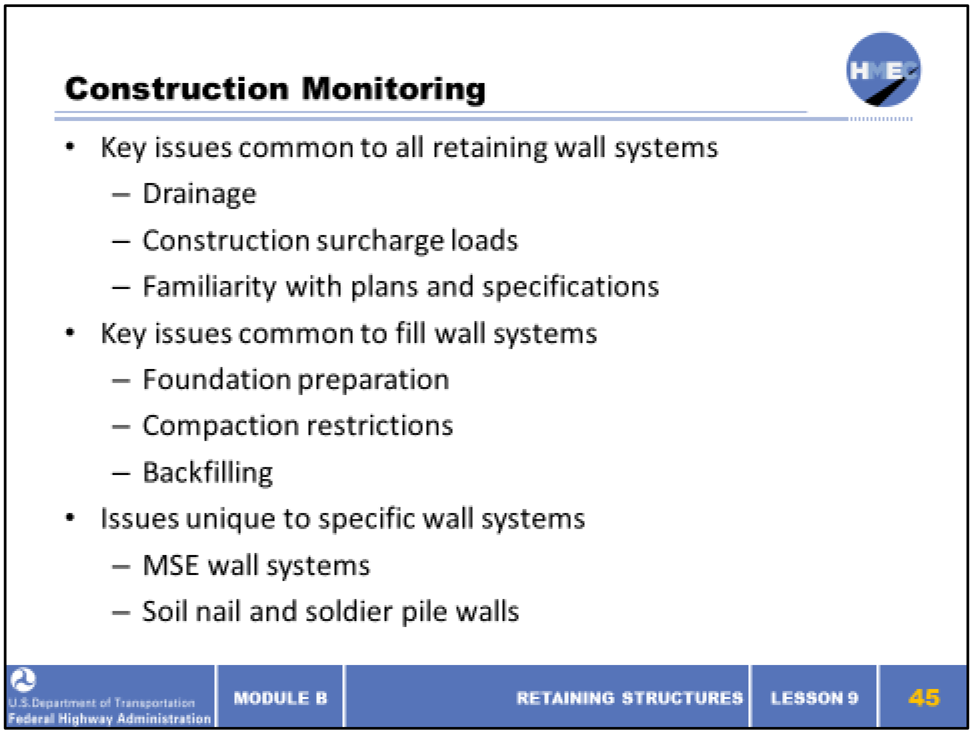


Some of these loads we have seen before, others are new and represent loads used to design retaining walls.

Slide 44


Design Soil Profile – HMEC Retaining Wall
HMEC
N = 30^0
γ_t = 115 pcf
22'
Wetland
21'
Soft Clay S_u = 500psf
γ_t = 120pcf
Weathered shale
Shale bedrock UC= 2000 psi
Q&A
Which systems do you think would be appropriate to consider for this site?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
RETAINING STRUCTURES
LESSON 9
44

Slide 45



Some issues are critical to the successful installation regardless of the retaining wall system being installed. As we discussed, drainage of groundwater and surface runoff must be properly addressed in design and construction. Note that surface water control by the contractor during wall construction is also crucial. If it is not controlled, the wall may not perform as expected or construction may not be possible. The plans and contractor submittals must clearly outline how drainage will be handled and those provisions for drainage must be followed.

Some construction surcharge loads are planned for in design, they become an issue during construction when they are unexpected or induce a load on the wall during a stage of construction that was not anticipated during design.

An inspector must be familiar with the plans and specifications in order to properly inspect the construction of the retaining wall system being installed. Several retaining wall systems are constructed using a combination of agency requirements and contractor submittals. The inspector must be familiar with both. Subsurface reports and boring logs and material specifications for backfill and other components, as well as requirements for spread or deep foundations may all be pertinent to wall construction.

Several issues that must be inspected are related to the wall system.

____

____

____

____

____

____

____

____

____

____

____

Slide 46

Placement of backfill and compaction are key elements of retaining wall construction. Backfill specification, especially gradation are key to drainage and compaction. A gradation that is free draining also reduces the corrosion potential of metallic reinforcement used in some MSE walls. Backfill specifications typically require select backfill be used for fill walls. The backfill must be relatively free draining and be placed and compacted in thin lifts. The placement sequence must ensure that unbalanced lateral pressures (from compaction effort) are not placed on the retaining wall. Some walls, such as the MSE shown here, require small lightweight compaction equipment to be used near the face of the wall to limit facing movement or distortions.

Slide 47

Incorrect placement of drainage outlet pipes may induce hydrostatic pressure on the wall and would be a problem for any fill wall. The top photograph shows an outlet pipe placed well above the footing of the MSE wall face. The portion of the backfill below that outlet pipe may not drain properly. Some drainage will occur through the joints between the precast panels. Those joints, both horizontal and vertical, must be covered with filter fabric (black strips) or backfill material will pipe through the joint. The bottom photograph depicts an improper backfilling technique used by the contractor. The area next to the wall and below the reinforcement straps should already be filled with backfill and compacted. This method often results in poor alignment and open joint issues with MSE walls

____

____

____

____

____

Slide 48

Results of Improper Backfilling
1
2
4
3
Have you been involved in MSE construction inspection?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
RETAINING STRUCTURES
LESSON 9
48

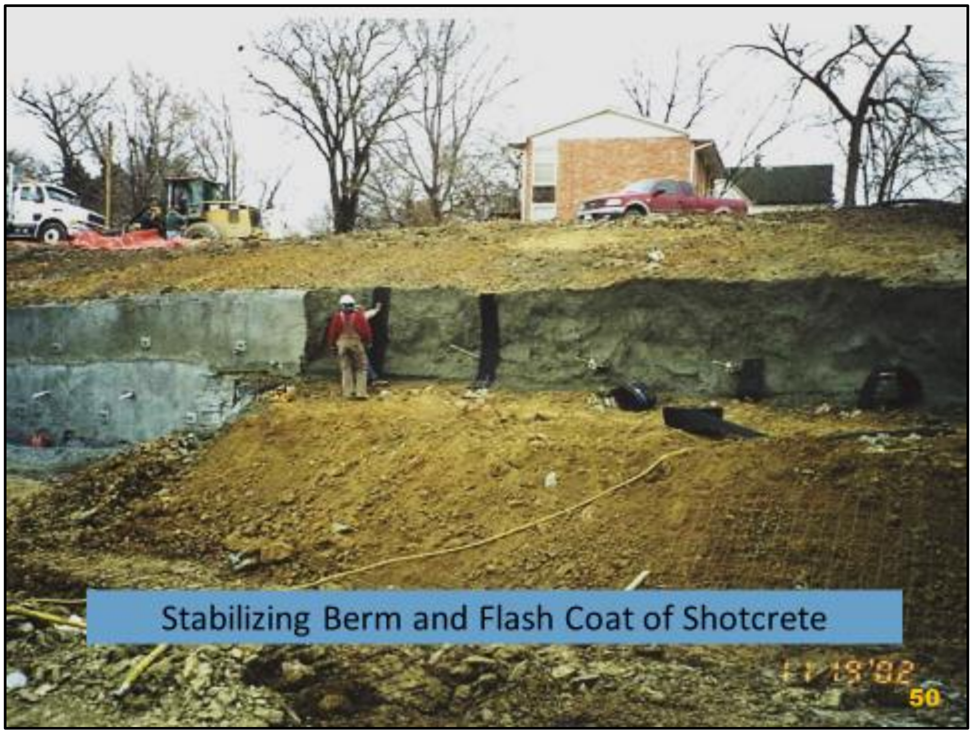
Slide 49

This slope stood until after the soil nails were placed but collapsed before the exposed face was shotcreted.

A similar issue exists for soldier pile walls, where the excavated face must stand unsupported until lagging is placed.

Case Study Notes: The contractor on this project had good success for most of the soil nail walls constructed. This one location failed, probably due to a slightly higher moisture content. This was a contractor means and methods issue and was corrected by the contractor without cost to the owner.

Slide 50



The old soil nails were removed, the face was cleaned up and cut back to virgin soil, a flash coat of shotcrete was placed, and the construction process began again. No problems were experienced with the repair process.

Case Study Notes: This soil nail wall was constructed as part of the reconstruction of I-235 through Des Moines, Iowa.

Slide 51

This photograph depicts damage from groundwater seepage through the shotcrete facing on a soil nail wall. Vertical drainage strips and weep holes were provided throughout the length of this wall. They did not provide adequate drainage for the seepage. The solution was to core 4-in. diameter holes well back into the retained soil and install drainage socks with a weep hole outlet. These were installed at all seep locations as well as on regular intervals along the wall. Since a CIP concrete facing was to be cast in front of this shotcrete facing, drainage through the permanent facing was also provided.

____

____

____

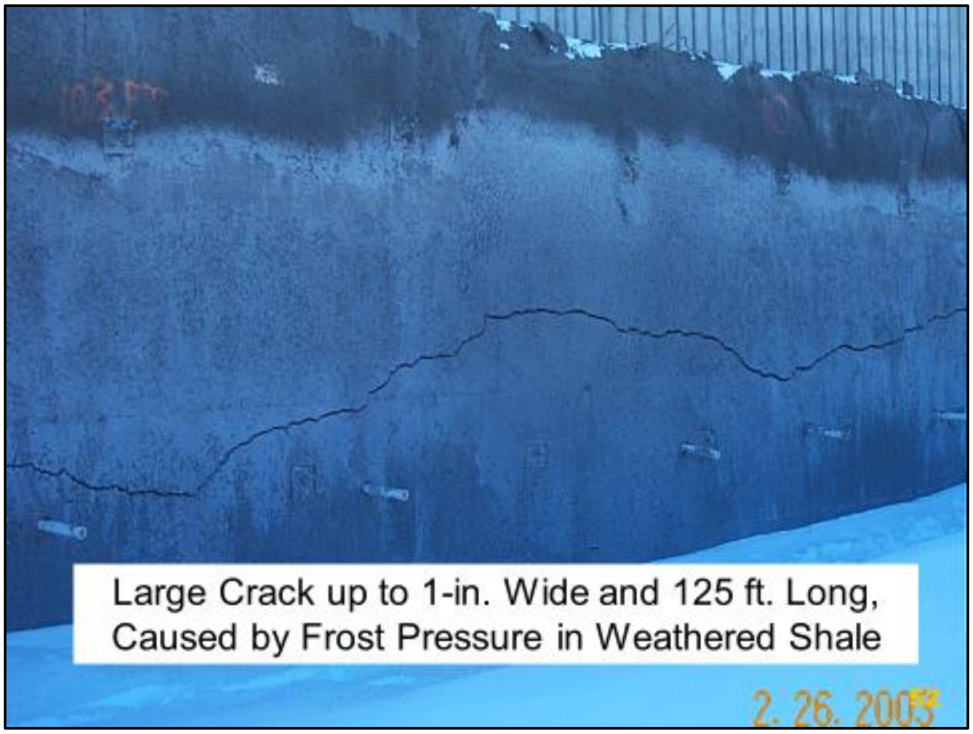
____

____

____

____

Slide 52



This section of soil nail wall was retaining weathered shale. Groundwater seeped along bedding planes in the shale to the back of the shotcrete face. During the cold Iowa winter, the water at the back face of the shotcrete froze and created enough frost pressure to crack this section of wall. The shotcrete is 6 in. thick and reinforced with wire mesh.

Case Study Notes: This wall was built as part of the I-235 reconstruction in Des Moines, Iowa. Evidence of the seepage and freezing of groundwater in the shale layer was found in exposed shale adjacent to this wall section. Future construction that would have shotcrete exposed over the winter included insulation behind the shotcrete.

____

____

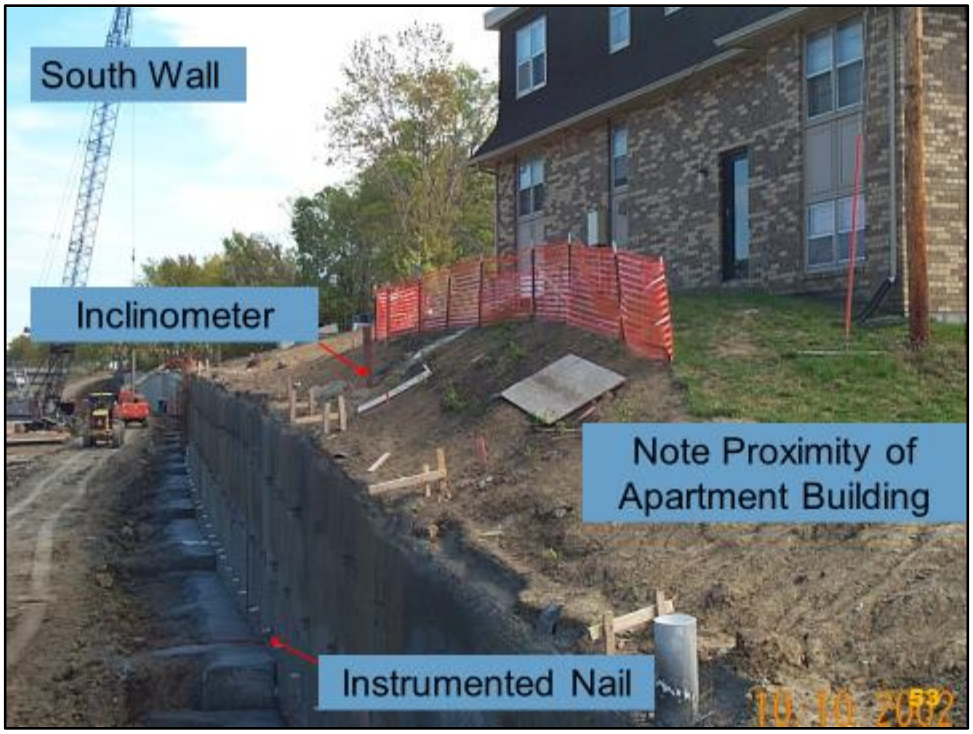
____

____

____

____

Slide 53



This soil nail wall was built because of the close proximity of the apartment building. One of the soil nails was instrumented with strain gauges along its length and a load cell at the shotcrete face. An inclinometer was also installed to measure movement behind the wall.

Case Study Notes: This soil nail wall was constructed as part of the reconstruction of I-235 in Des Moines, Iowa. The inclinometer and visual evidence at the apartment indicated that the soil nail wall deformed laterally enough to cause damage to the apartment building. It was determined that additional, longer soil nails were required to stabilize the wall. These were installed at the contractor's expense. The owner/agency purchased the building and had it demolished.

____

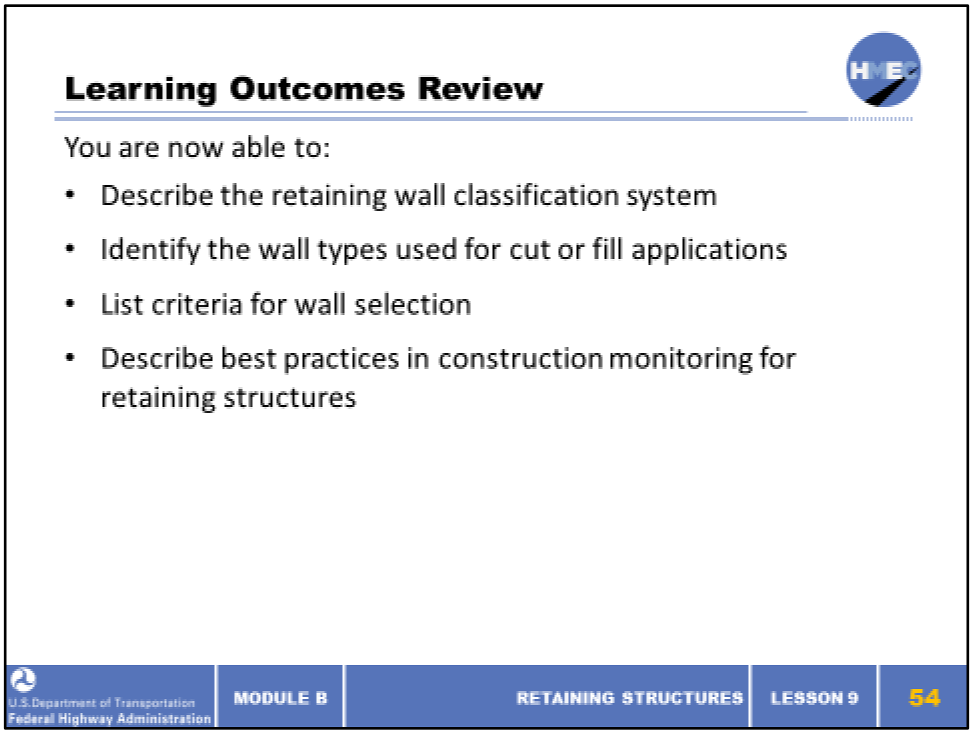
____

____

____

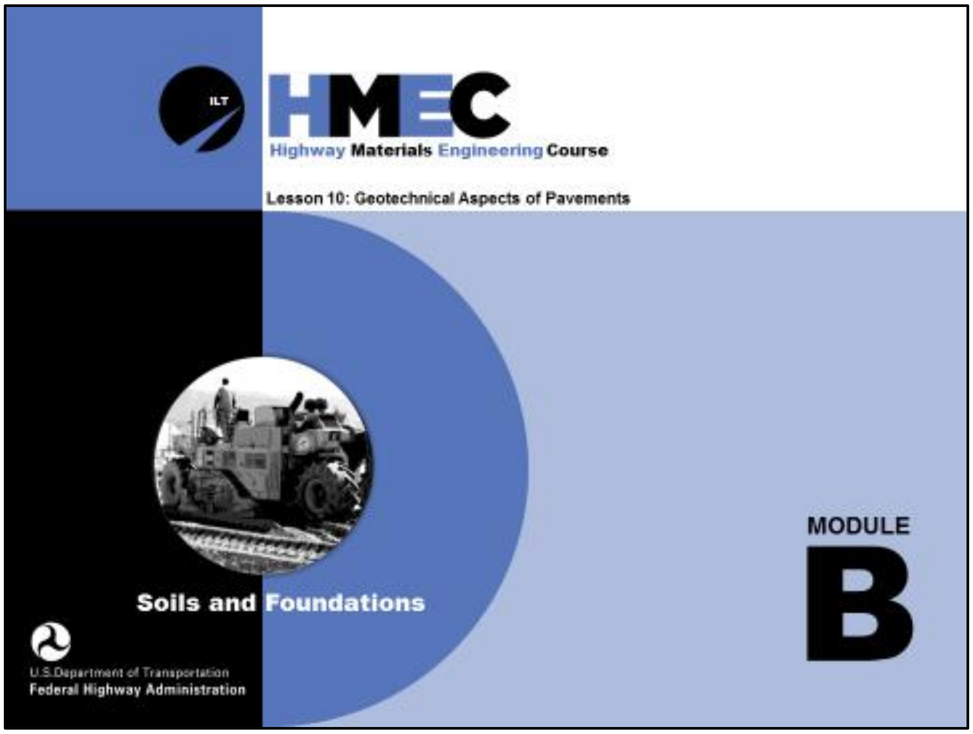
____

Slide 54

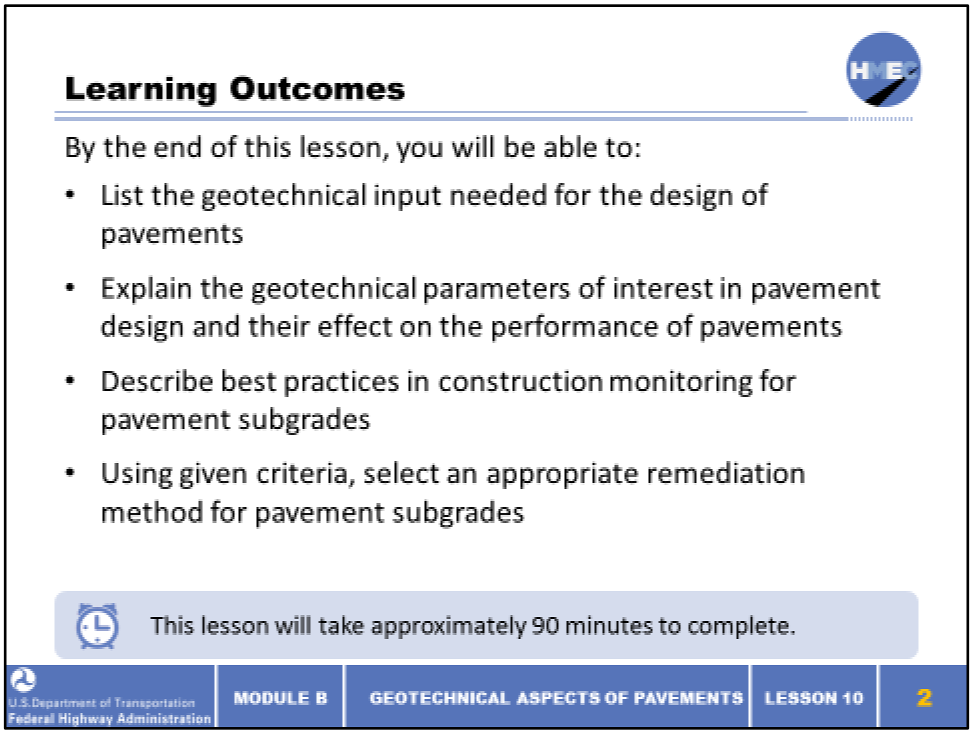

Learning Outcomes Review
You are now able to:
• Describe the retaining wall classification system
• Identify the wall types used for cut or fill applications
• List criteria for wall selection
• Describe best practices in construction monitoring for retaining structures
U.S. Department of Transportation
Federal Highway Administration
MODULE B
RETAINING STRUCTURES
LESSON 9
54

This page is intentionally blank.

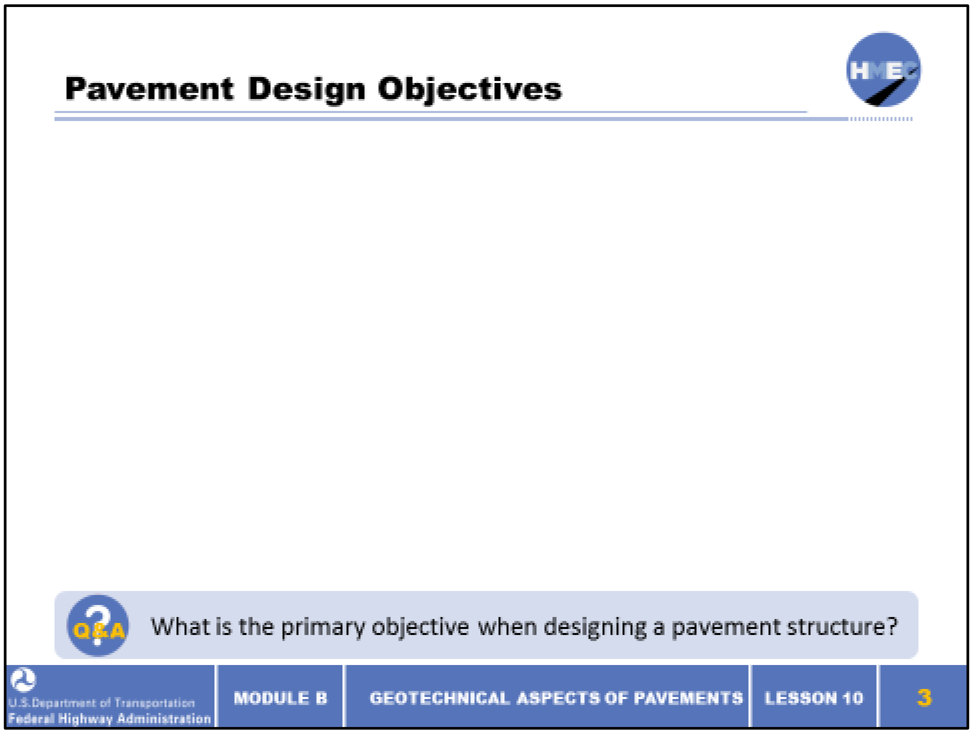
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 10: Geotechnical Aspects of Pavements
MODULE
B
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration

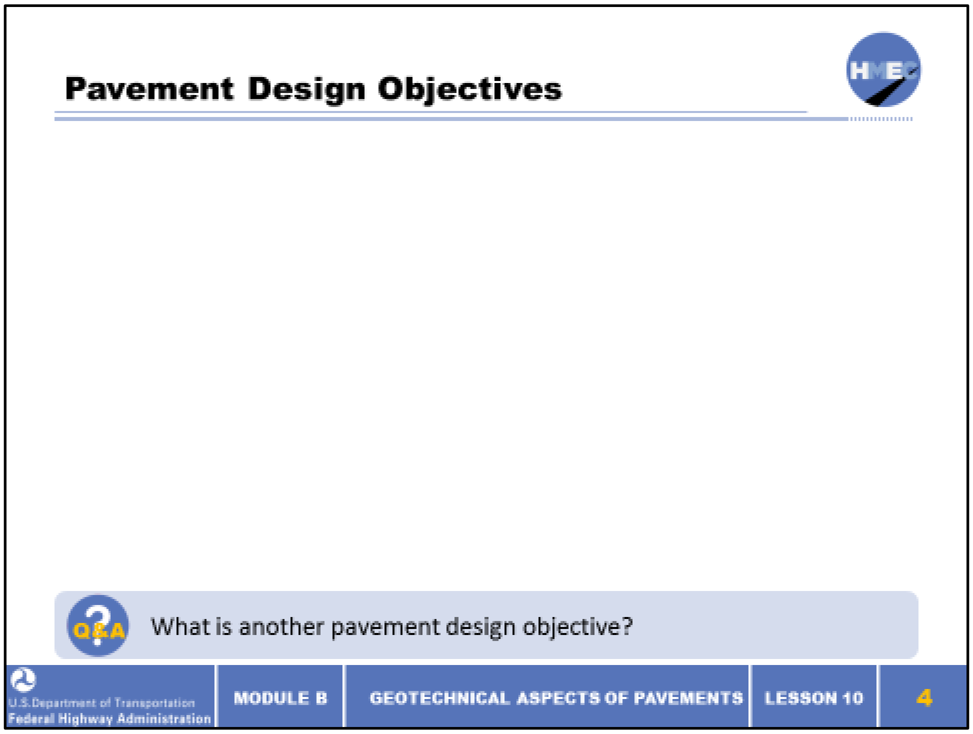
Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• List the geotechnical input needed for the design of pavements
• Explain the geotechnical parameters of interest in pavement design and their effect on the performance of pavements
• Describe best practices in construction monitoring for pavement subgrades
• Using given criteria, select an appropriate remediation method for pavement subgrades
This lesson will take approximately 90 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
2

Slide 3


Pavement Design Objectives
HMEC
Q&A
What is the primary objective when designing a pavement structure?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
3

Slide 4



Other design objectives not listed on slides include:

- Low noise (increasingly important for urban interstates and expressways); and
- Economical design ("an engineer is someone who can do for one dollar what anyone could do for two").

____

____

____

____

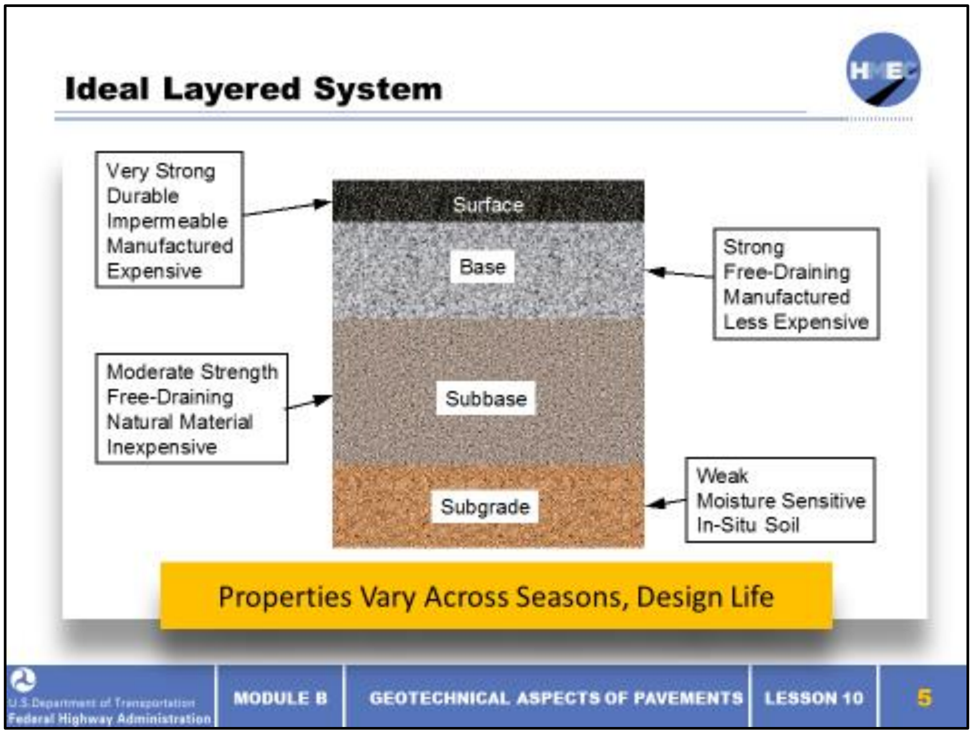
____

____

____

____

Slide 5



Changes in material characteristics going from top to the bottom of the pavement structure:

- Manufactured → natural;
- Stronger/stiffer → weaker/softer;
- Free-draining → poorly draining (except for surface layer, which ideally is a moisture barrier);
- Moisture insensitive → moisture susceptible; and
- Most expensive → least expensive.

____

____

____

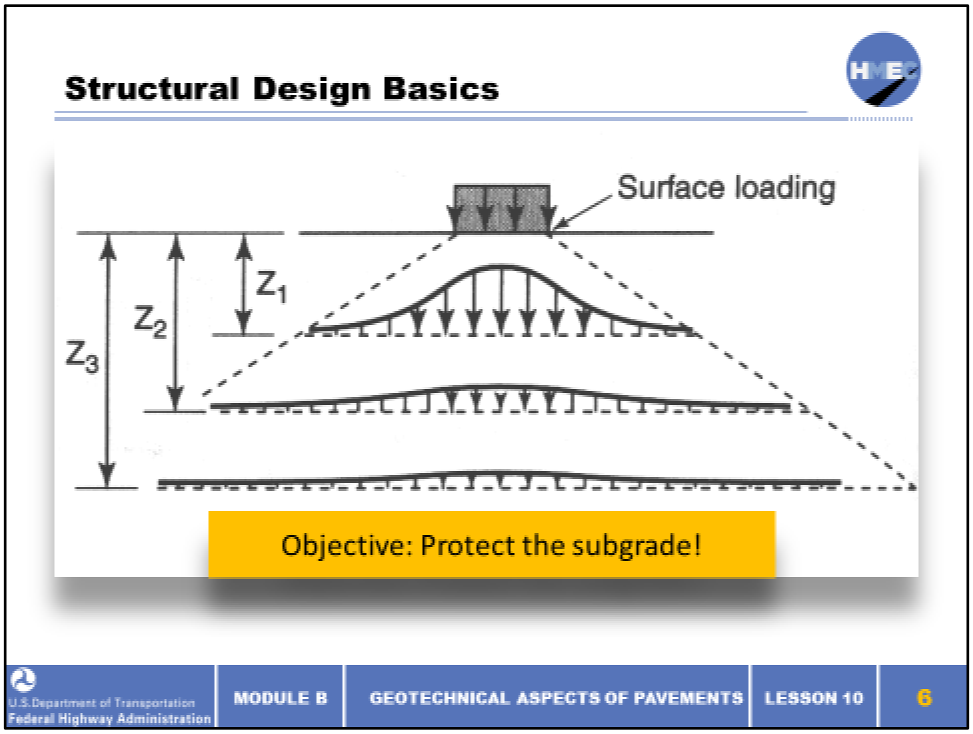
____

____

____

____

Slide 6



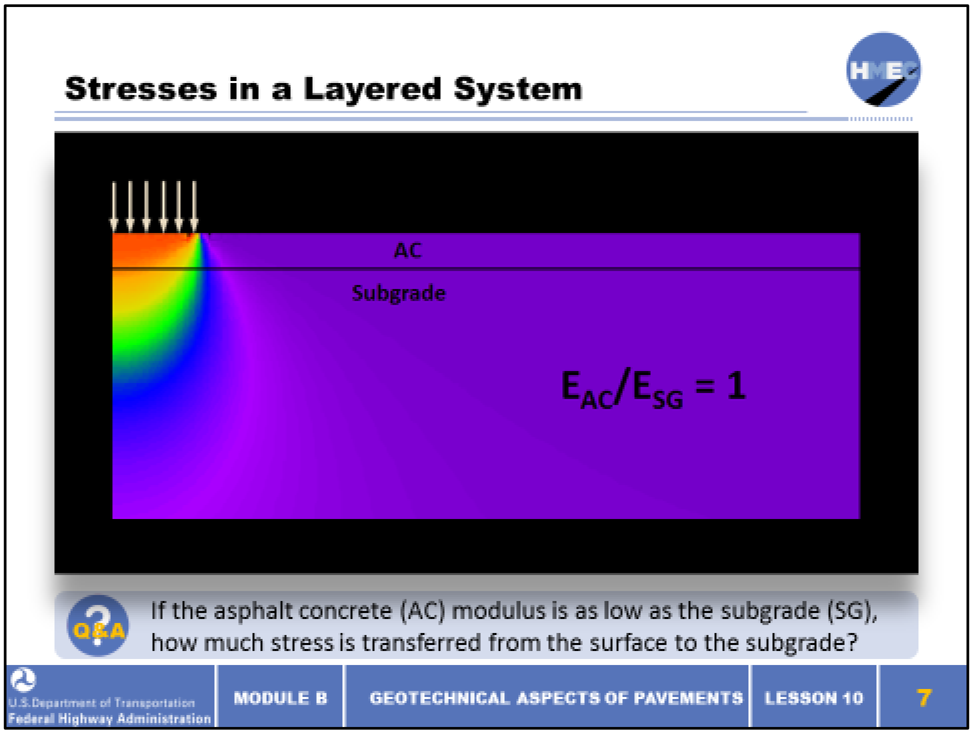
Induced stress distribution from surface load decreases in peak magnitude and increases in horizontal extent with increasing depth.

The objective of pavement design is to get this distribution of load to lower layers to happen faster than it would in a homogenous system. i.e., less stress on lower layers.

The main purpose of pavement structure is to reduce stresses to acceptable levels at the top of the subgrade—i.e., to protect the subgrade (this was the primary basis of many early pavement design methods).

The unbound soil layers in a pavement provide a substantial part of the overall structural capacity of the system, especially for flexible pavements. As shown in this figure, the stresses induced in a pavement system by traffic loads are highest in the upper layers, and they diminish with depth. Consequently, higher quality—and generally more expensive—materials are used in the more highly stressed upper layers of all pavement systems, and lower quality and less expensive materials are used for the deeper layers of the pavement.

Slide 7



The next three slides show an animation of how the vertical stress bulb below a surface load on a two-layer system shrinks as the modulus ratio increases.

This first slide is for homogenous conditions.

____

____

____

____

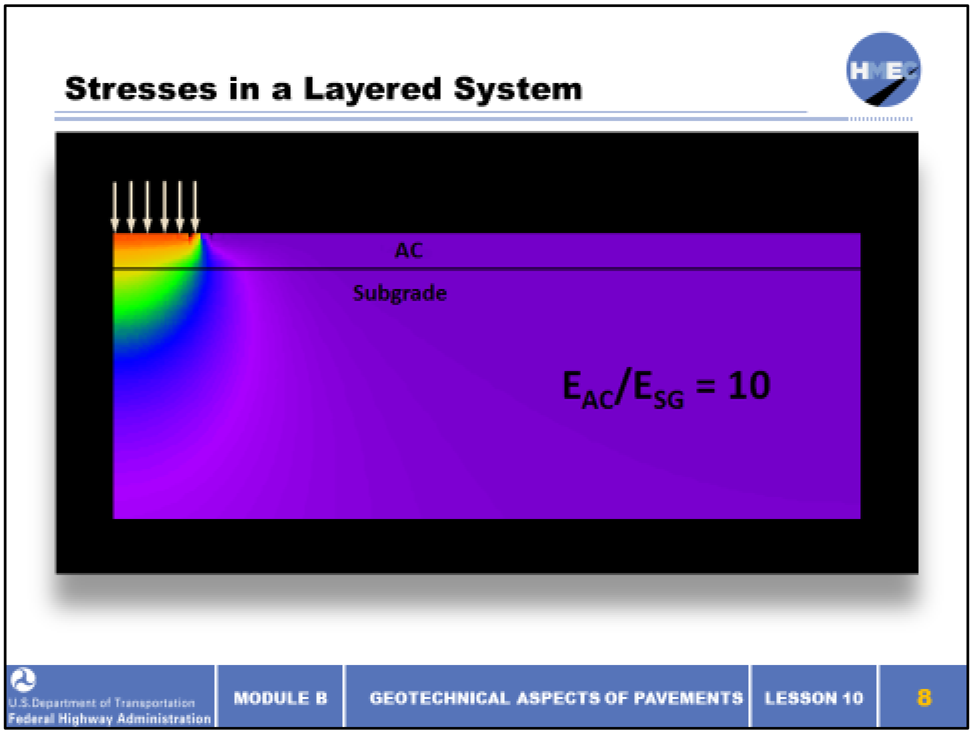
____

____

____

____

Slide 8



Modulus ratio of 10. The stress bulb moves up, but there's still considerable load transferred into the subgrade.

____

____

____

____

____

____

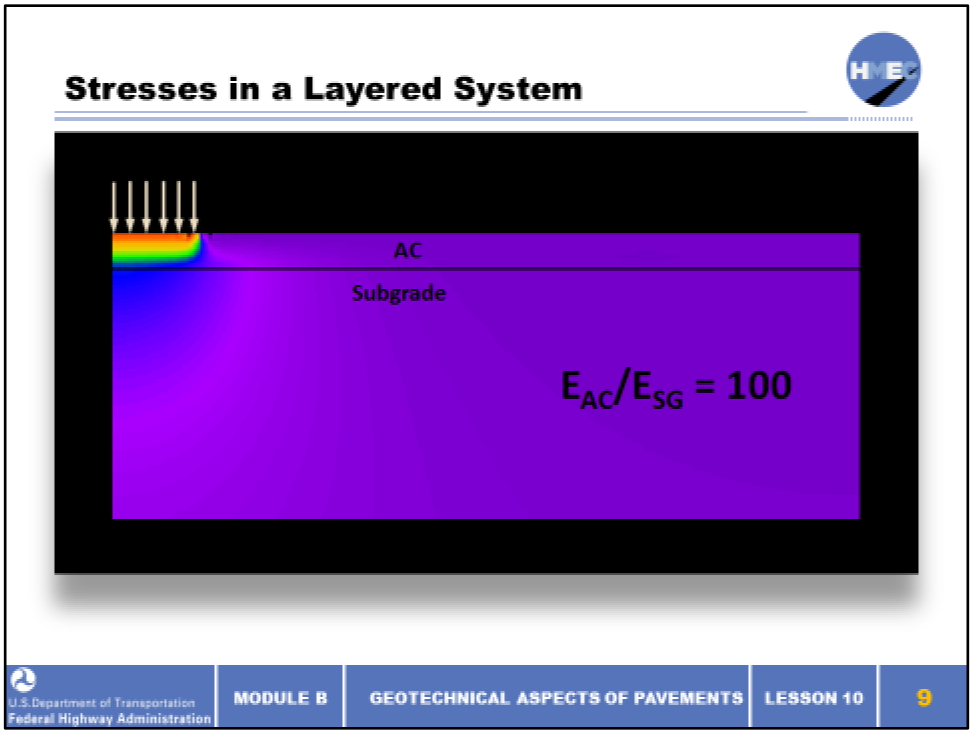
____

____

____

____

Slide 9



A modulus ratio of 100 is comparable to conditions for full depth flexible pavements. Now the stress bulb has moved up very significantly, and the higher stresses are contained in the AC.

____

____

____

____

____

____

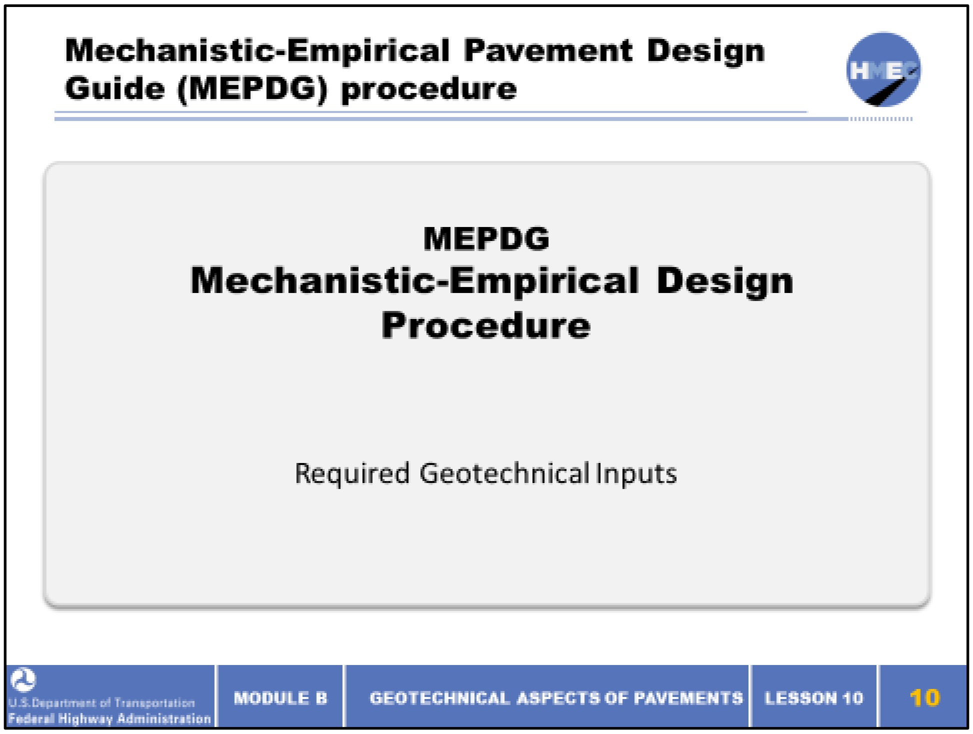
____

____

____

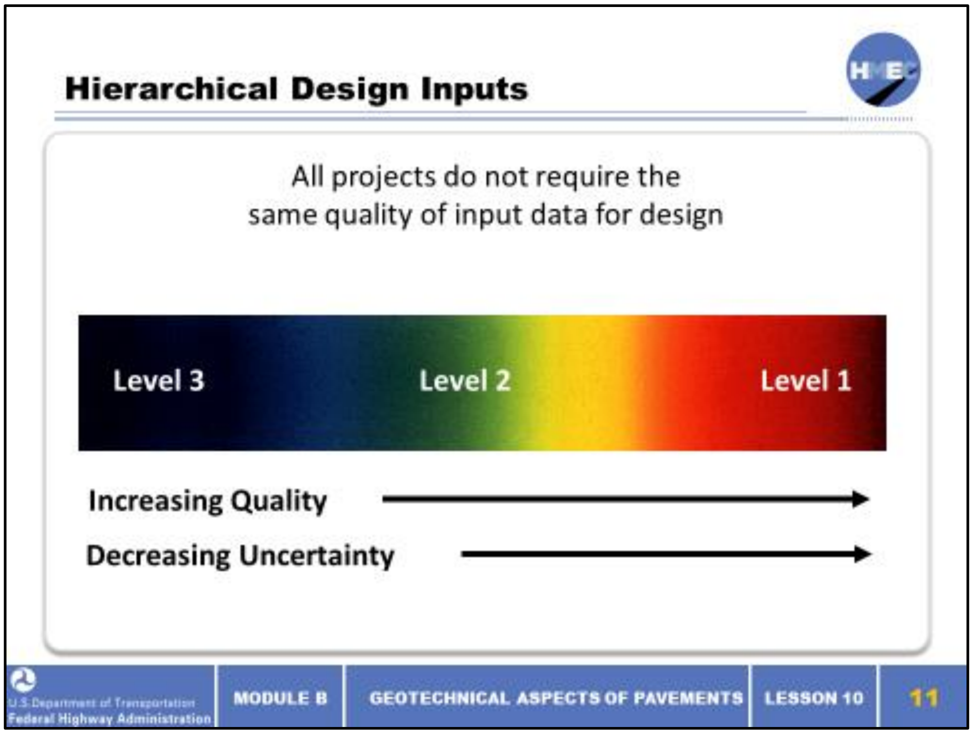
____

Slide 10



The mechanistic-empirical methodology that is the basis of the AASHTO MEPDG requires substantially more input information than needed by the empirical design procedures in the 1993 AASHTO guide.

Slide 11



The MEPDG formalizes the notion that larger, more expensive, and more important projects warrant more effort to develop high quality inputs than do smaller, less expensive, and comparatively unimportant designs.

Higher quality inputs are expected to produce higher quality designs because of reduced design uncertainty. Pavement engineers have always implicitly recognized this; the MEDPG simply makes the issue of input quality more explicit and visible.

____

____

____

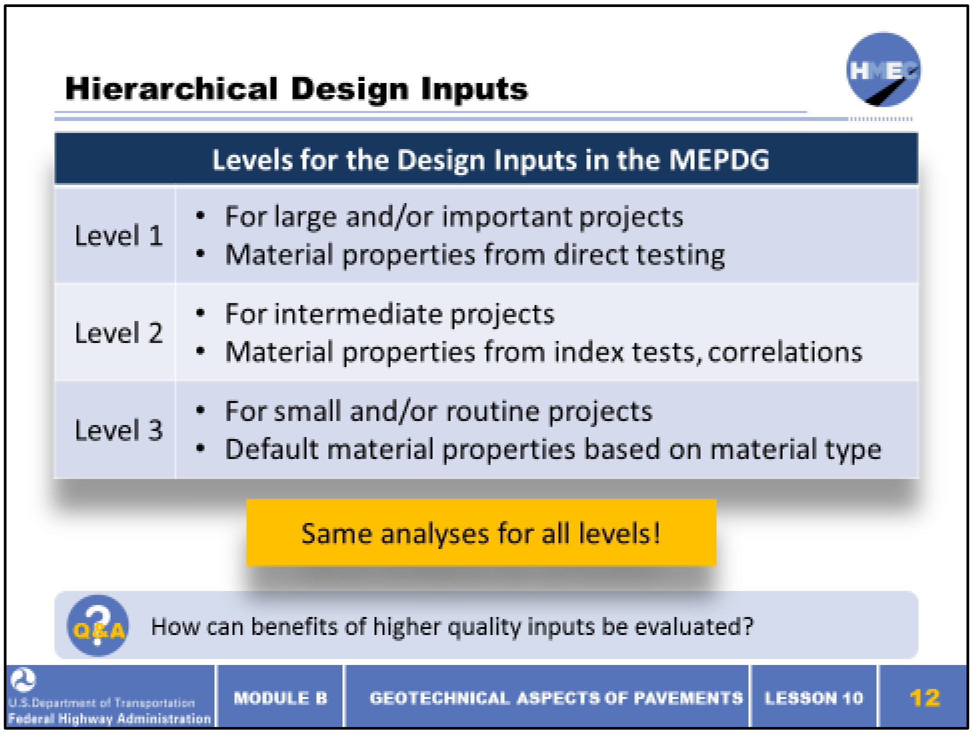
____

____

____

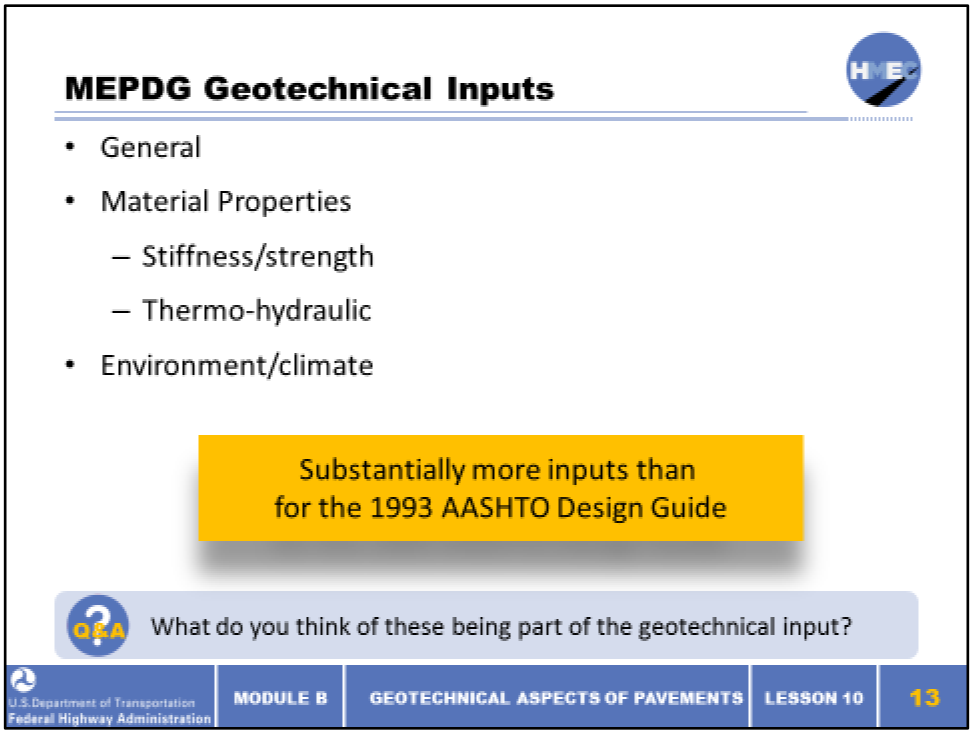
____

Slide 12



The same input parameters and the same analysis methods are used for all three levels. (Note: This latter point is not strictly true, as some of the analysis procedures change slightly for level 1 inputs—e.g., the use of finite element rather than layered elastic theory for the response analysis of flexible pavements.)

Slide 13



Environment/climate properties are included as "geotechnical" because (a) much of their effect is on geotechnical materials (moisture content, freeze/thaw) and (b) the geotechnical engineer in many agencies may be responsible for compiling these inputs.

Slide 14

General Inputs

Property	Description	Level		
		1	2	3
	Material type	✓	✓	✓
γ_t	In-situ total unit weight	✓	✓	✓
K_0	Coefficient of lateral earth pressure	✓	✓	✓

U.S. Department of Transportation Federal Highway Administration | MODULE B | GEOTECHNICAL ASPECTS OF PAVEMENTS | LESSON 10 | 14

There are three general properties needed for each of the three levels of design.

Slide 15

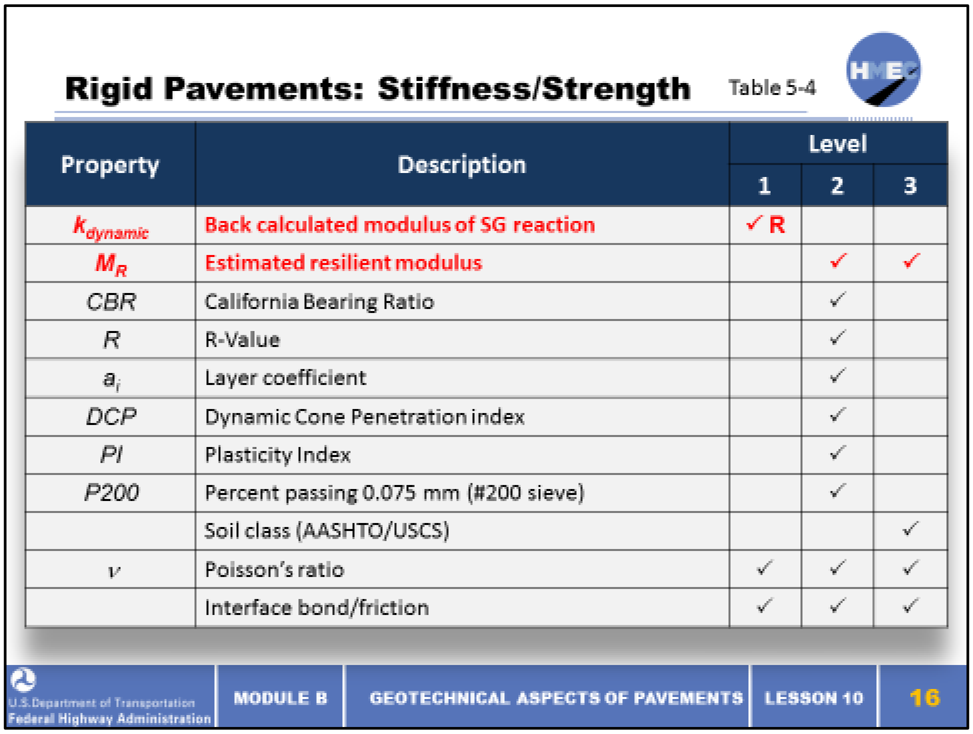
Flexible Pavements: Stiffness/Strength

Property	Description	Level 1	Level 2	Level 3
k_1, k_2, k_3	Nonlinear resilient modulus parameters	✓ N		
M_R	Back calculated resilient modulus	✓ R		
M_R	Estimated resilient modulus		✓	✓
CBR	California Bearing Ratio		✓	
R	R-value		✓	
a_i	Layer coefficient		✓	
DCP	Dynamic Cone Penetration index		✓	
PI	Plasticity Index		✓	
P200	Percent passing 0.075 mm (#200 sieve)		✓	
	Soil class (AASHTO/USCS)			✓
ν	Poisson's ratio	✓	✓	✓

U.S. Department of Transportation Federal Highway Administration | MODULE B | GEOTECHNICAL ASPECTS OF PAVEMENTS | LESSON 10 | 15

Not all properties listed here are required for all designs. Some properties substitute for others, depending upon the input hierarchical level and/or type of construction (e.g., CBR can substitute for M_R at level 2; level 1 M_R inputs are different for new/reconstruction versus rehab). N is for new pavement construction, R is for rehabilitation of existing pavement.

Slide 16

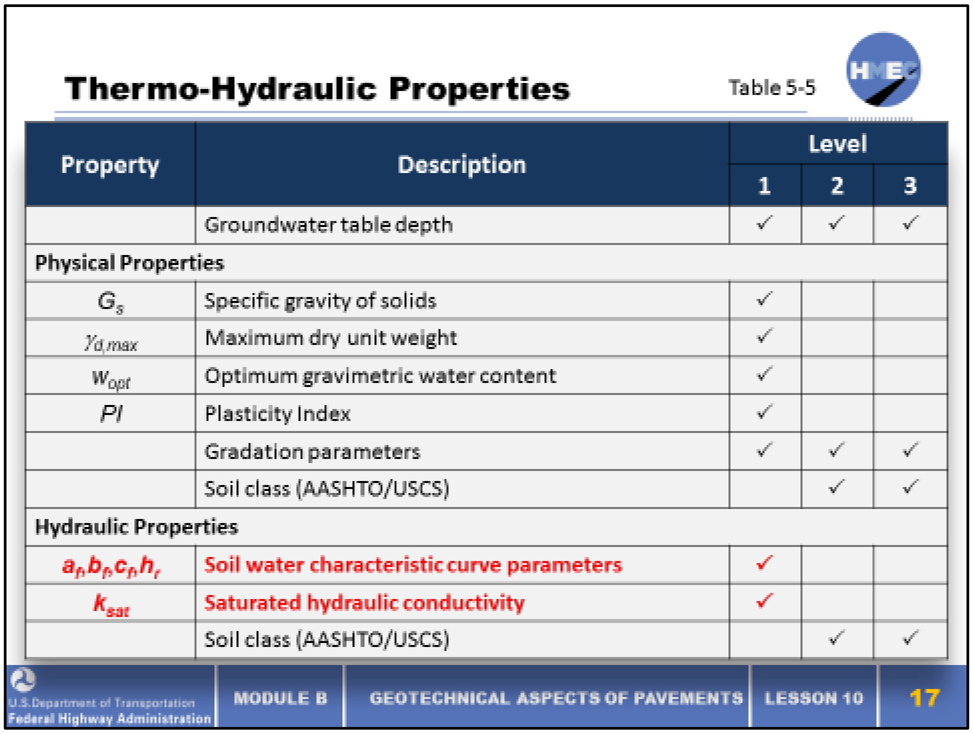


Rigid Pavements: Stiffness/Strength Table 5-4

Property	Description	Level 1	Level 2	Level 3
$k_{dynamic}$	Back calculated modulus of SG reaction	✓ R		
M_R	Estimated resilient modulus		✓	✓
CBR	California Bearing Ratio		✓	
R	R-Value		✓	
a_i	Layer coefficient		✓	
DCP	Dynamic Cone Penetration index		✓	
PI	Plasticity Index		✓	
P200	Percent passing 0.075 mm (#200 sieve)		✓	
	Soil class (AASHTO/USCS)			✓
ν	Poisson's ratio	✓	✓	✓
	Interface bond/friction	✓	✓	✓

Many rigid pavement material property inputs are similar to those for flexible pavements. This is intentional; the MEPDG tries to treat rigid and flexible pavements as similarly as possible. R stands for rehabilitation of existing pavement.

Slide 17



Thermo-Hydraulic Properties

Table 5-5

Property	Description	Level 1	Level 2	Level 3
	Groundwater table depth	✓	✓	✓
Physical Properties				
G_s	Specific gravity of solids	✓		
$\gamma_{d,max}$	Maximum dry unit weight	✓		
w_{opt}	Optimum gravimetric water content	✓		
PI	Plasticity Index	✓		
	Gradation parameters	✓	✓	✓
	Soil class (AASHTO/USCS)		✓	✓
Hydraulic Properties				
a_f, b_f, c_f, h_r	Soil water characteristic curve parameters	✓		
k_{sat}	Saturated hydraulic conductivity	✓		
	Soil class (AASHTO/USCS)		✓	✓

Thermo-hydraulic properties (except for saturated hydraulic conductivity/permeability) will be unfamiliar to many geotechnical and pavement engineers.

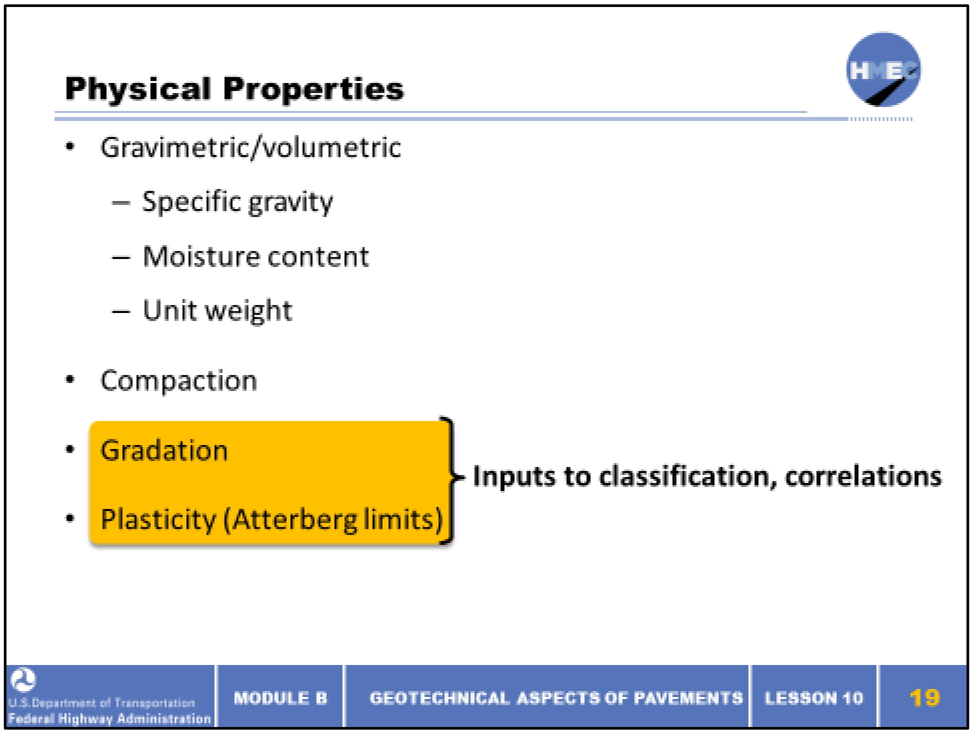
Slide 18

Material Properties

- Physical
- Mechanical
- Thermo-hydraulic
- Distress model

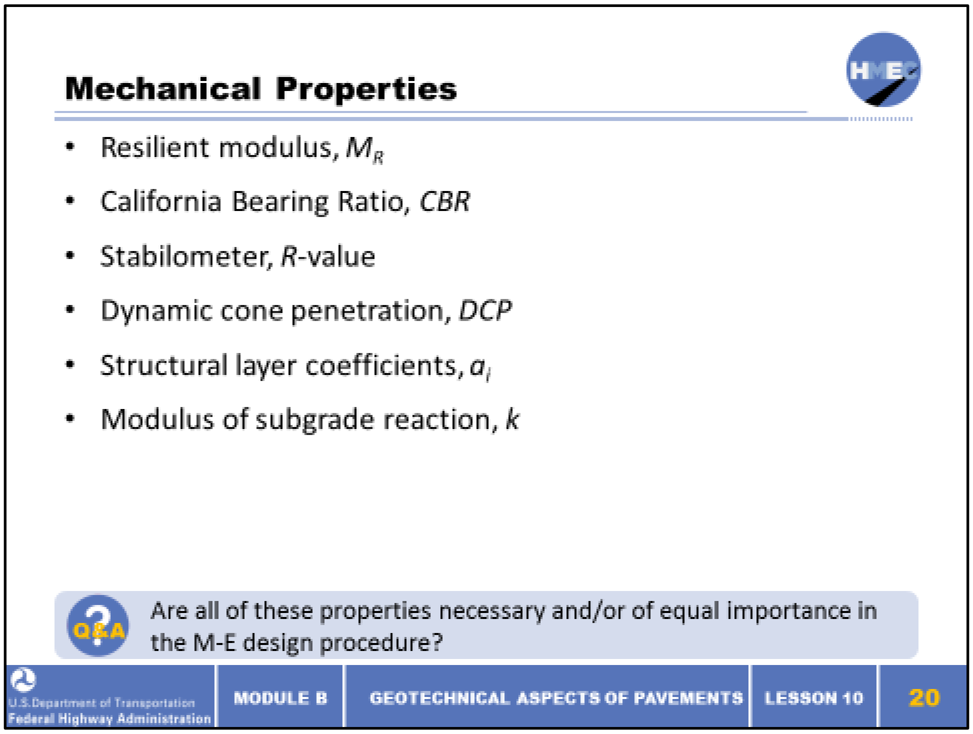
U.S. Department of Transportation Federal Highway Administration | MODULE B | GEOTECHNICAL ASPECTS OF PAVEMENTS | LESSON 10 | 18

Slide 19



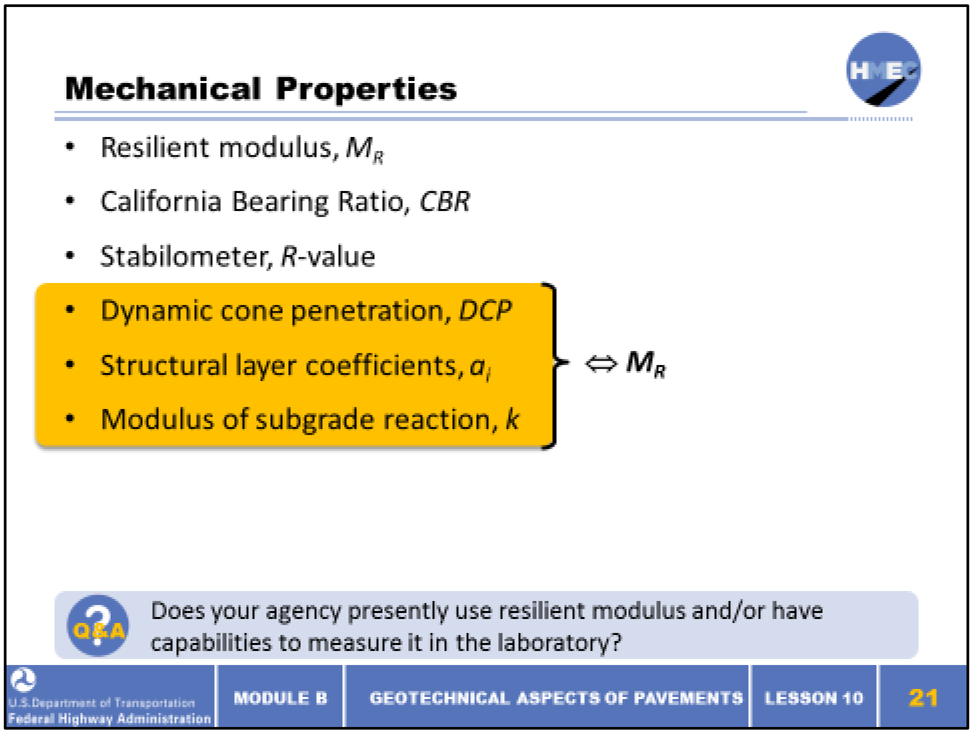
Physical properties provide the most basic description of unbound materials. These properties are also often used in correlations for more fundamental engineering properties, such as stiffness or permeability. The principal physical properties of interest are specific gravity of solids, water content, unit weight (density), gradation characteristics, plasticity (Atterberg limits), classification, and compaction characteristics.

Slide 20



The relative stiffness of the various layers dictate the distribution of stresses and strains within the pavement system. There are many mechanical properties for quantifying the strength or stiffness of the materials.

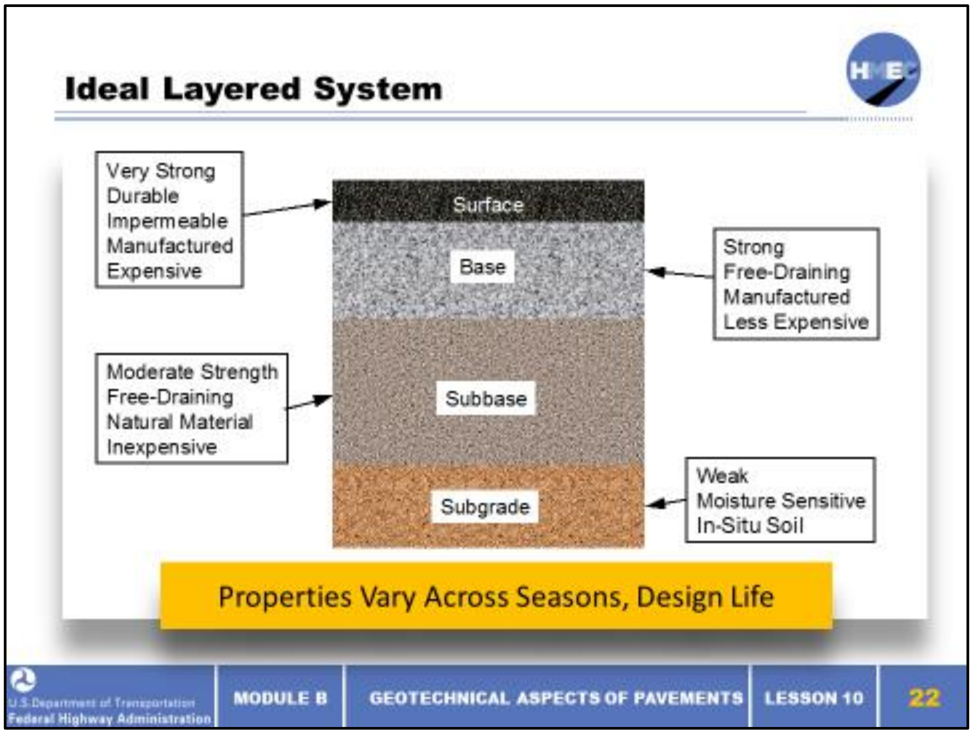
Slide 21



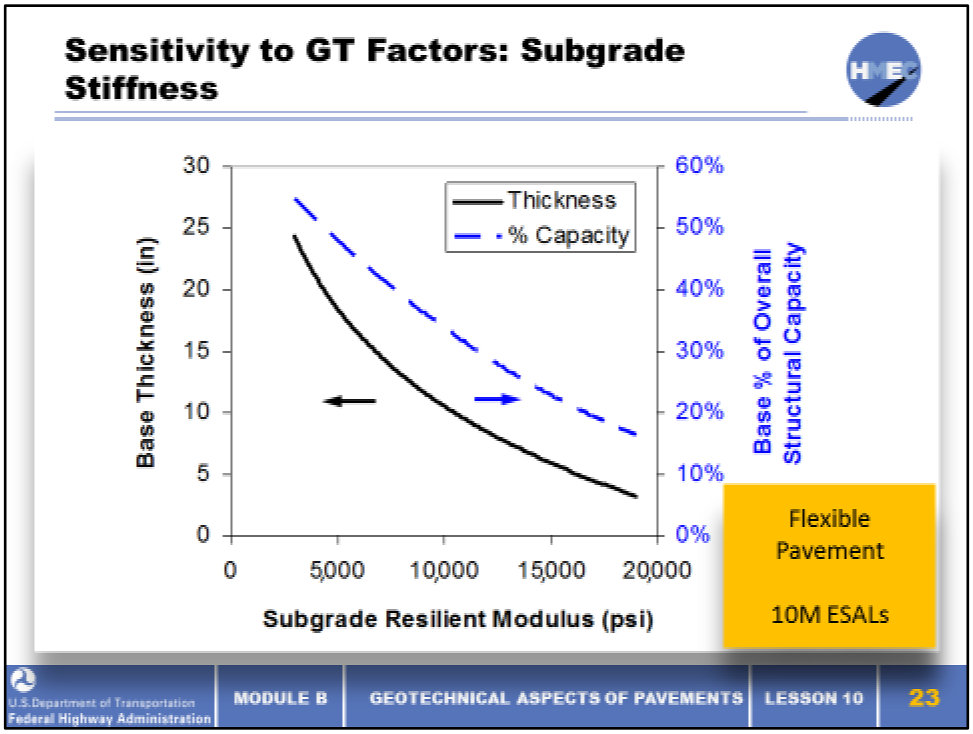
The preferred method for characterizing the stiffness of unbound pavement materials is the resilient modulus M_R, which is defined as the unloading modulus in cyclic loading.

The MEPDG procedure recognizes the need for backward compatibility with other properties used in the past to characterize unbound materials, in particular the California Bearing Ratio and the stabilometer R-value. These index material properties continue to be used by many highway agencies. Correlations are provided in both design procedures for relating CBR and R-values to M_R.

Slide 22


Ideal Layered System
Very Strong
Durable
Impermeable
Manufactured
Expensive
Surface
Base
Strong
Free-Draining
Manufactured
Less Expensive
Moderate Strength
Free-Draining
Natural Material
Inexpensive
Subbase
Subgrade
Weak
Moisture Sensitive
In-Situ Soil
Properties Vary Across Seasons, Design Life
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
22

Slide 23



Observations:

• Required structure increases sharply for very poor subgrade conditions (very low subgrade CBR).

• Unbound granular base (i.e., "geotechnical" materials) may provide between 20% to 50% of overall structural capacity for a flexible pavement.

____

____

____

____

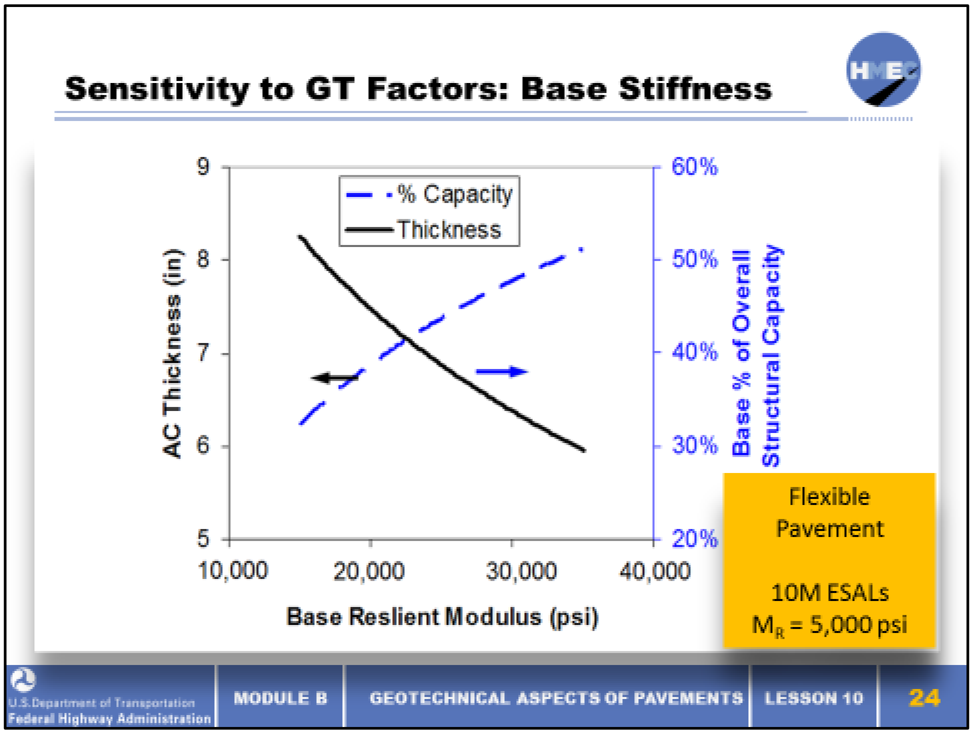
____

____

____

____

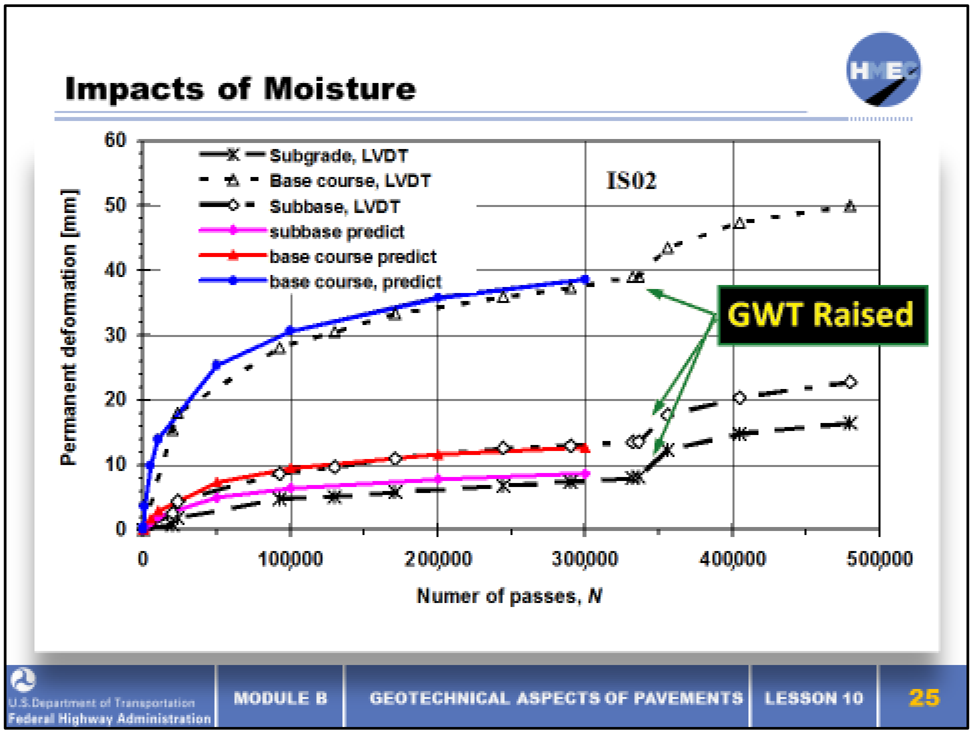
Slide 24



Quality of geotechnical materials (e.g., unbound granular base) has significant effect on required thickness for surface layer.

Again, unbound granular base may provide up to 50% of overall structural capacity for a flexible pavement.

Slide 25



This plot presents findings from a field test of a lightly surfaced flexible pavement in Iceland. The water table was raised to 30 cm below the top of sand subgrade after 330,000 load cycles. This caused a sharp increase in rate of permanent deformation of unbound materials. Moisture/water significantly affects pavement performance/life.

____

____

____

____

____

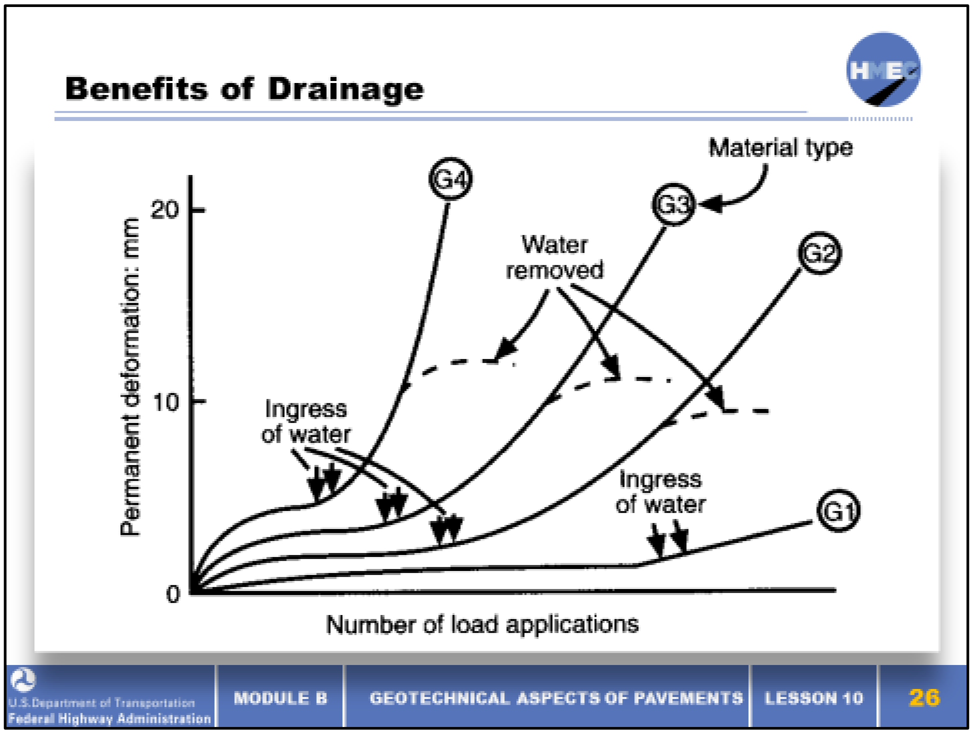
____

____

____

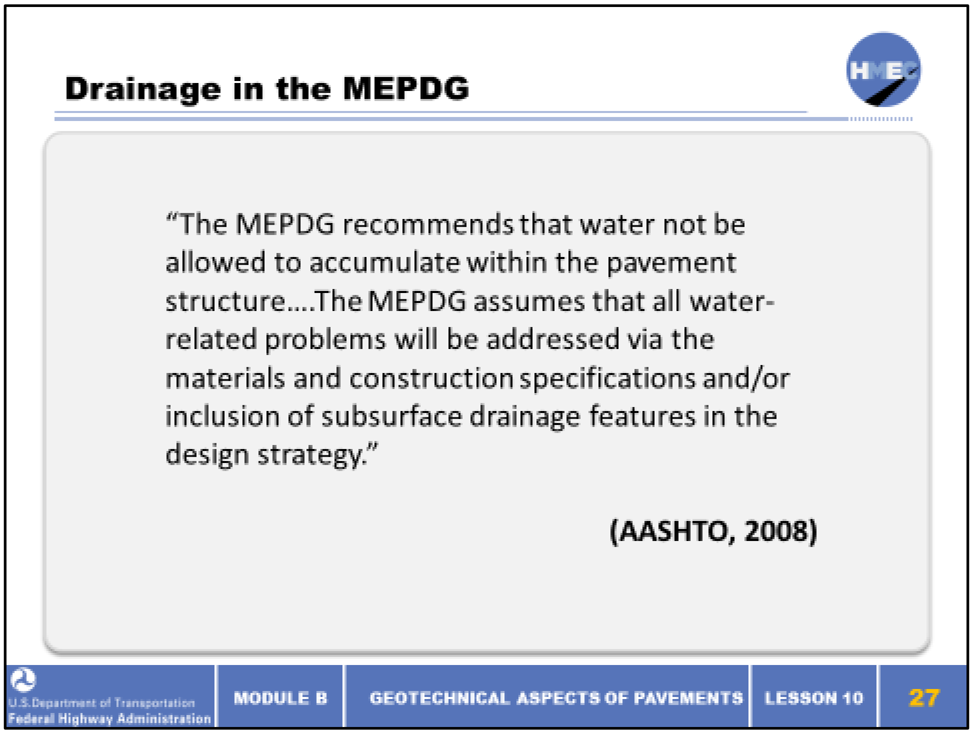
____

Slide 26



Several different base materials of different qualities (G1 = best, G4 = worst) are shown in this plot. Perforated pavement surfaces in this study allowed rapid ingress of water upon flooding of surface; this caused a sharp increase in rate of permanent deformation, particularly for the lower quality base materials. Drainage of water (dashed curves) arrested the rate of permanent deformation increase.

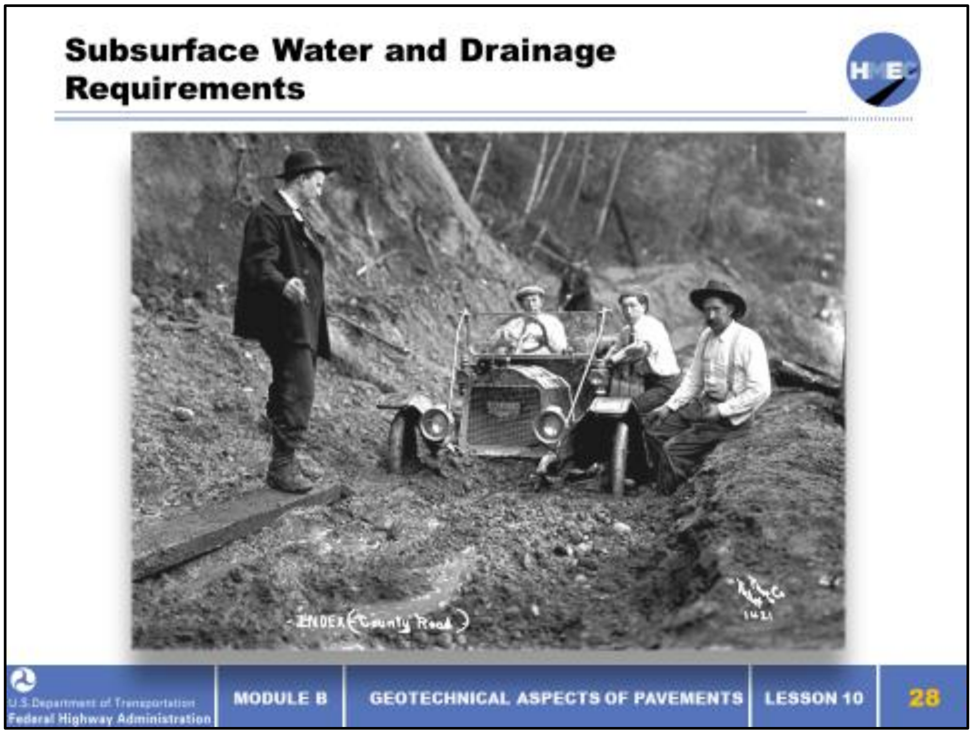
Slide 27



Currently, the MEPDG recommends that water not be allowed to accumulate within the pavement structure. The MEPDG assumes that all water-related problems will be addressed via the materials and construction specifications and/or inclusion of subsurface drainage features in the design strategy. (You may let participants read this on their own instead of reading it to them.)

Obviously, this assumption is a big limitation in the current version of the MEPDG. The expectation is that the capability to factor in water in the pavement structure will be restored in future versions.

Slide 28



We've known about drainage for a long time. Even the Romans knew the importance of drainage, and this is why some of their roads still exist today. In more modern times, the origins of drainage design is tied to problems with farm-to-market roads in Iowa in the 1920s. A young Iowa State professor (Spangler) started emphasizing drainage and developed early drainage design procedures.

____

____

____

____

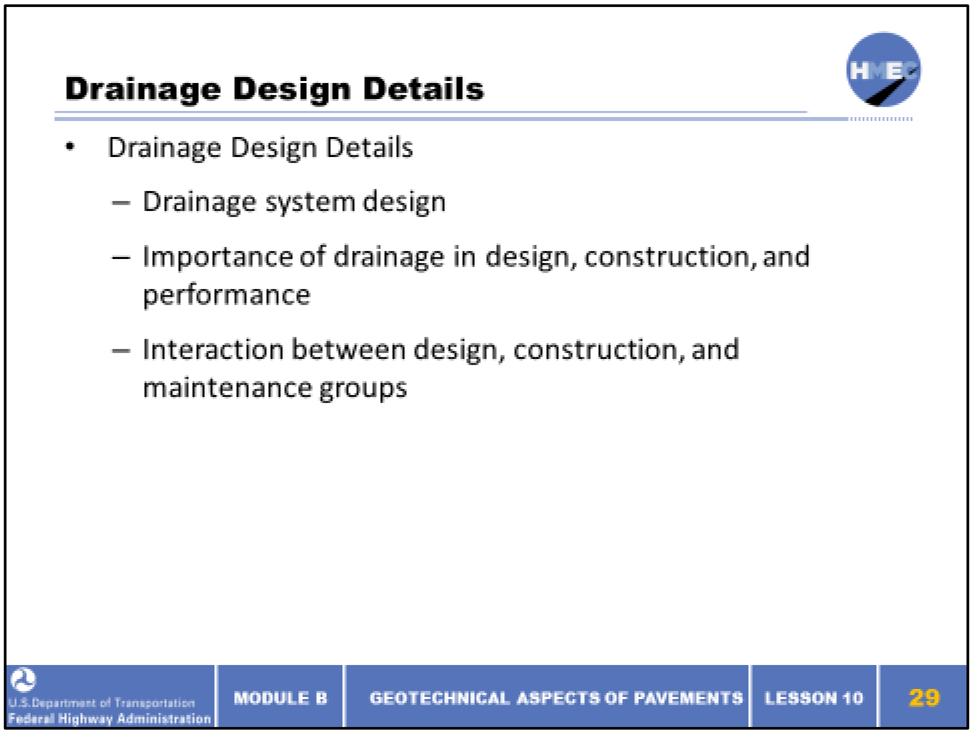
____

____

____

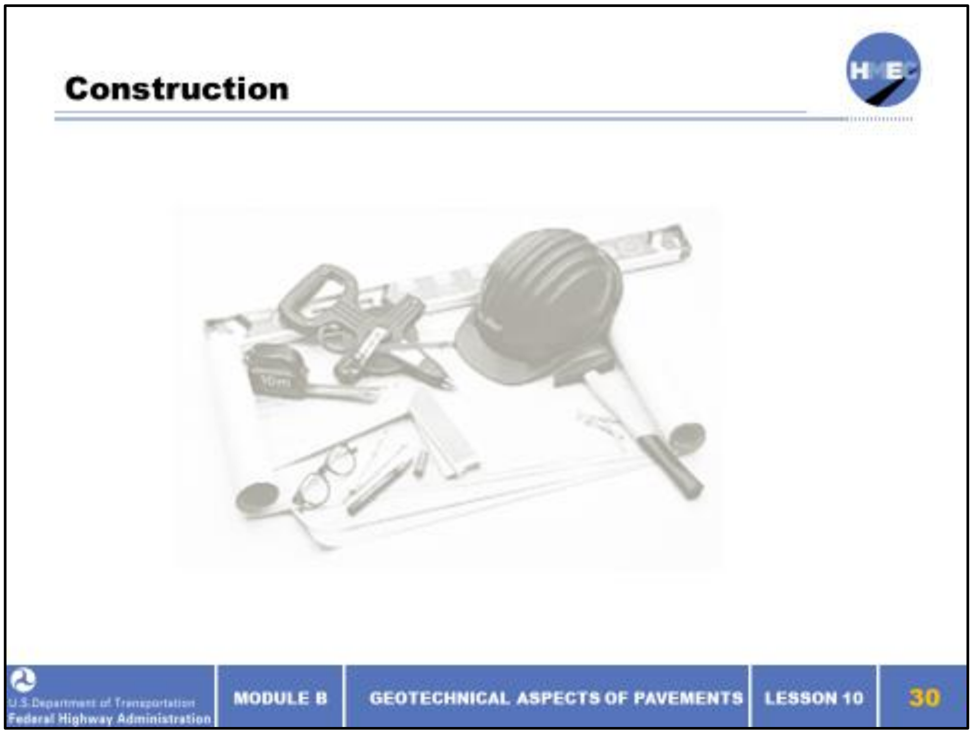
____

Slide 29

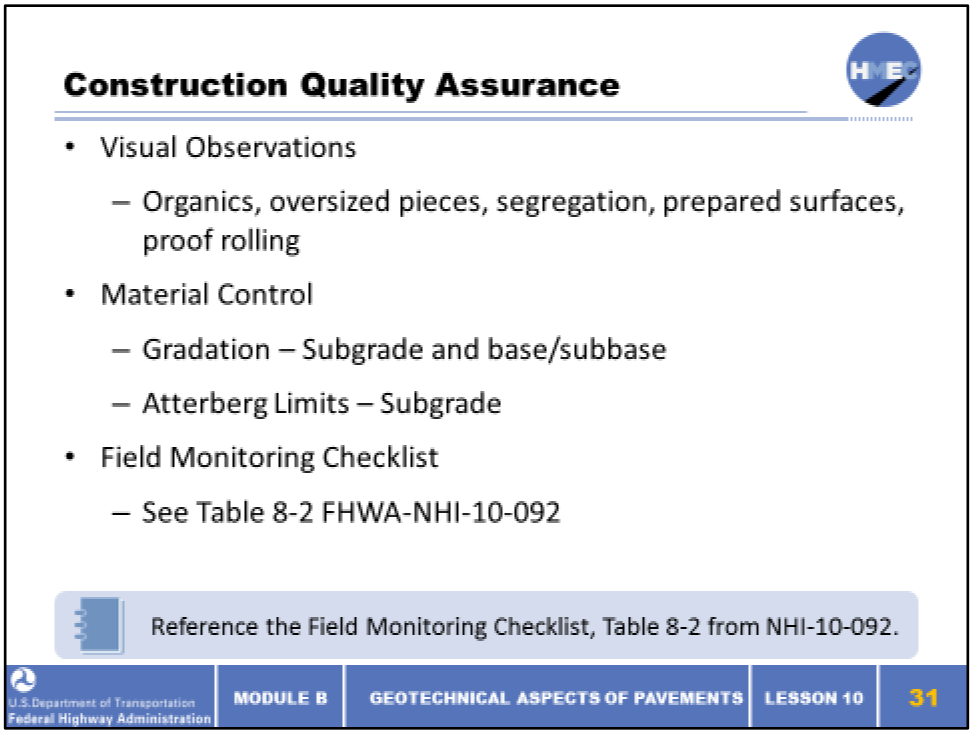


Drainage of pavements requires interaction/coordination between design, construction, and maintenance.

Slide 30


Construction
HMEC
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
30

Slide 31

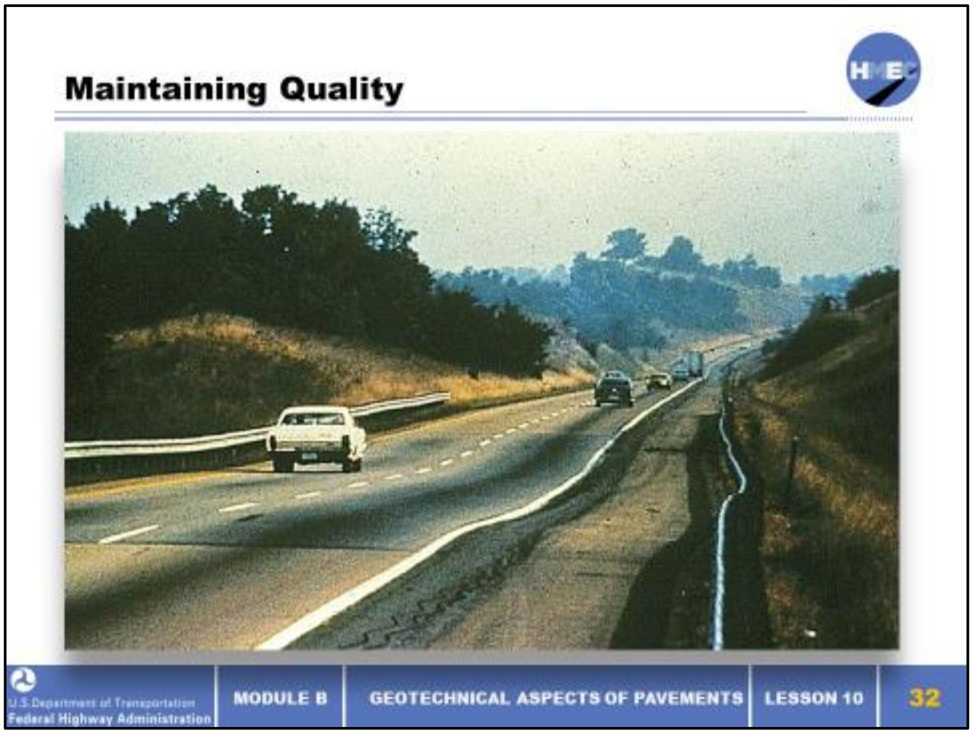


The agency and the contractor will use visual observations and tests to ensure quality. Visual observations of materials and of subgrade strength are typically used. The gradation of base and subbase materials are normally tested for conformance to specifications. Atterberg limit measures are used to check fine-grained subgrades. Most agencies have a standardized quality assurance procedures and checklists.

Table 8-2. Field Monitoring Checklist

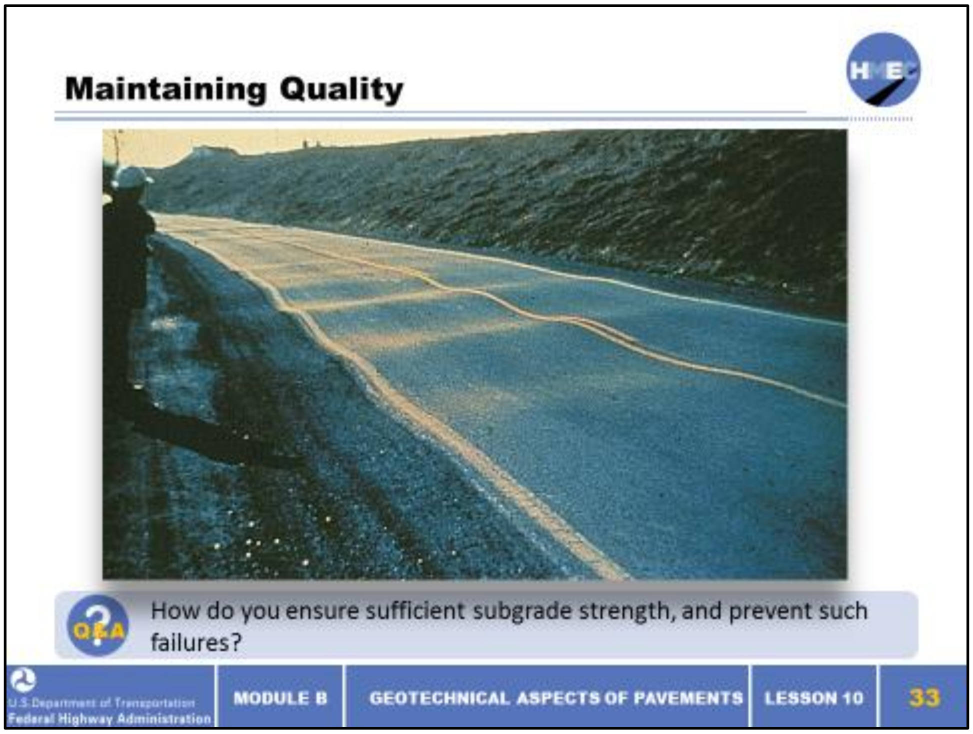
1. Read the specifications and become familiar with - site preparation - material requirements - construction procedures - grade/slope requirements - drainage requirements - tolerances of each of the above requirements - testing requirements - acceptance/rejection criteria
2. Review the construction plans and become familiar with - lines, grades, and layer thickness requirements - temporary and permanent drainage features, locations, and details - details for utility construction, special requirements - demolition (if rehabilitation or reconstruction project) - corrective action requirements for weak, yielding, unstable materials (undercut/replace, chemical stabilization, geosynthetics) - construction sequence
3. Review material requirements, equipment requirements, and approved submittals.
4. Check site conditions. - observe clearing, grubbing requirements, and activities – document the final condition in accordance with the specification - observe haul patterns - observe response to load (deflection under heavy equipment traffic) - assess need for drainage system - perform foundation acceptance testing as required (*e.g.,* proof rolling, DCP, etc.)
5. On site monitoring and testing. - observe and document hauling, placement, and spreading adequacy (segregation, loose lift thickness, moisture content reasonableness) - review truck tickets and ensure that materials are from approved sources (generally for engineered products such as aggregate base or flowable fill, bedding material, and off-site borrow, etc.) - randomly sample for material compliance - assess moisture content reasonableness (not overly wet or dry in conformance with the specification) - document compaction efforts (size and type of equipment, number of passes) - test for compaction (determine conformance to specifications with respect to density and moisture – Proctor check point, if necessary) and assess stability - non-conformance corrective action – proof roll and assess extent/depth of affected area; determine and document responsibility (owner vs. G.C.) and begin corrective action - measure and document stabilizing agent type and rate of application, mixing adequacy and depth, compactive effort, moisture reasonableness and compaction compliance results (for stabilization/modification layers or treatments). Sampling and testing of treated layers and determination of strength in the laboratory may also be required.

Slide 32



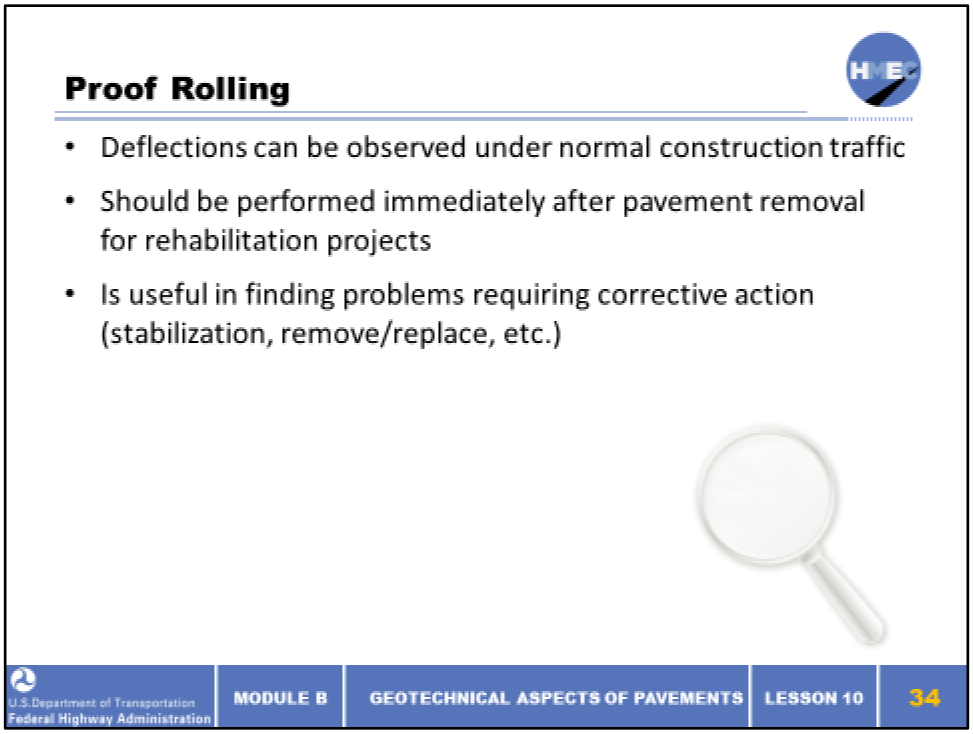
This is an old photo, but is a good illustration of a weak subgrade in a fill area. The better, stronger subgrade support on more competent materials in the cut section is obvious. This becomes a maintenance problem, and reflects poorly on the contracting agency.

Slide 33



This rolling pavement is due to subgrade variability and failure.

Slide 34



The main problem with proof rolling is that it is very subjective. More modern methods are available. But proof rolling is better than no check.

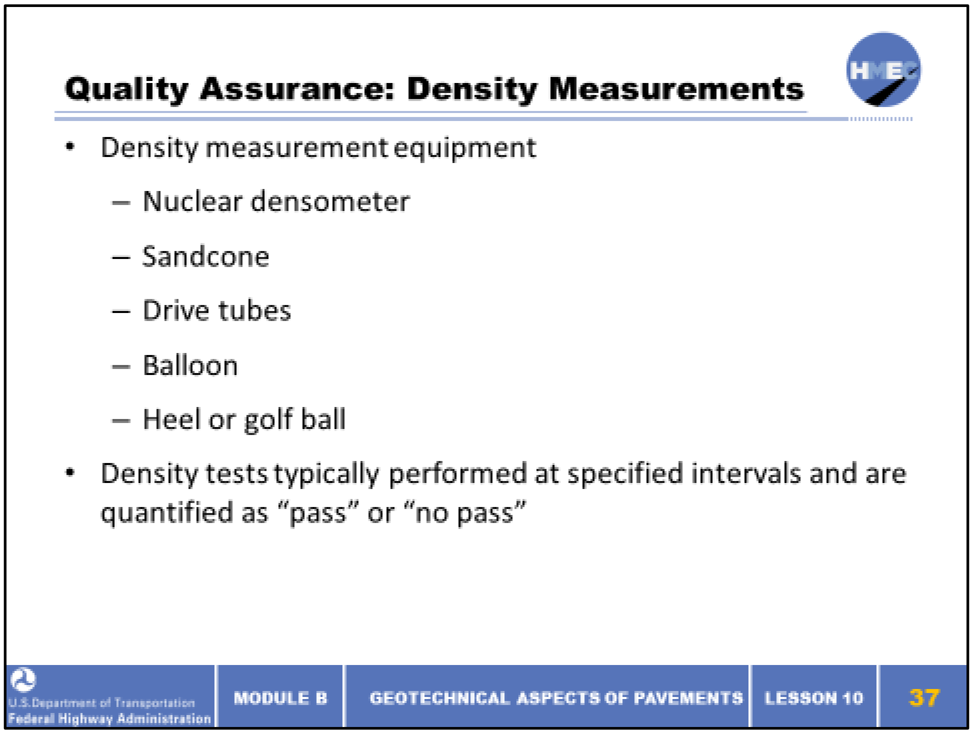
Slide 35

Another proof rolling problem is the standardization of equipment. Different vehicles, different weights allows for identification of soft versus stiff zones, but cannot confirm overall adequacy.

Slide 36

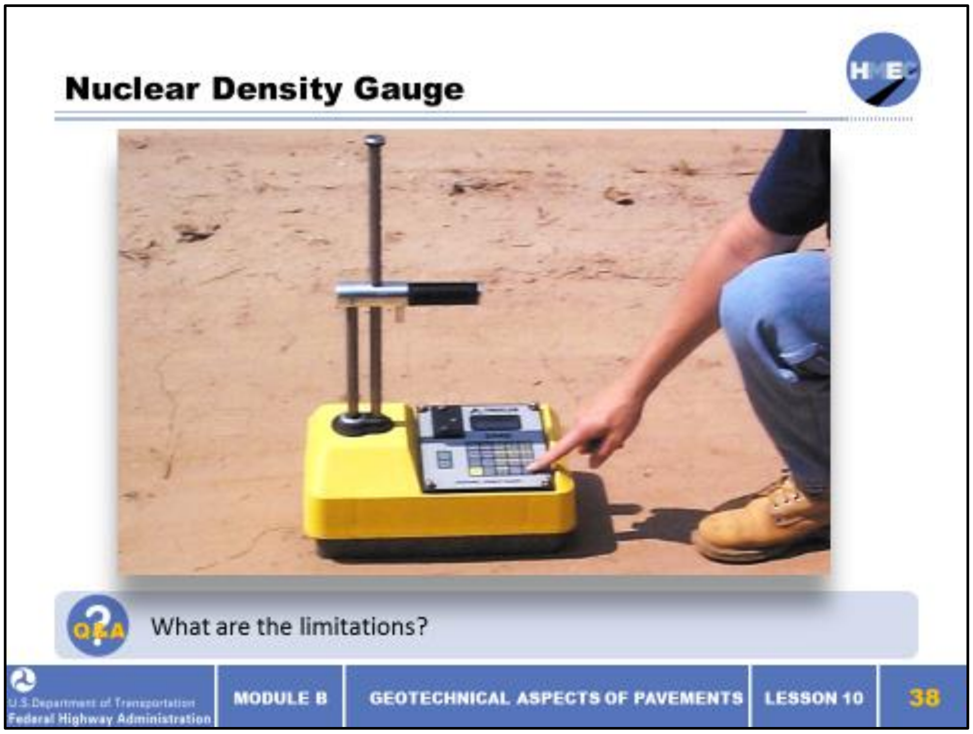
This is a standardized vehicle—an Ohio DOT standard proof roller. This does improve technique a bit.

Slide 37



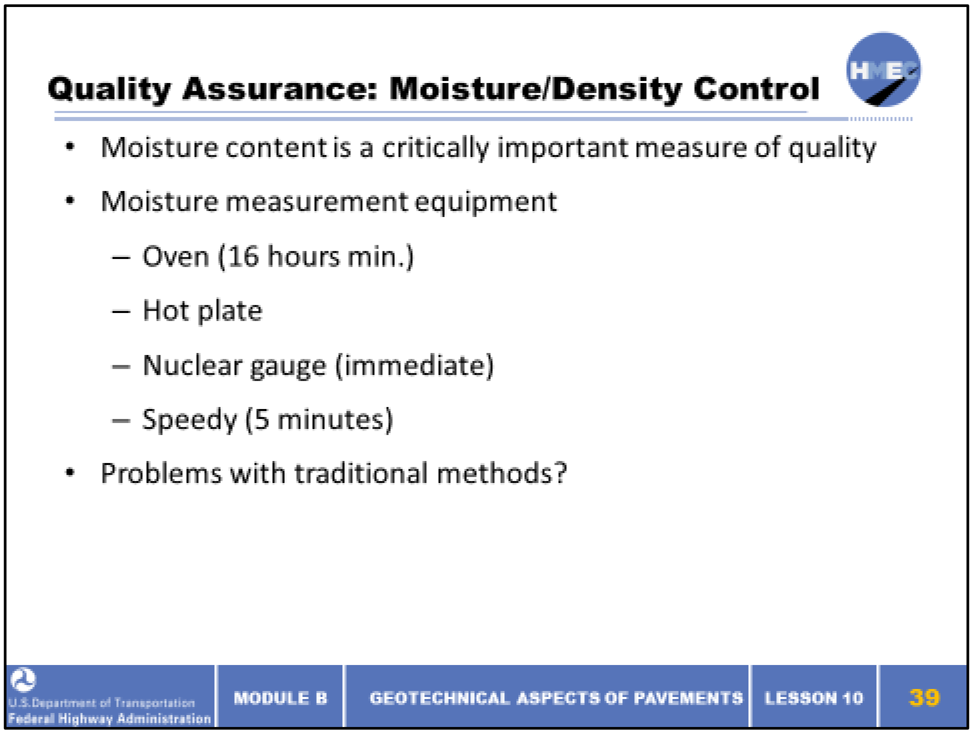
Note that today, more agencies are placing more of an emphasis on statistical characterization of constructed materials—not just pass or no pass.

Slide 38



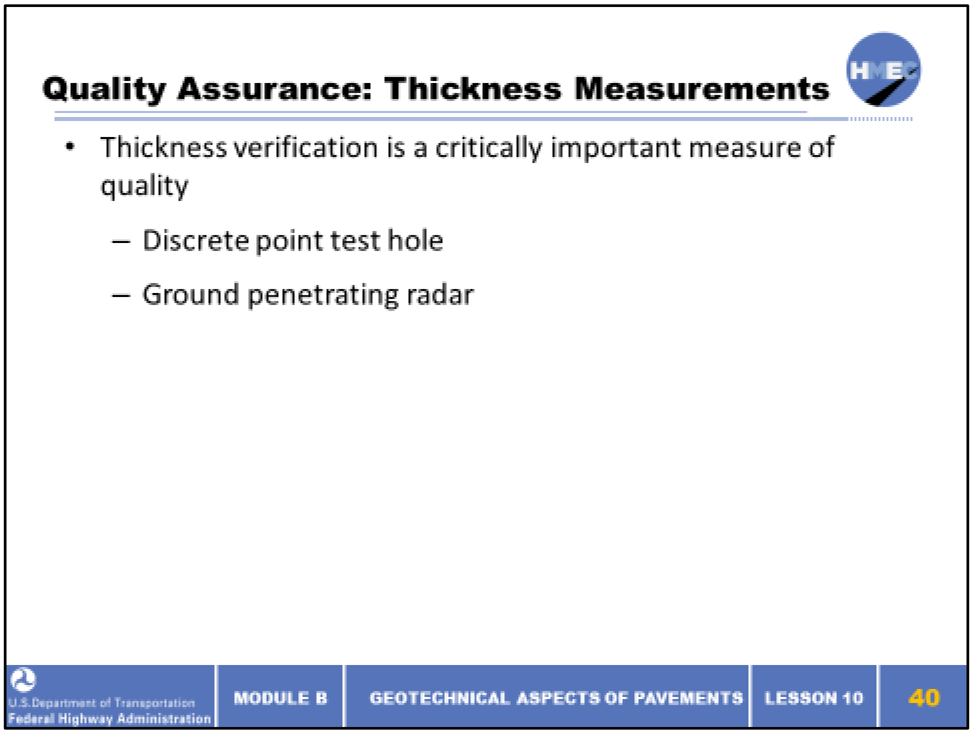
Example of density measuring equipment.

Slide 39



One practical problem: Moisture content changes over time. Traditional methods are point estimates only.

Slide 40



Layer thickness is a critical parameter in pavement design.

Conventional thickness measurement is also only a point estimate.

Ground penetrating radar (GPR) provides a complementary method to conventional thickness measurements and can identify variations in thickness.

____

____

____

____

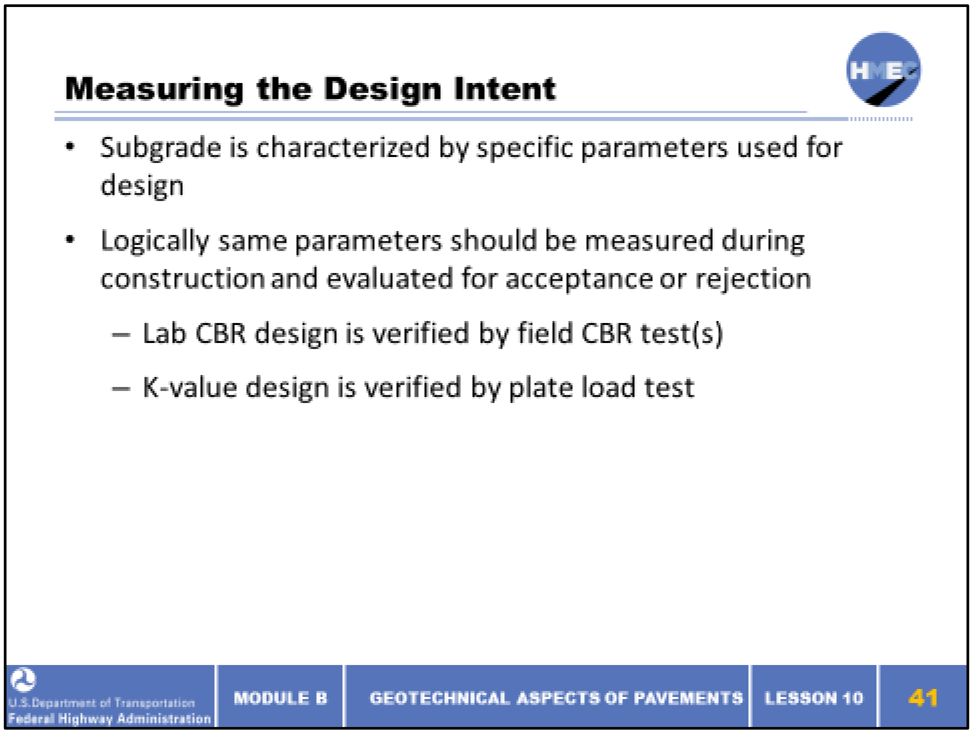
____

____

____

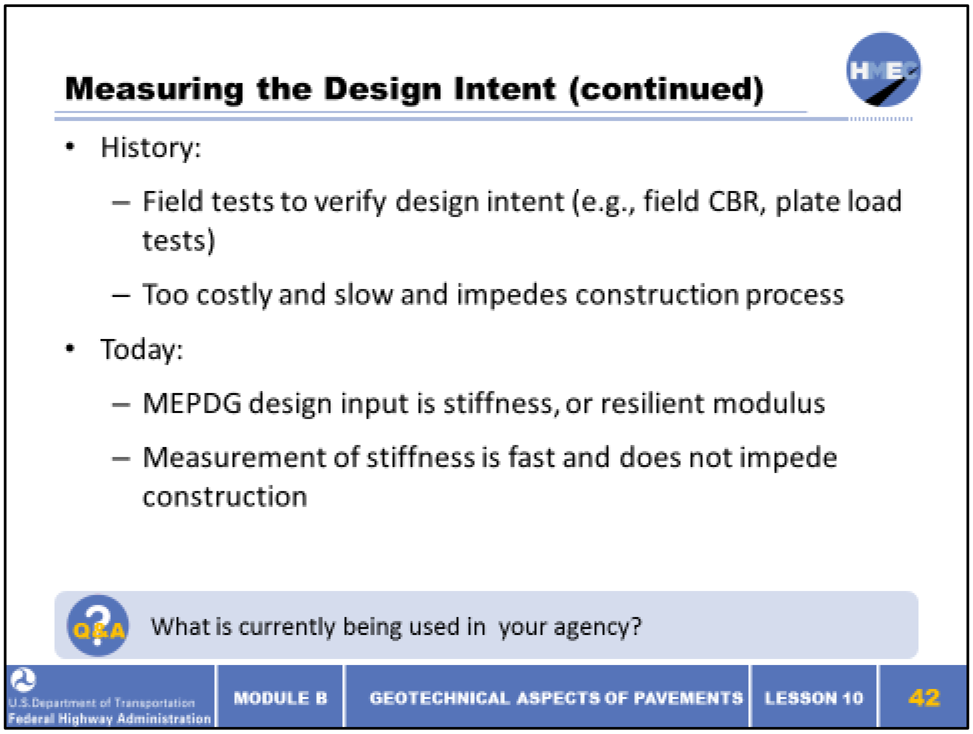
____

Slide 41



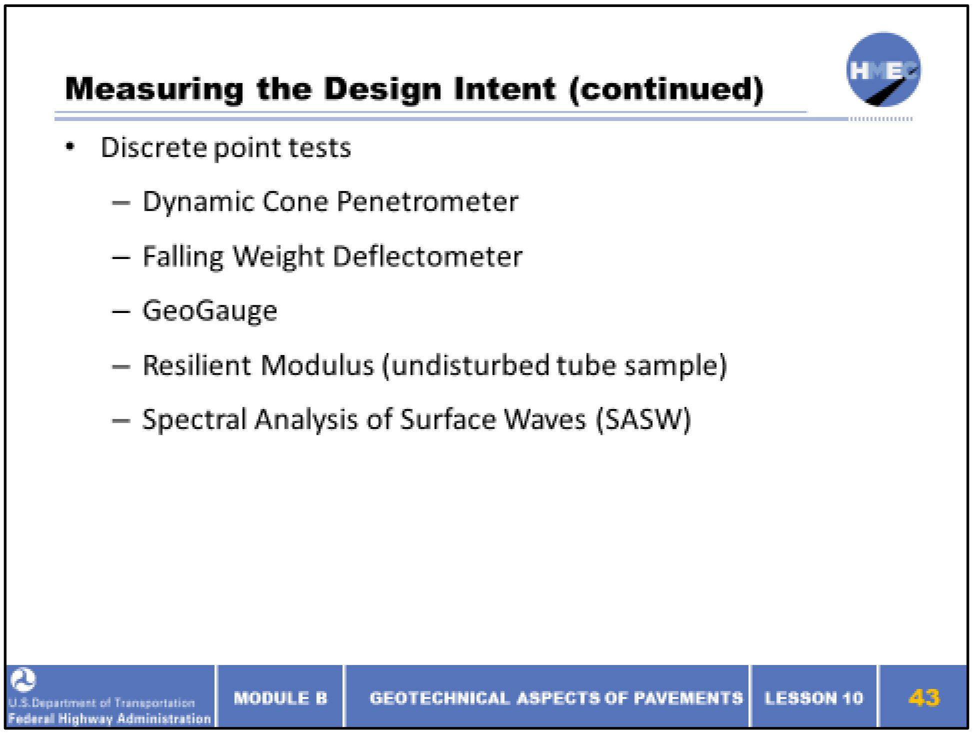
We use specific subgrade parameters (ex. M_R) for design of a pavement. But generally we are measuring density and moisture in the field. Do we know that the design parameter is being met? Not really, we can use some correlations, but certainly it is more advantageous to measure the actual design parameter.

Slide 42



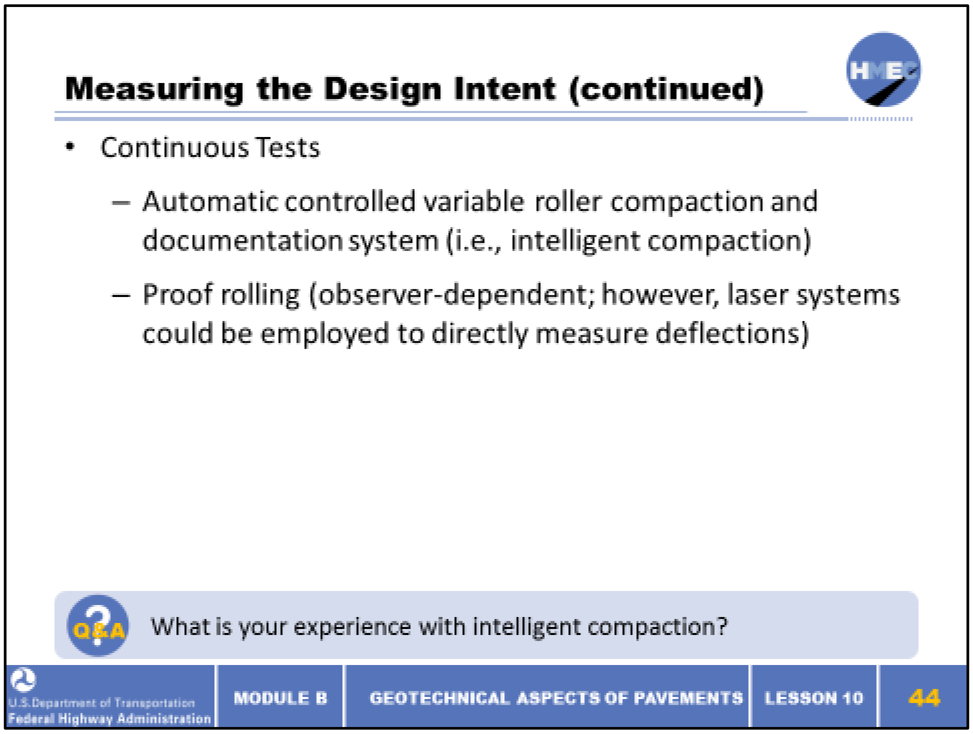
Past, present, and future of measuring the design intent. Today, the measurement of stiffness or resilient modulus is at FHWA.

Slide 43



Here are tests that are used to provide direct measurement of design properties.

Slide 44

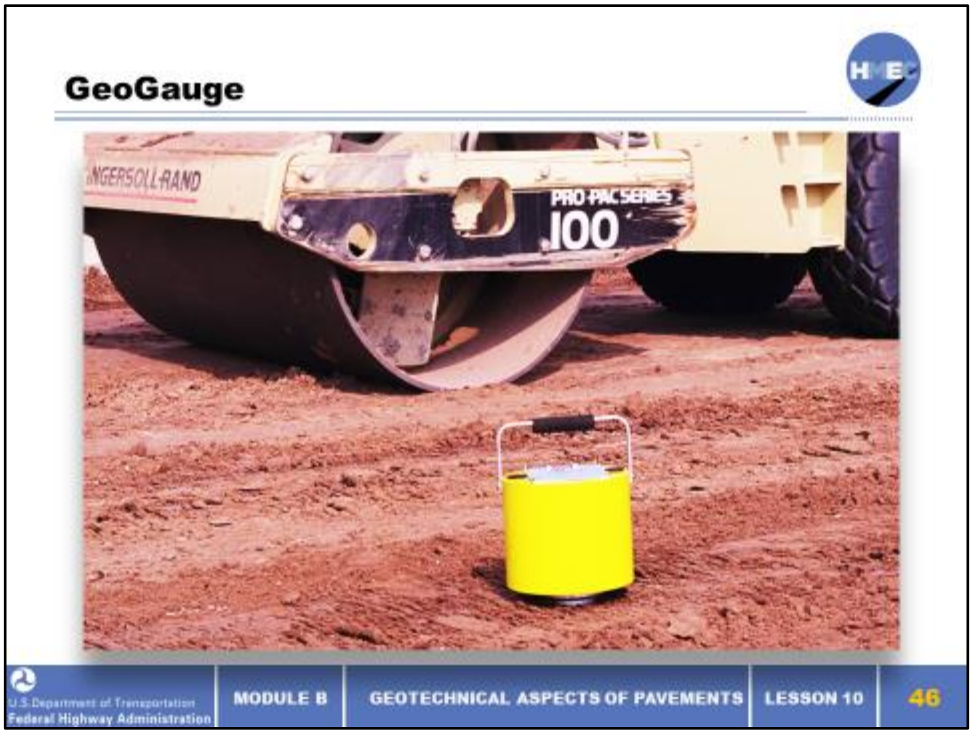


Proof rolling with laser systems is very simple, inexpensive, and it removes the subjective nature of measurements. Several States have or are now evaluating this.

Slide 45

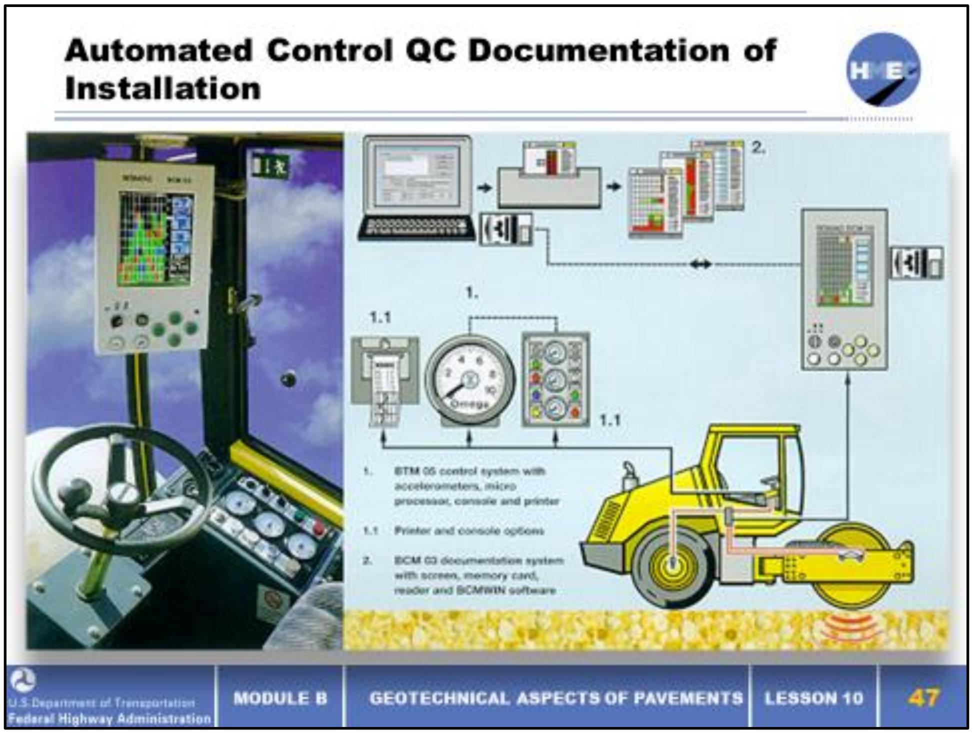
The use of a lightweight FWD suitable for testing subgrades and base layers provides a point measurement. The operation can be somewhat subjective with the raising and release of the weight. This is better controlled with trigger weight releases. Note that trailer-mounted FWD are also available and used.

Slide 46



This is another point method. The GeoGauge was developed by FHWA.

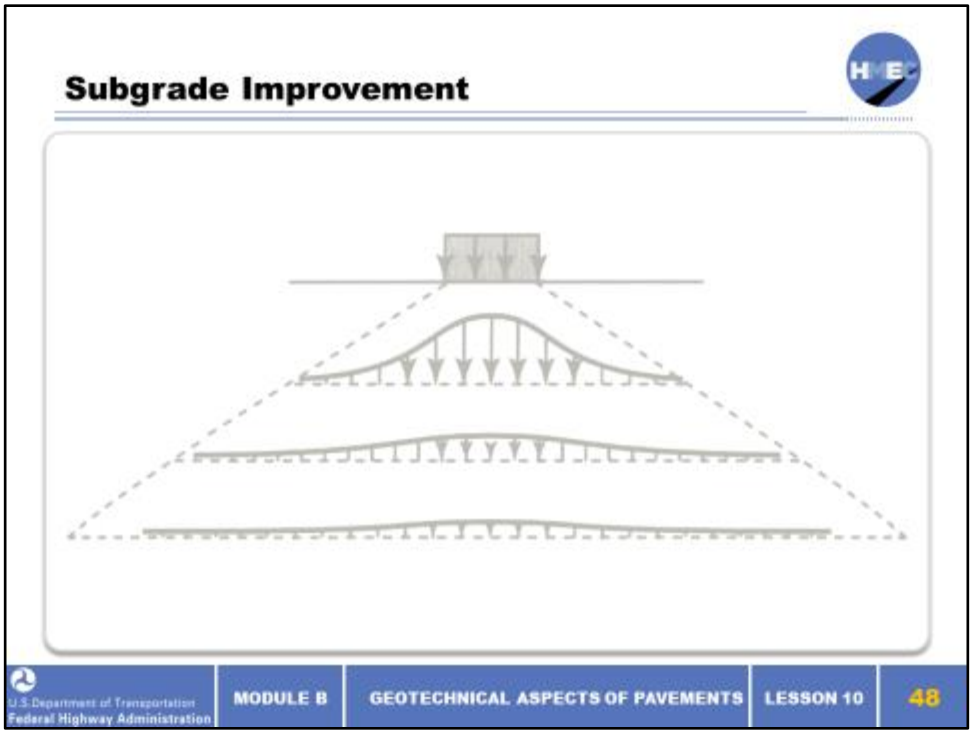
Slide 47



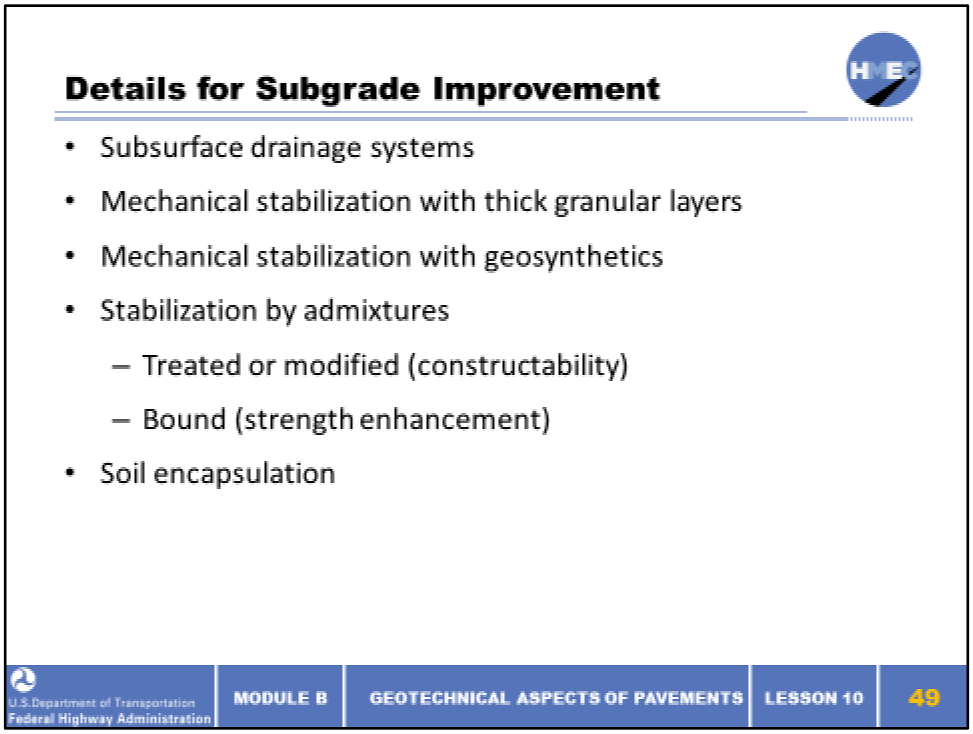
Processors are used to measure interaction of drum-soil. This provides instant feedback and roller operator has a map of area being compacted. Map is color coded to indicate where design compaction level has been met, and where additional effort is required. Current intelligent compaction systems work very well for asphalt and well for base layers, but more work is still required for subgrade compaction applications (moisture variability issues).

____

Slide 48

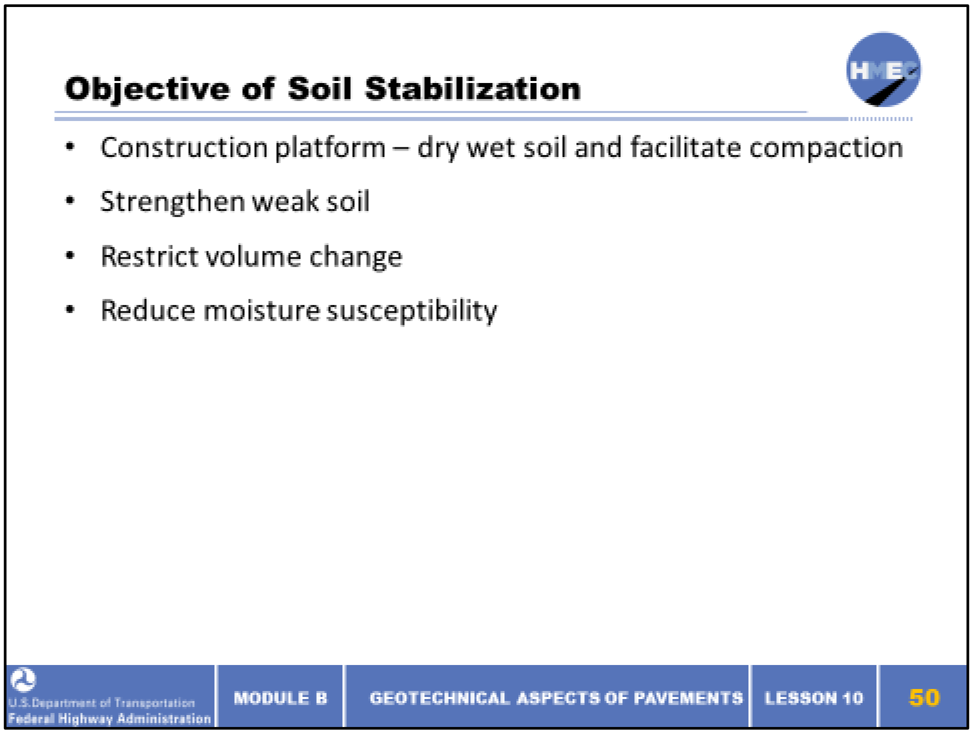

Subgrade Improvement
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
48

Slide 49



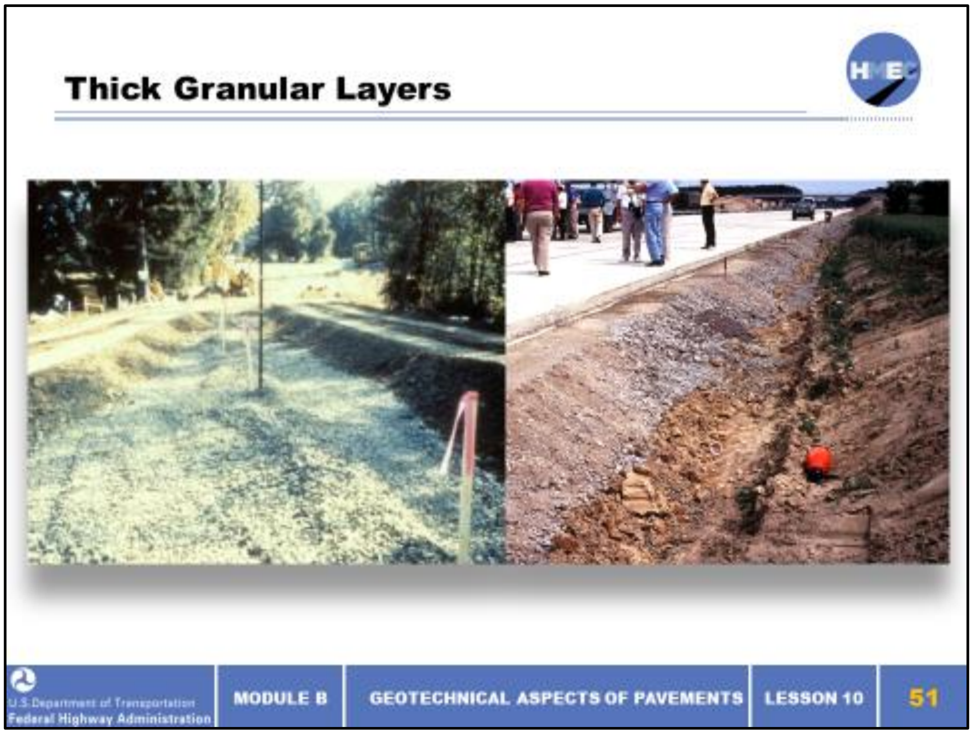
There are several options for improving a pavement subgrade.

Slide 50



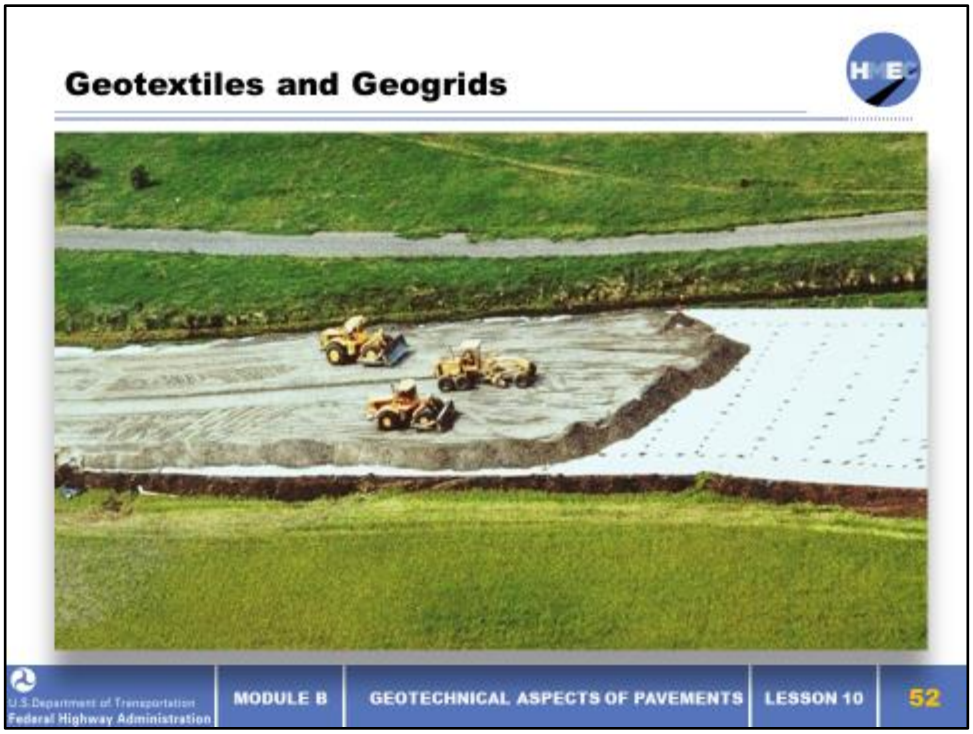
If drainage alone is not sufficient, the subgrade may need to be stabilized

Slide 51



This may require additional subgrade excavation, i.e., cut and fill. From a design perspective, there is a limit of contribution to pavement. For example, the AASHTO 1993 permits consideration of gravel thickness beyond 1.6 ft. as a structural layer—i.e., can treat it as an additional subbase.

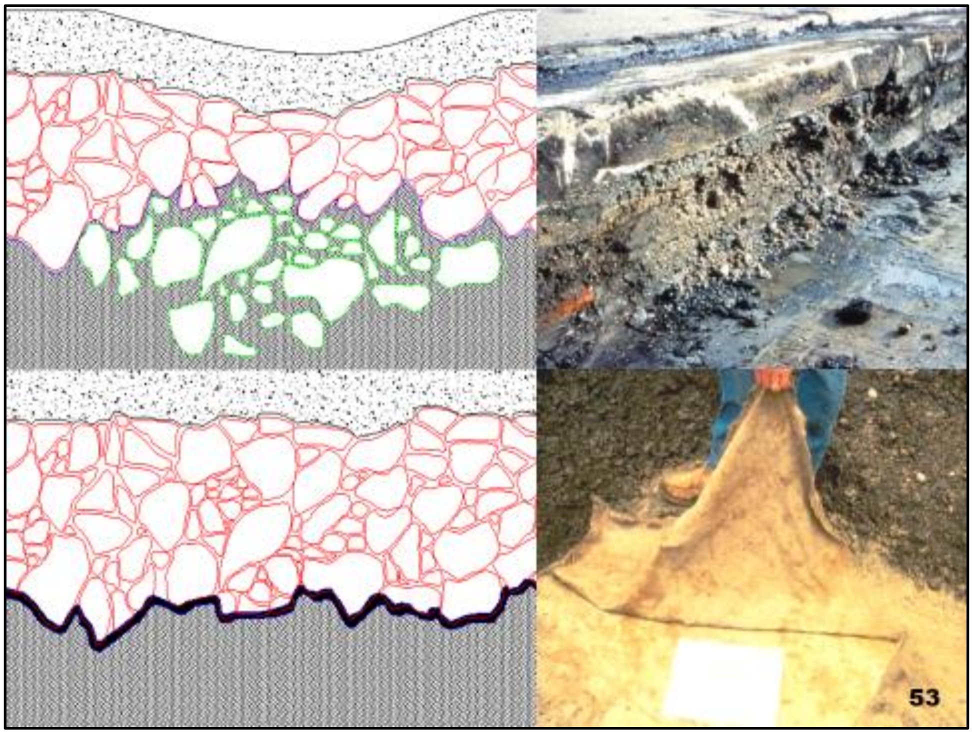
Slide 52



This slide used to introduce mechanical stabilization of soft subgrade soils using geotextiles and/or geogrids and a reduced amount of gravel to provide a working platform. Geosynthetic stabilization is a combination of the separation, filtration, and reinforcement functions. Drainage can also play a role.

Geosynthetics can be used to mitigate cracking of flexible pavement placed on swelling subgrade as you discussed in Lesson 4.

Slide 53

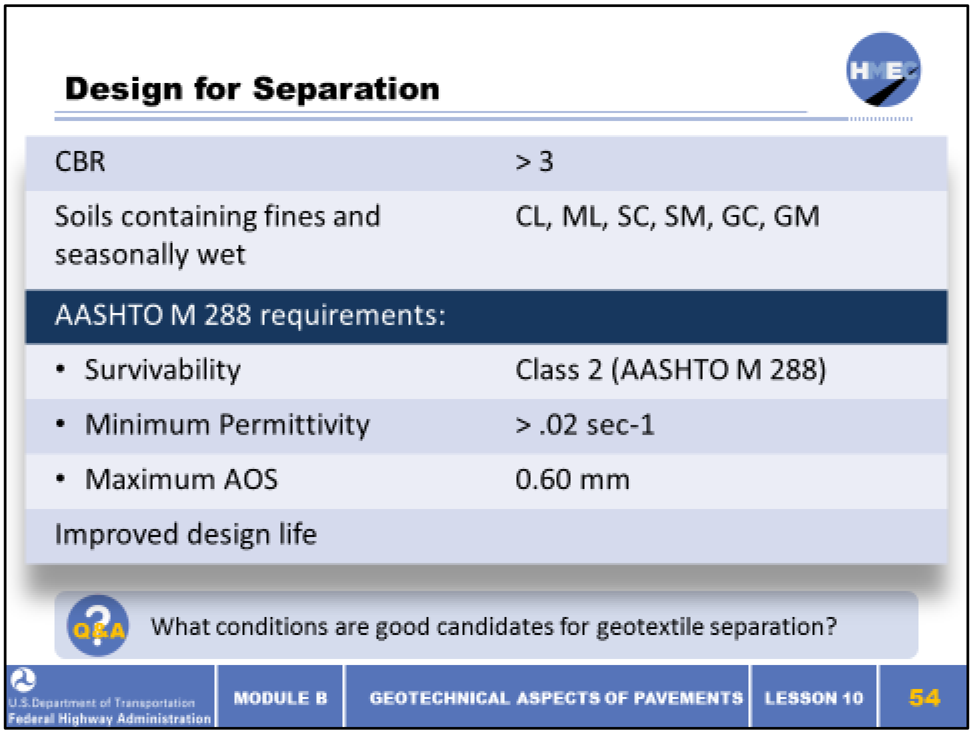


It is important that the base course does not migrate into a soft subgrade, as this reduces the effective base course thickness. Likewise, it is important to prevent pumping of fines from the subgrade up into the base course aggregate. The zone of aggregate contaminated with fines will have a stiffness equal to the subgrade.

The upper right-hand photo shows an excavated pavement that failed. The base course aggregate is fully contaminated with fines. The lower right-hand photo shows an excavated pavement that used a geotextile separator to prevent fines (light colored material) from contaminating the aggregate (darker material on top).

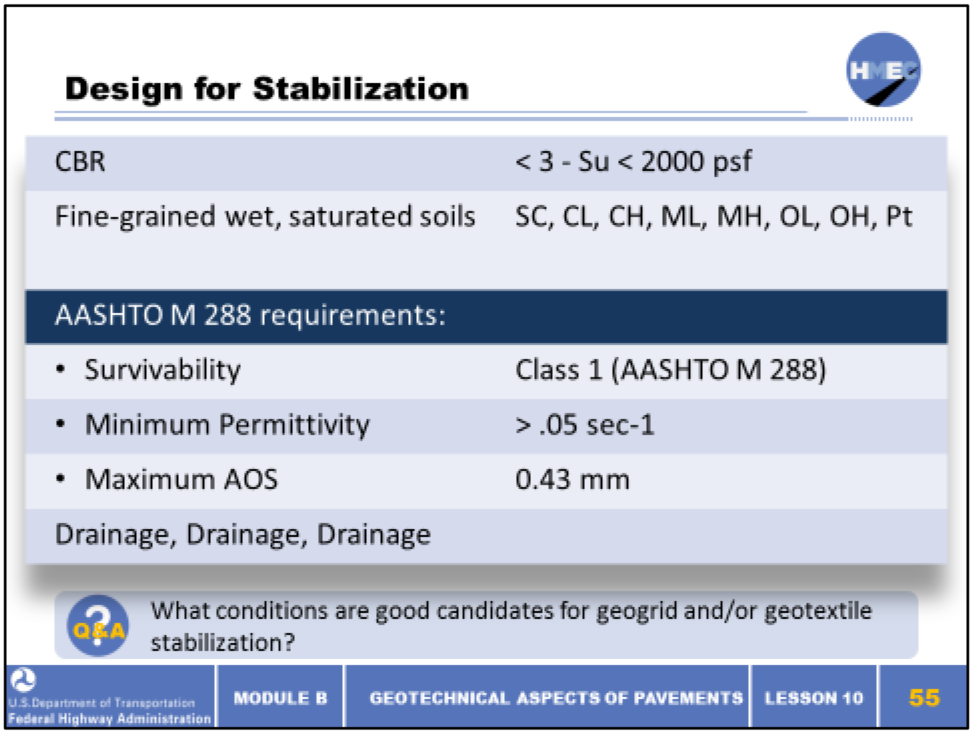
In stabilization, the geotextiles must act both as separators and filters to allow the water to rapidly drain from the subgrade. Otherwise, pore water pressures are generated, and pumping of the material occurs. Nonwoven geotextiles are good for this as they provide enough in-plane permeability for drainage and don’t need to drain much water to relieve excess pore water pressures. For geogrids, either the granular material above the geogrid must act as a separator/filter or a nonwoven geotextile must be used with the geogrid

Slide 54



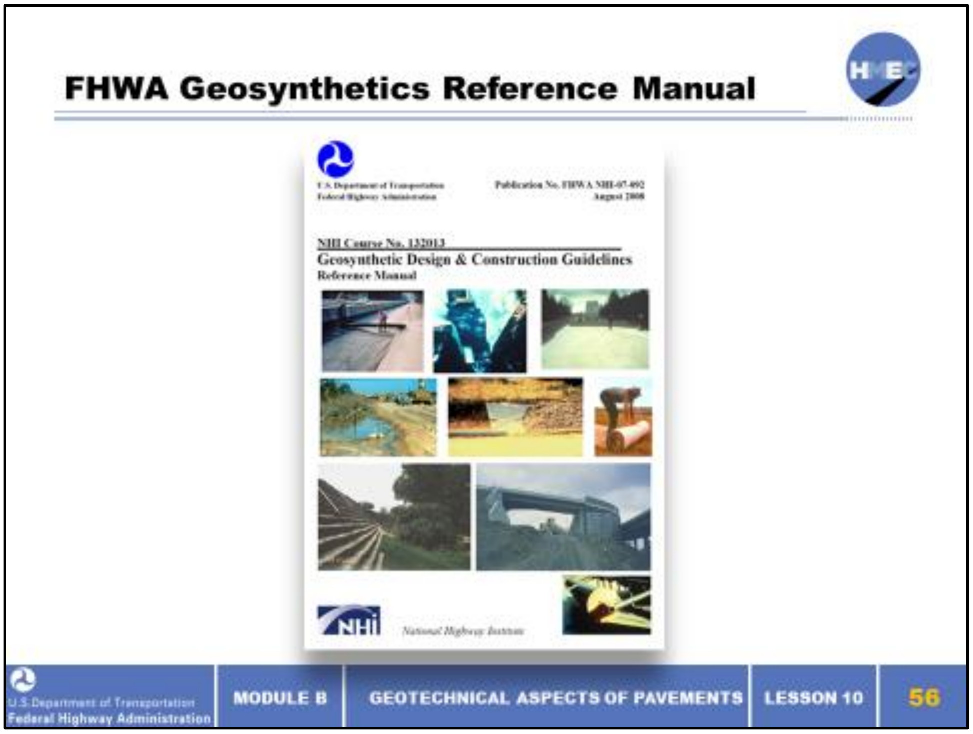
The conditions here are a good candidate for use of a geotextile separator in the pavement. Note that for fine-grained subgrades, a geotextile separator is cheap insurance. The typical separator cost is about $1/sy; if it extends road life by 1 yr., it more than pays for itself. Long-term studies indicate increase in design life of 50% or more. AASHTO M 288 is the Standard Specification for Geotextile Specification for Highway Applications, from the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, and is widely used by transportation agencies to specify different types and grades of geotextiles for a variety of transportation applications.

Slide 55



For these conditions, more than a separator is needed. These conditions require stabilization (more strength to survive construction and some drainage capability).

Slide 56



As discussed in the manual, field monitoring of roadways using geotextiles have found an improvement in subgrade strength over time, providing further potential improvement to roadway performance.

For more information and training, reference NHI Geosynthetic Engineering courses provided by the National Highway Institute.

____

____

____

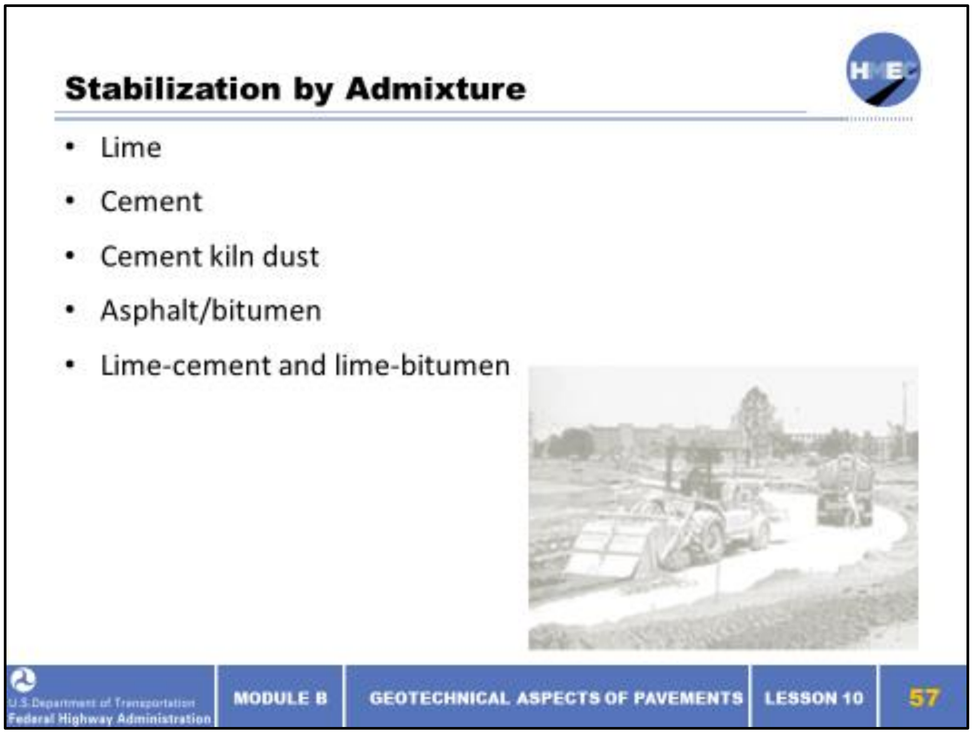
____

____

____

____

Slide 57

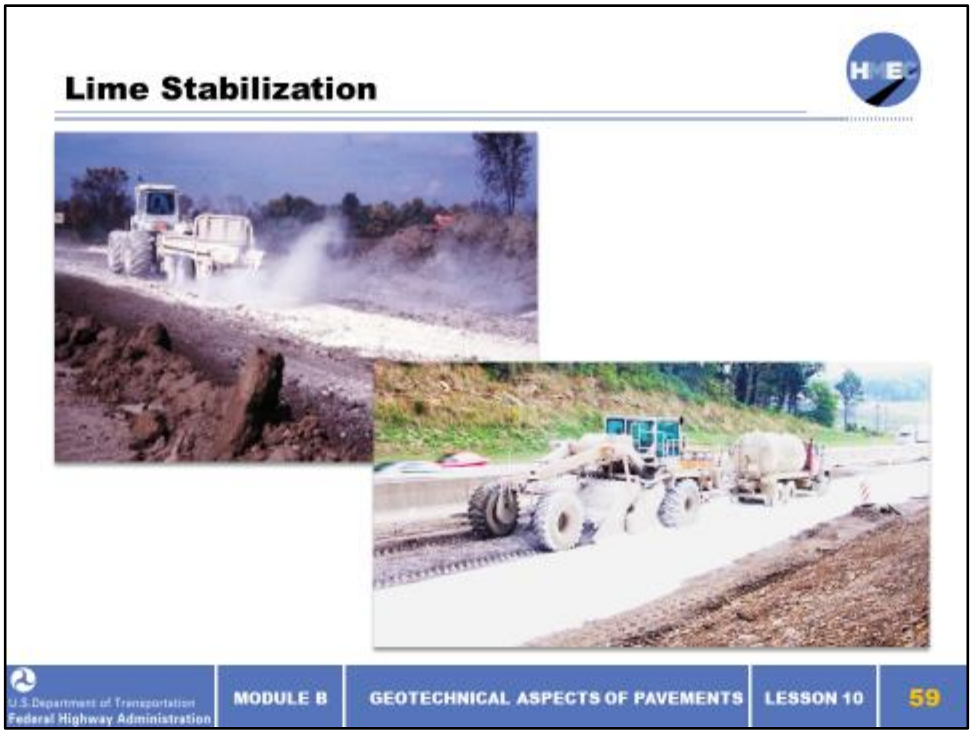


Chapter 7 and Appendix F in FHWA-NHI-10-092 Reference Manual describe details of each admixture stabilization technique: when to use, how to use, what to be careful of, etc. We will not go through design details here, but will look at a few photos to show construction with admixtures.

Slide 58

Admixture stabilization requires some specialized equipment. Today's equipment is much better than that from a decade ago. Some of the bad experiences in the past with mixing, etc. may no longer be relevant. One major downside of lime stabilization is the required curing period, which slows down construction.

Slide 59

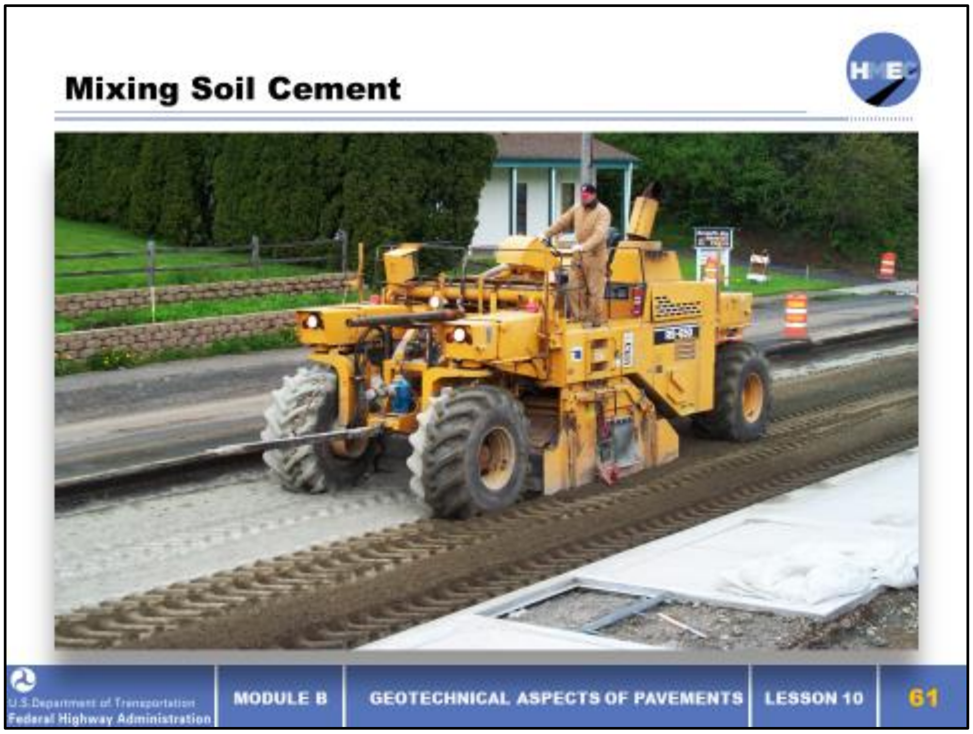


In the special case where lime (or other pozzolanic modifier such as cement or fly ash) is to be utilized to enhance the load carrying capability of the soil, additional effort is required to mix the modifier into the soil prior to compaction.

Slide 60

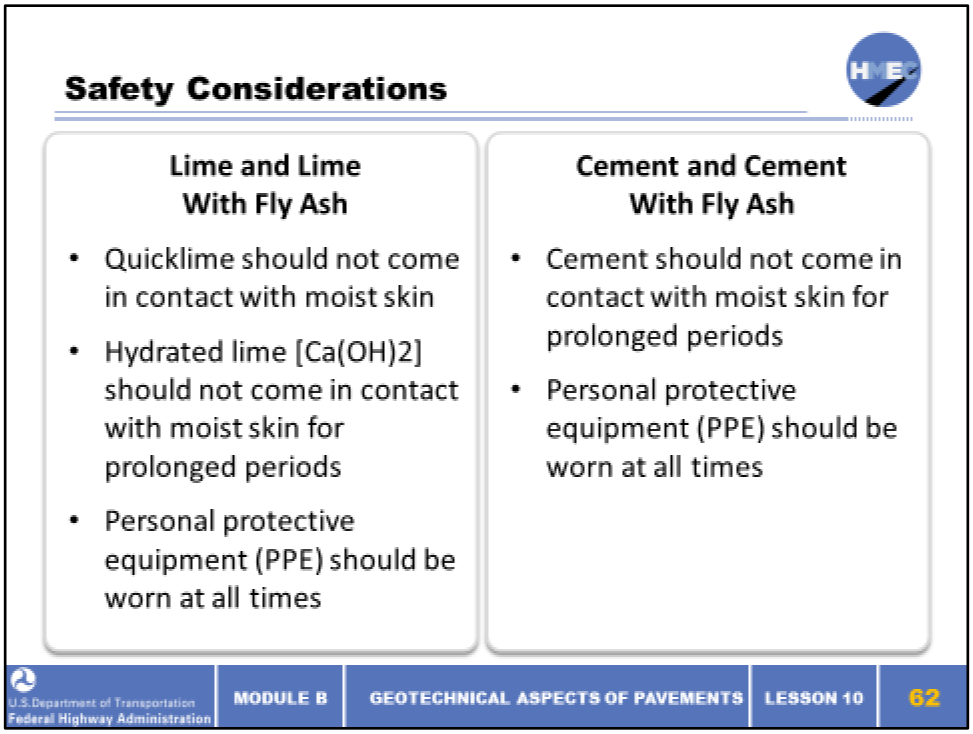
Dust can be a major problem, and may preclude the use of lime for some projects. The mixing in pug mills can be used to reduce the dust problem. Another solution to the dust problem is to use a slurry treatment.

Slide 61



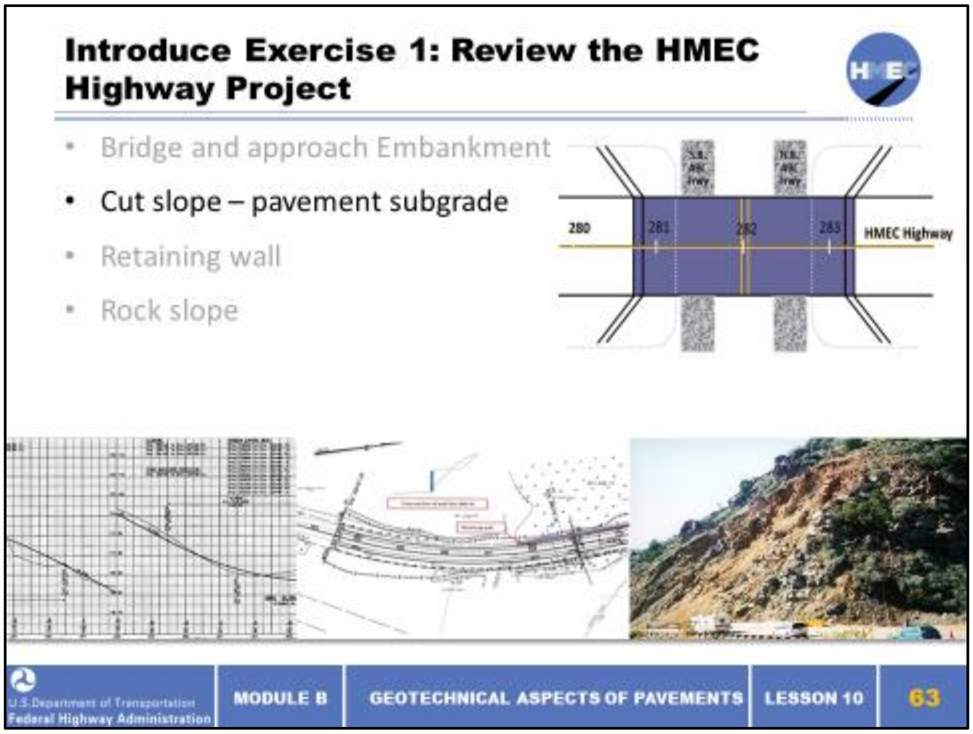
In-place mixing of cement into the subgrade is shown here. Portland cement is widely used for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness.

Slide 62



Fly ash and cement does not prevent the use of lime, but is a consideration when selecting, specifying, and constructing with a stabilizer.

Slide 63



The HMEC Highway, the example project for the exercises in this manual, involves the reconstruction and upgrading of an existing county road in the upper Midwest United States. The HMEC Highway project will allow you to apply what is discussed during the course to a realistic project. Although the project is fictitious, the components and issues it highlights could be encountered by you on one of your agency's projects.

____

____

____

____

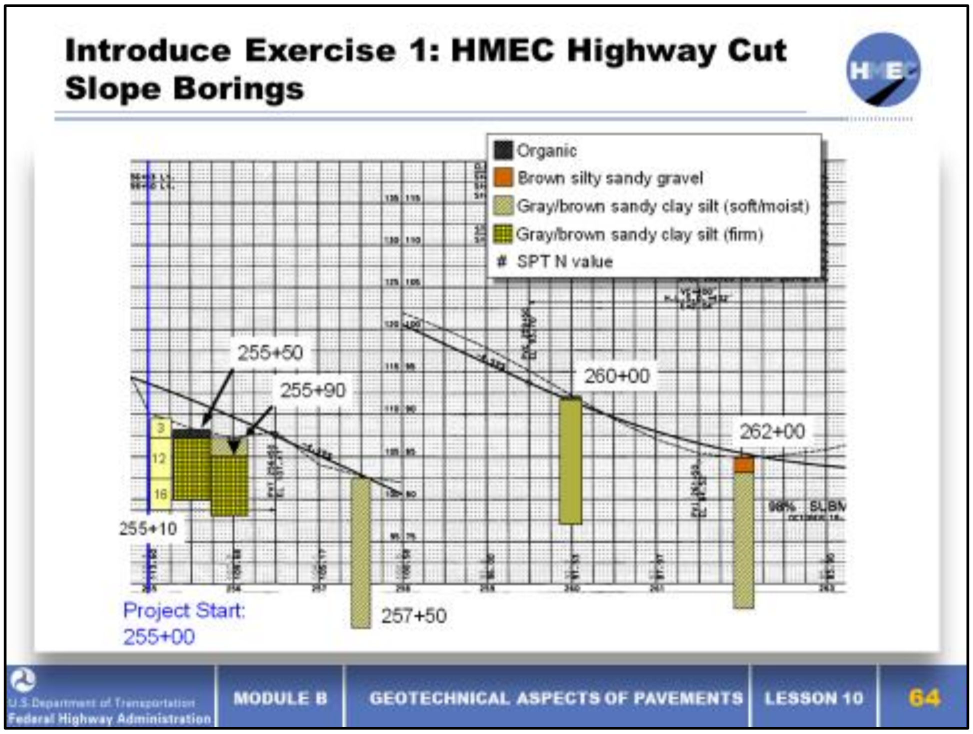
____

____

____

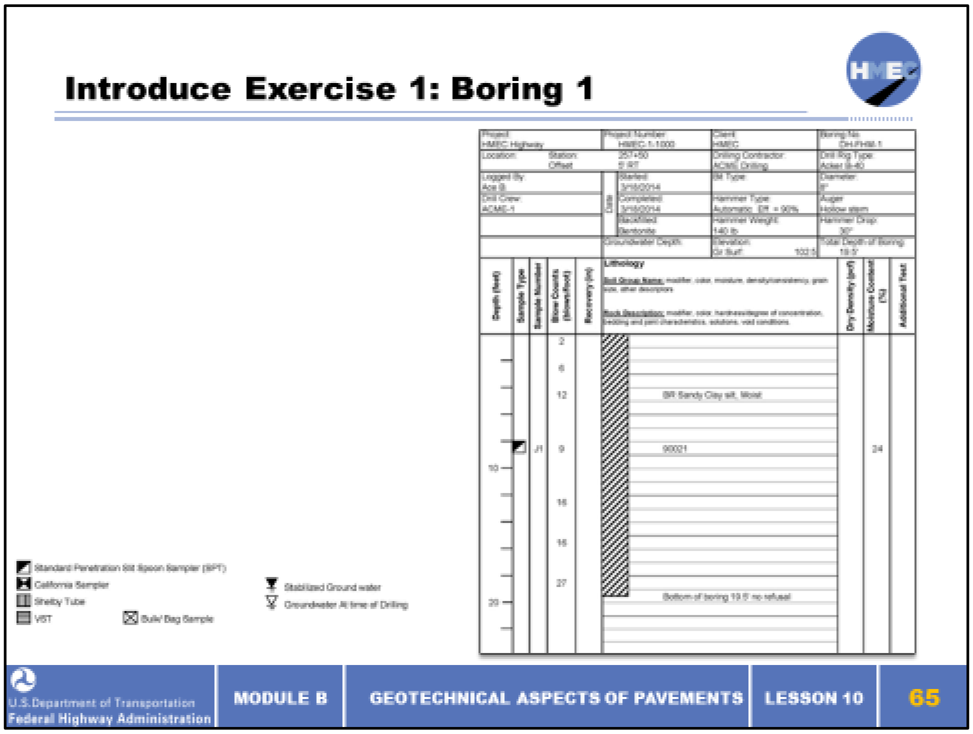
____

Slide 64



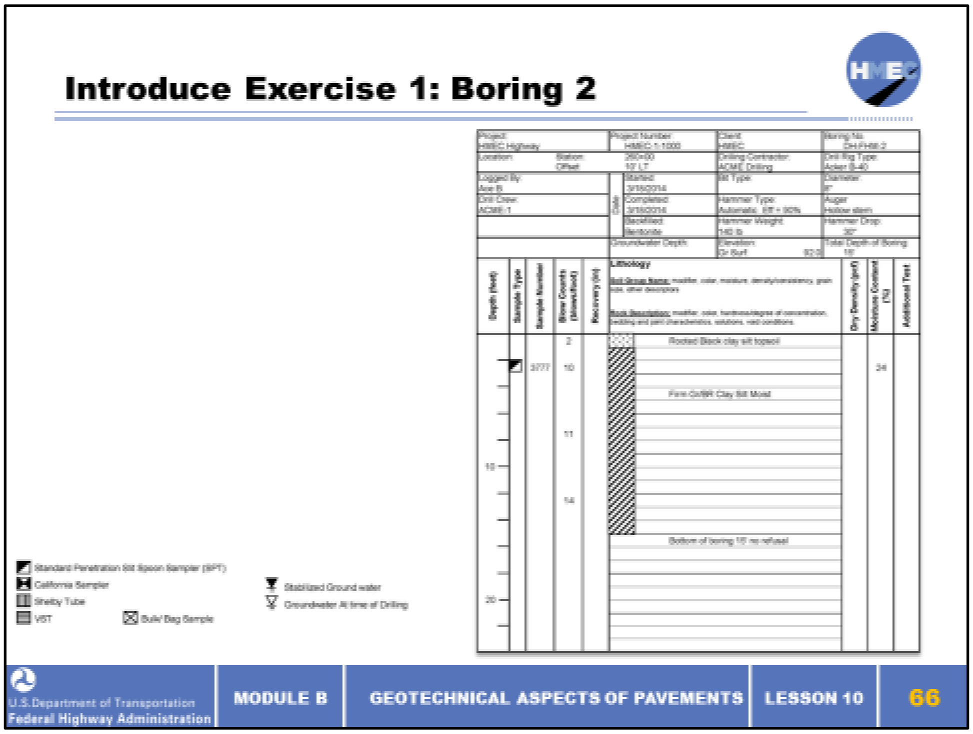
The maximum spacing is 200 ft., therefore for a 300-ft. cut slope, we recommended at least two borings. The depth of the boring should be at least 15 ft. below the ditch line elevation. SPT blow counts and disturbed samples should be obtained at 5-ft. intervals.

Slide 65



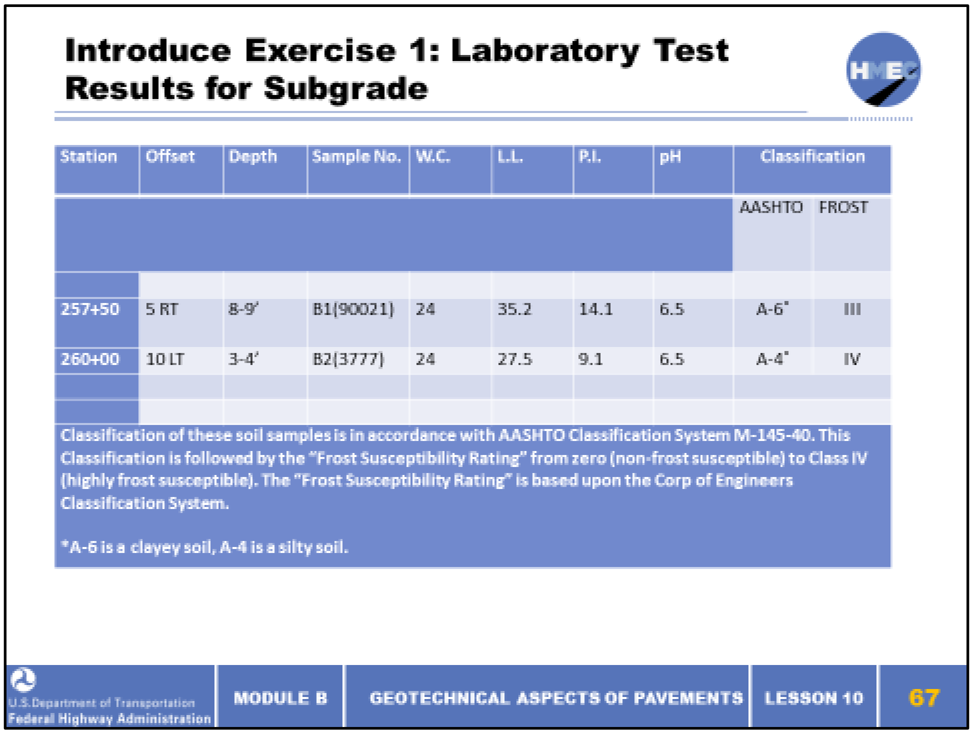
Two borings were made on this section. Laboratory tests were performed on two samples taken during the borings. This is boring number one.

Slide 66



Two borings were made on this section. Laboratory tests were performed on two samples taken during the borings. This is boring number two.

Slide 67



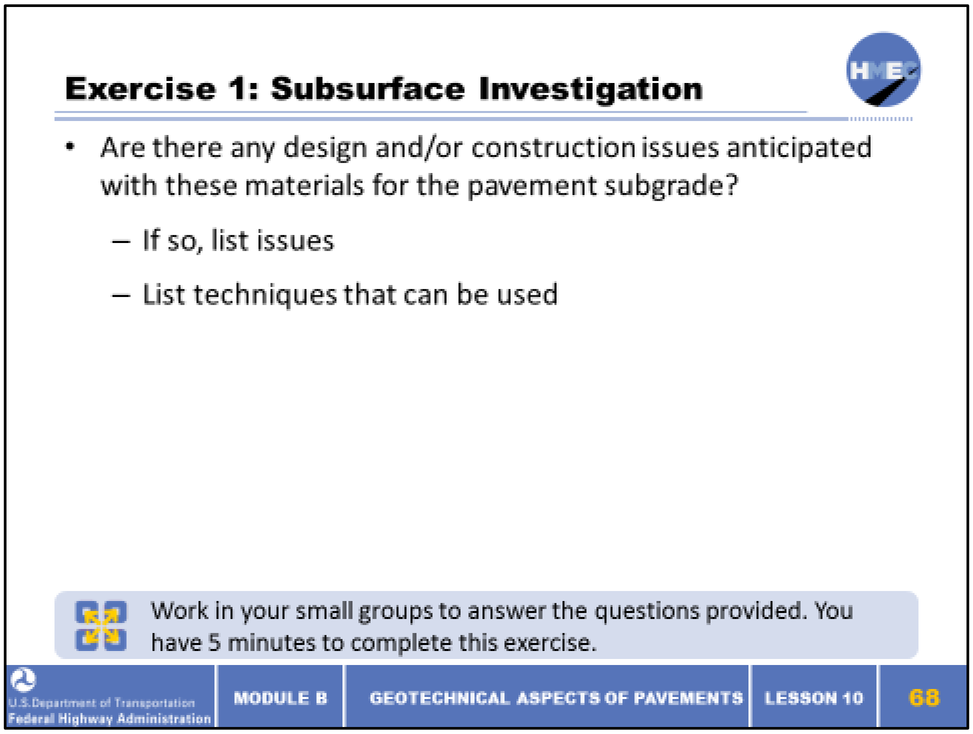
Station	Offset	Depth	Sample No.	W.C.	L.L.	P.I.	pH	Classification	
								AASHTO	FROST
257+50	5 RT	8-9'	B1(90021)	24	35.2	14.1	6.5	A-6*	III
260+00	10 LT	3-4'	B2(3777)	24	27.5	9.1	6.5	A-4*	IV

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This Classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the Corp of Engineers Classification System.

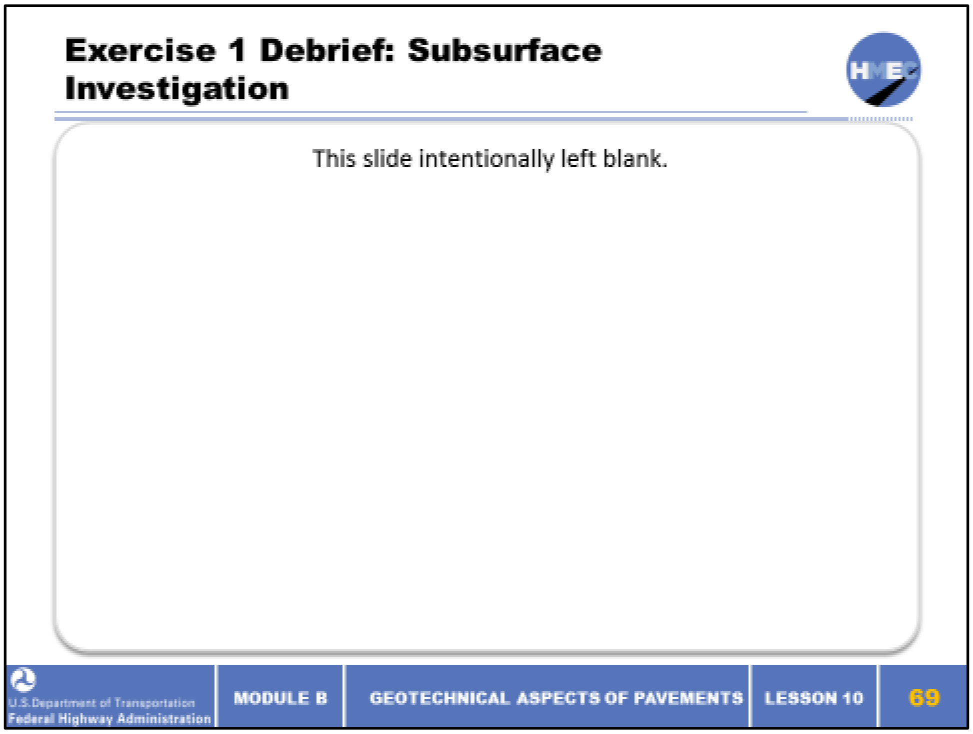
*A-6 is a clayey soil, A-4 is a silty soil.

These are the test results for the two samples taken during Borings 1 and 2.

Slide 68

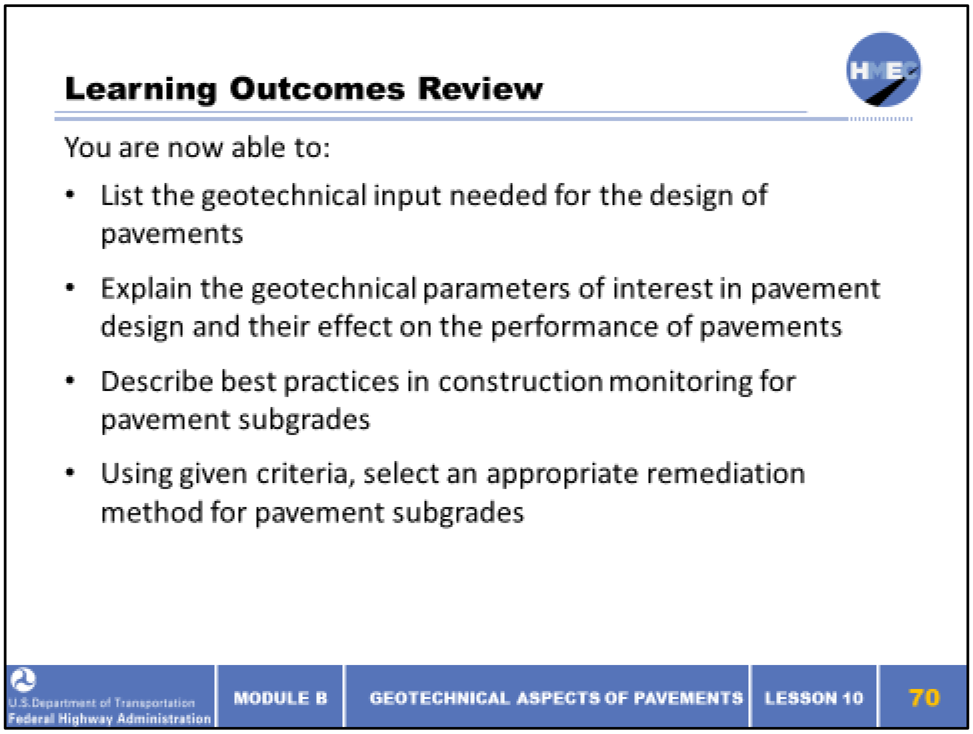

Exercise 1: Subsurface Investigation
HMEC
• Are there any design and/or construction issues anticipated with these materials for the pavement subgrade?
– If so, list issues
– List techniques that can be used
Work in your small groups to answer the questions provided. You have 5 minutes to complete this exercise.
U.S. Department of Transportation Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
68

Slide 69



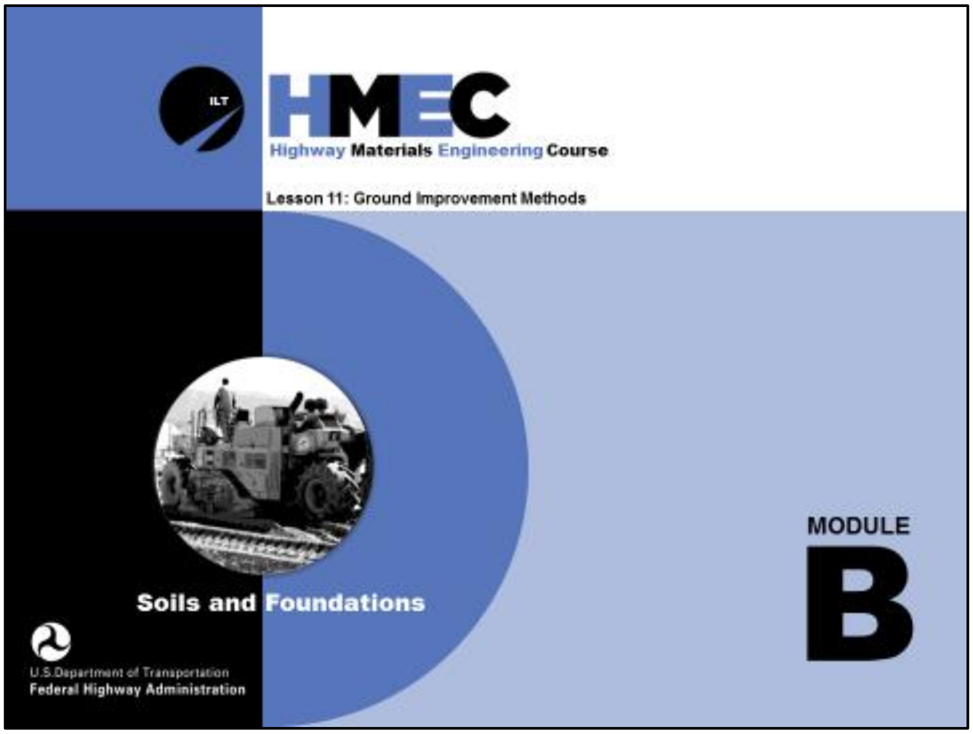
What do the logs and tests reveal?

Slide 70

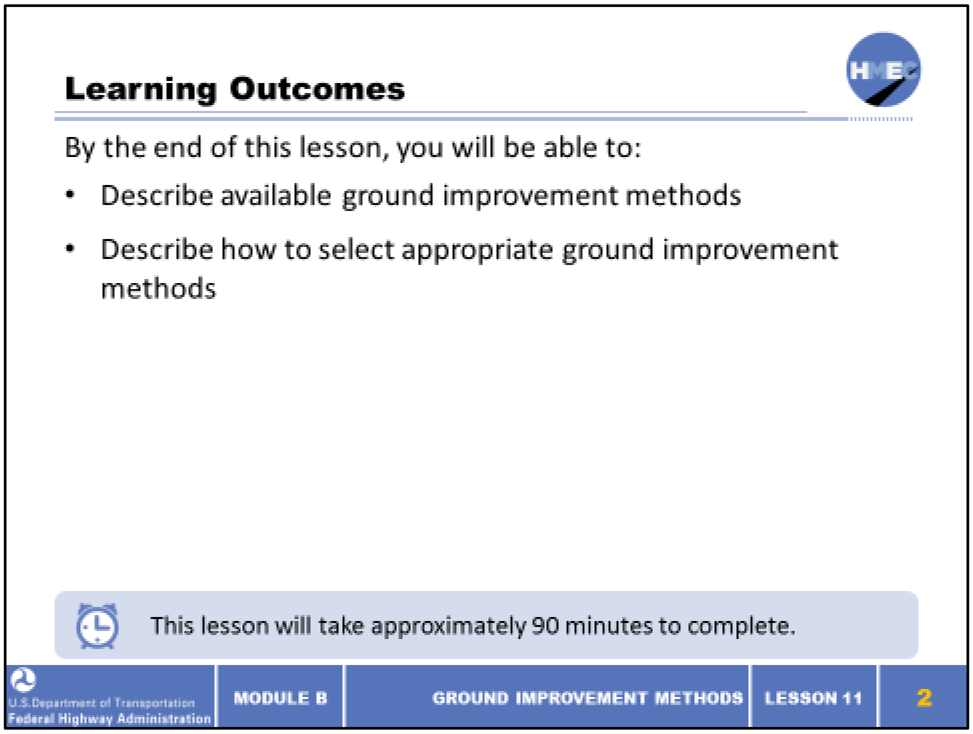

Learning Outcomes Review
You are now able to:
• List the geotechnical input needed for the design of pavements
• Explain the geotechnical parameters of interest in pavement design and their effect on the performance of pavements
• Describe best practices in construction monitoring for pavement subgrades
• Using given criteria, select an appropriate remediation method for pavement subgrades
U.S. Department of Transportation Federal Highway Administration
MODULE B
GEOTECHNICAL ASPECTS OF PAVEMENTS
LESSON 10
70

This page is intentionally blank.

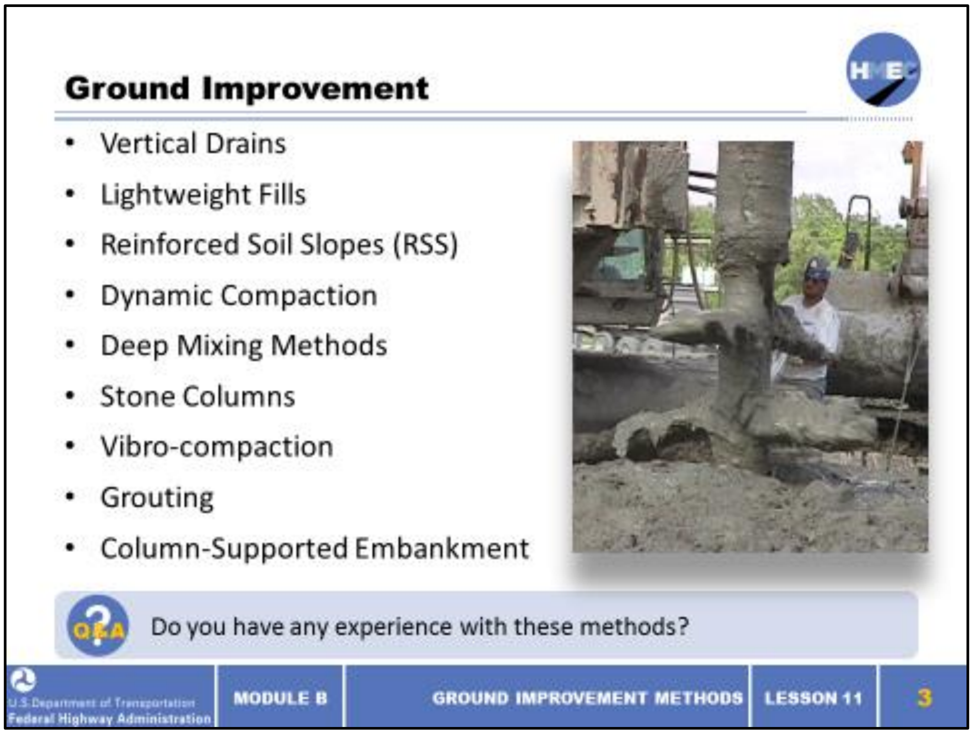
Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 11: Ground Improvement Methods
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
Describe available ground improvement methods
Describe how to select appropriate ground improvement methods
This lesson will take approximately 90 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
2

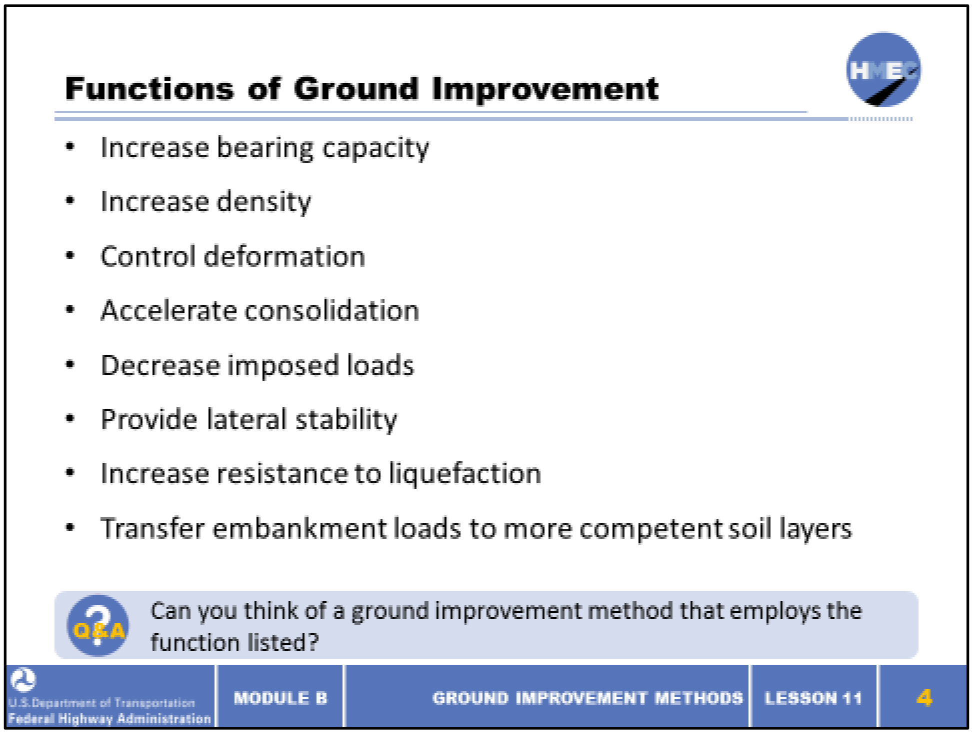
Slide 3



Ground improvement technologies are geotechnical construction methods used to alter and improve poor ground conditions in order for embankment and structure construction to meet project performance requirements, in which soil replacement is not feasible for environmental or technical reasons, or it is too costly.

There are other ground improvement methods and variations of the ones listed here. We are going to discuss these nine methods to get you acquainted with a few of the technologies.

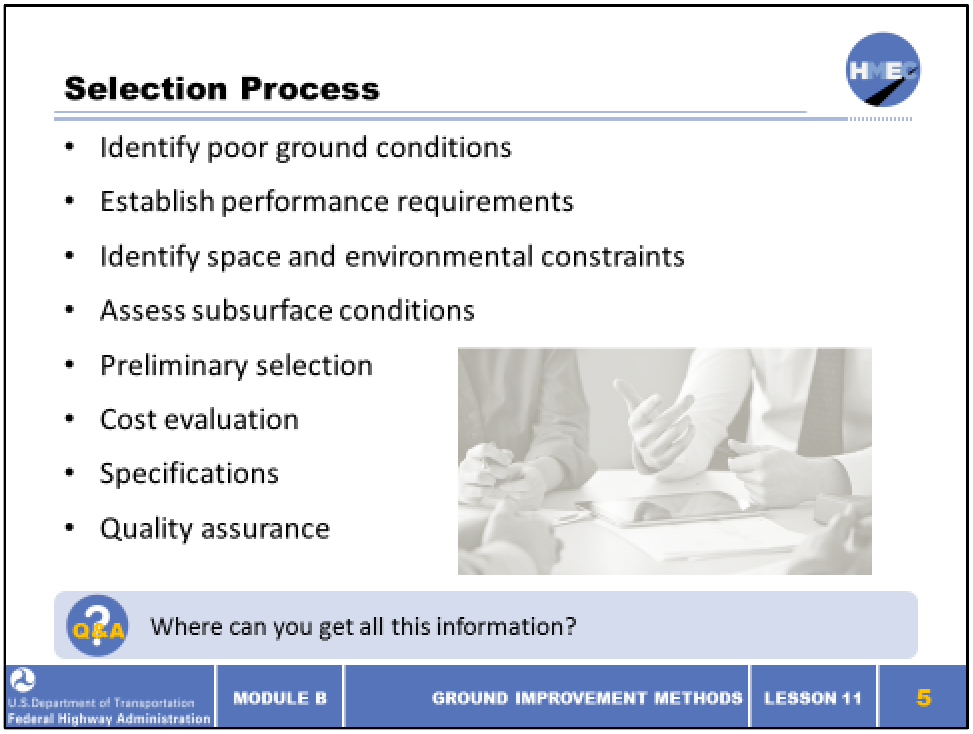
Slide 4



There are three strategies available to accomplish the above functions representing different approaches.

• The first method is to increase the shear strength, density, and/or decrease the compressibility of the foundation soil;
• The second method is to use a lightweight fill embankment to reduce significantly the applied load to the foundation; and
• The third method is to transfer loads to a more competent deeper layer.

Slide 5



Poor ground conditions through a proper and complete site investigation must be identified. Determine what soil problem must be solved. Establish the performance requirements that must be met for your structure/embankment to function as expected. How much space is available for installation and are there any environmental constraints, (ROW, overhead clearance, disposal of excavation, adjacent structures) which might eliminate some methods? Subsurface conditions such as boulders, dense layers overlying compressible layers, and groundwater conditions may limit the use of some methods. The preliminary selection will most likely include multiple methods. Those “finalists” must be evaluated for cost, established specifications, and quality assurance methods that will be used.

____

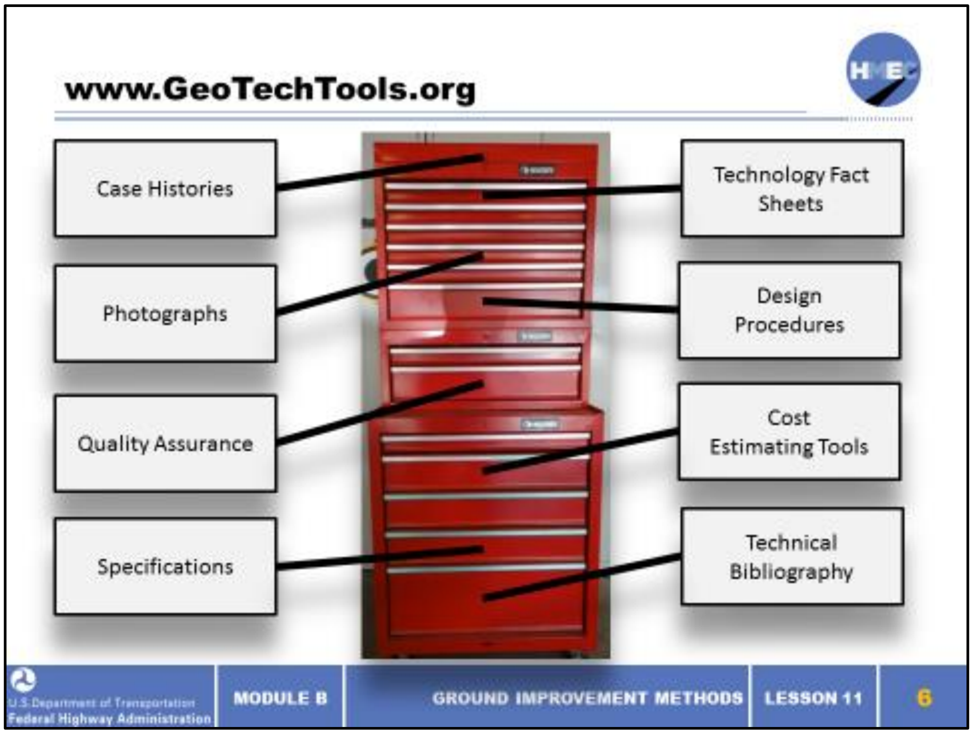
____

____

____

____

Slide 6



The system GeoTechTools allows engineers to access an organized array of various information products, or engineering tools, for each candidate technology. It includes the eight "engineering tools" listed here. There are technology fact sheets, design procedures, photographs, specifications, cost estimating tools, quality assurance procedures, case histories, and a technical bibliography for each of approximately 50 technologies. We say approximately 50 because this is a living Web site and new technologies will be added as they become available.

____

____

____

____

____

____

____

Slide 7

Engineers face many obstacles when using a "new" technology. The technology simply may be "new" to their agency or just "new" to the agency management that might ask what other agencies have used this technology. And what was their experience with it?

GeoTechTools gives the engineering team what they need to convince management to consider the use of this technology and all the tools needed to "engineer" with this technology.

____

____

____

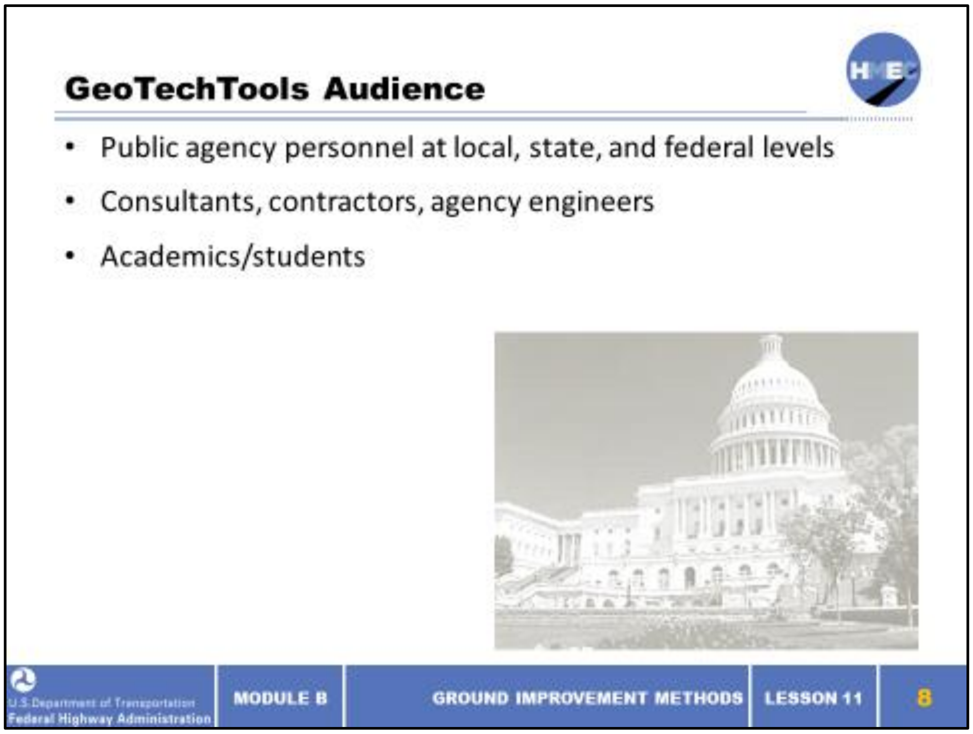
____

____

____

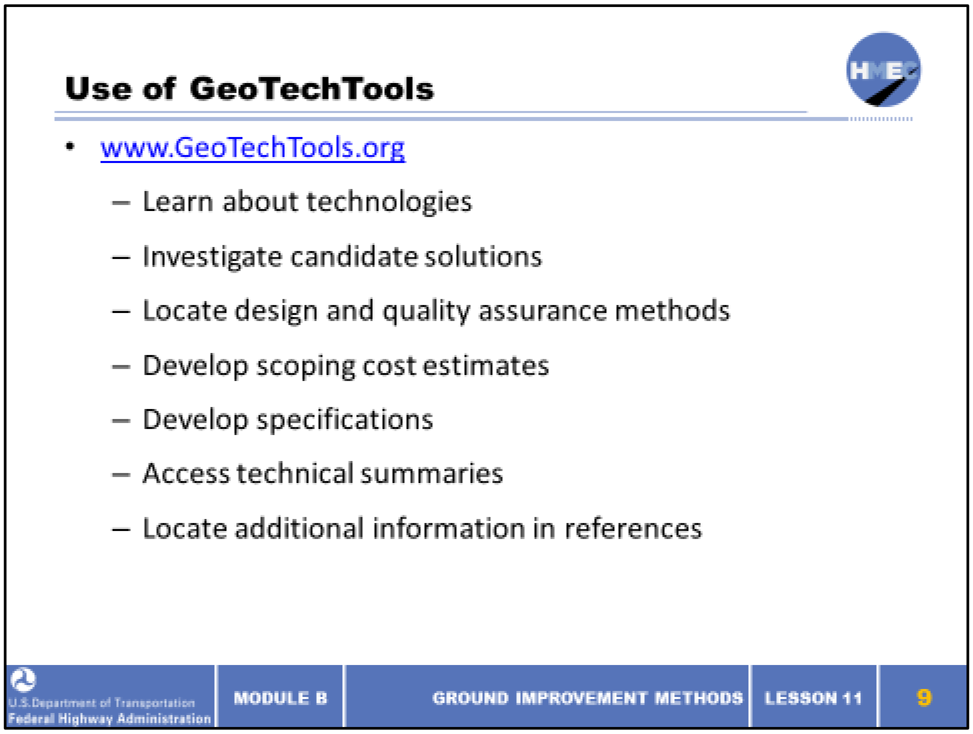
____

Slide 8



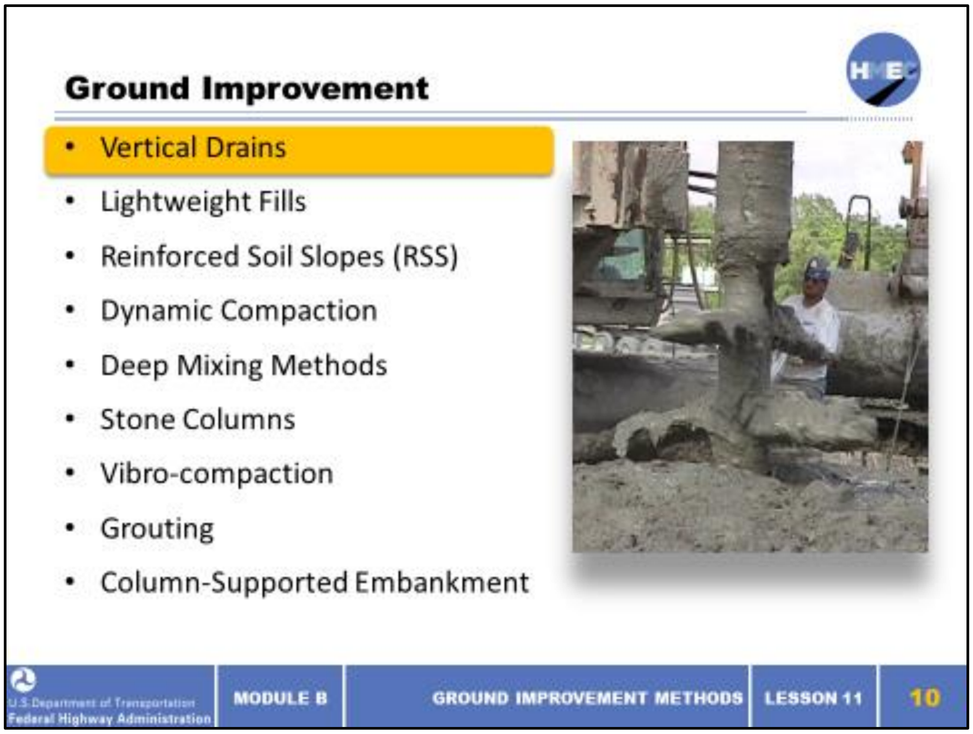
It is very useful to groups dealing with transportation design and construction, such as yourselves. Furthermore, it is applicable to private works design and construction. And it is applicable to works worldwide.

Slide 9

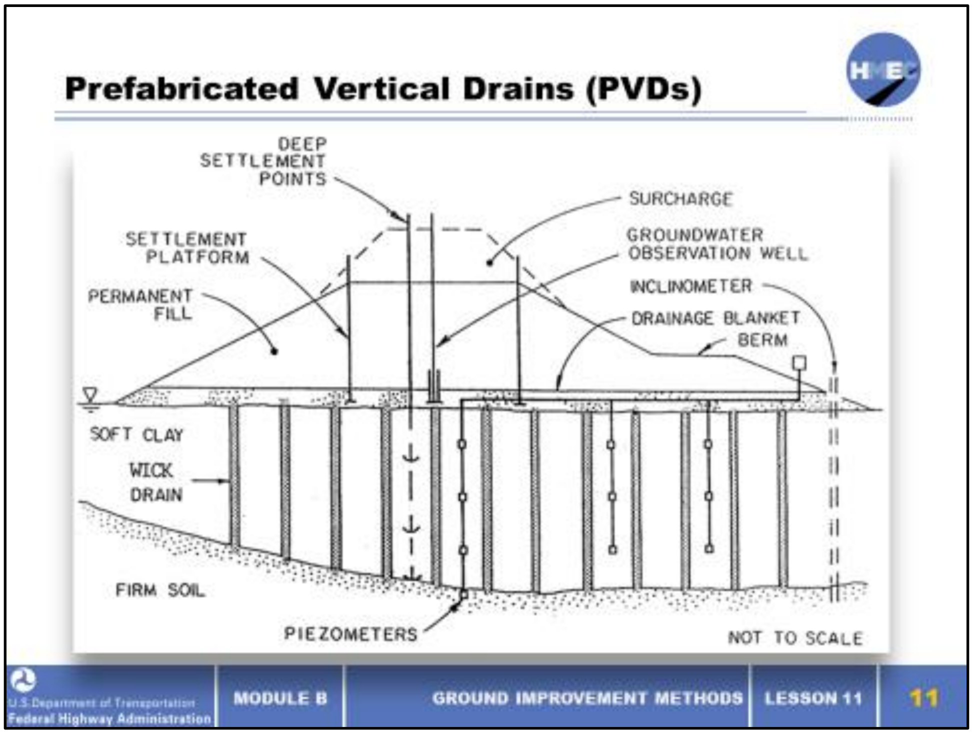


Note that cost data included on the Web site is for preliminary scoping to compare candidate technologies. It is not to be used for detailed cost estimates once an appropriate technology is selected

Slide 10


Ground Improvement
• Vertical Drains
• Lightweight Fills
• Reinforced Soil Slopes (RSS)
• Dynamic Compaction
• Deep Mixing Methods
• Stone Columns
• Vibro-compaction
• Grouting
• Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
10

Slide 11



In practice, PVDs are most commonly used in consolidation situations where the soil to be treated is a moderately to highly compressible soil with low permeability and fully saturated in its natural state. Such soils are typically described as silts, clays, organic silts, organic clays, muck, peat, swamps, muskeg, and sludge.

When used in conjunction with surcharging or preloading, the principal benefits of using PVDs are to:

- Decrease the settlement time required such that final construction can be completed in a reasonable time, with minimal post-construction settlement.
- Decrease the amount of surcharge or preload material required to achieve a settlement in the given time.
- Increase the rate of strength gain due to consolidation of soft soils when stability is of concern.

PVDs can be used in conjunction with a surcharge to further accelerate consolidation. However, PVDs do not add any strength to the soil; and therefore, the stability of the embankment with surcharge must be evaluated. Note that stage construction may be used to solve a stability issue.

____

____

____

____

____

____

____

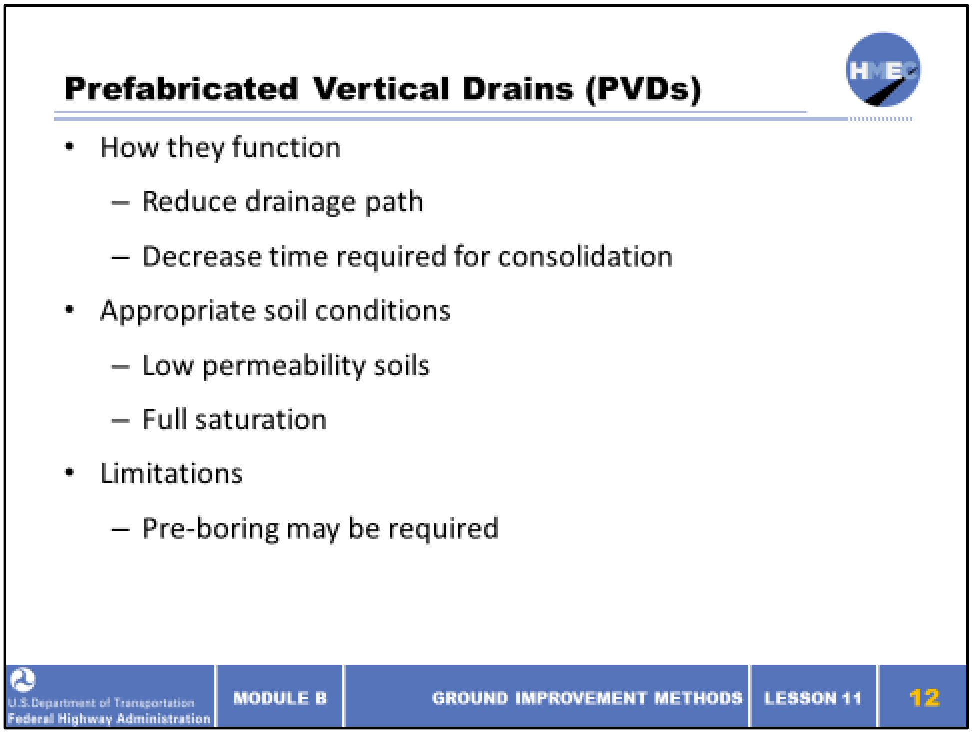
____

____

____

____

Slide 12



By reducing the length of the drainage path for pore water, PVDs reduce the amount of time required for consolidation. As the consolidation occurs, there will a strength gain in the soil. PVDs are most effective for low permeability soils since they may not be necessary for higher permeability soils.

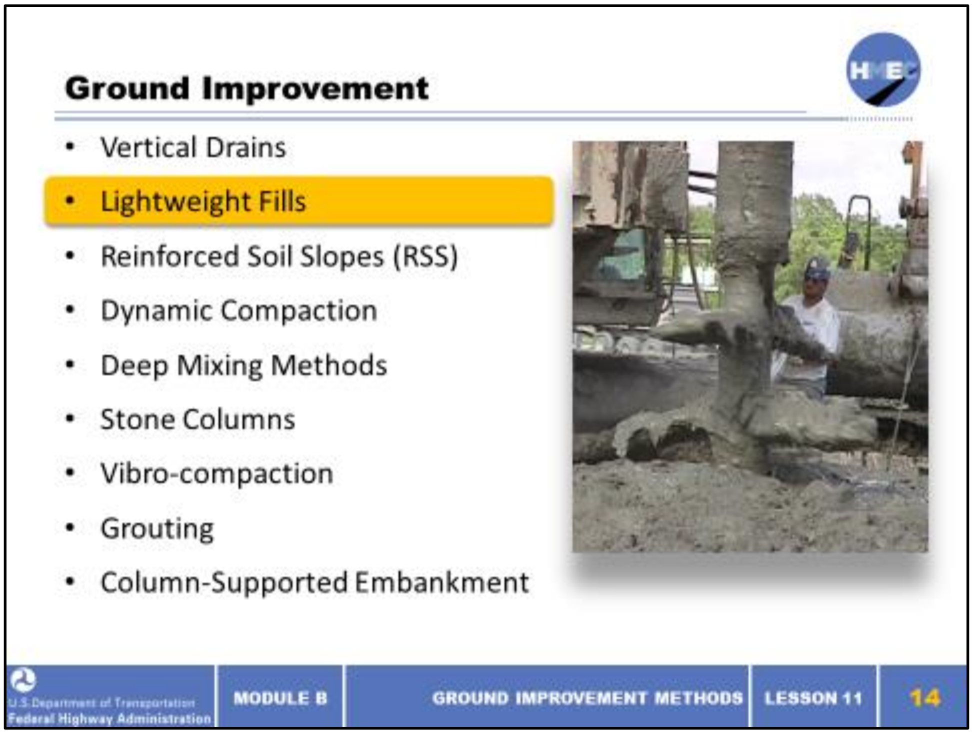
If the soil layer being consolidated is not fully saturated, excess pore water pressure will not build up, and pore water drainage will not occur.

Slide 13

A layer of 12–18 SPT blows/ft. firm glacial clay existed above the compressible layer, and the contractor’s equipment could not install the PVD through the layer. Therefore, pre-boring was required.

Case Study Notes: Pre-boring to install PVDs on a widening project of I-35 north of Des Moines, Iowa.

Slide 14


Ground Improvement
HMEC
• Vertical Drains
• Lightweight Fills
• Reinforced Soil Slopes (RSS)
• Dynamic Compaction
• Deep Mixing Methods
• Stone Columns
• Vibro-compaction
• Grouting
• Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
14

Slide 15

Lightweight fill may be constructed from several different materials, including (clockwise from top left) geofoam, foamed concrete, shredded tires, and expanded shale.

Lightweight fill materials have many potential highway applications, principally where the capacity of the underlying soft soil is too low to carry the design load and/or the estimated settlement of the embankment is too large. Under these conditions, either the foundation soils must be improved or the embankment load reduced. Lightweight fills fulfill the latter function. The use of a lightweight fill material would likely result in reduced settlement and increased stability. Also, lightweight fill can be used to reduce the horizontal forces applied to earth retaining structures and to increase an embankment's resistance to seismic loads (low unit weight density results in lower seismic inertial forces).

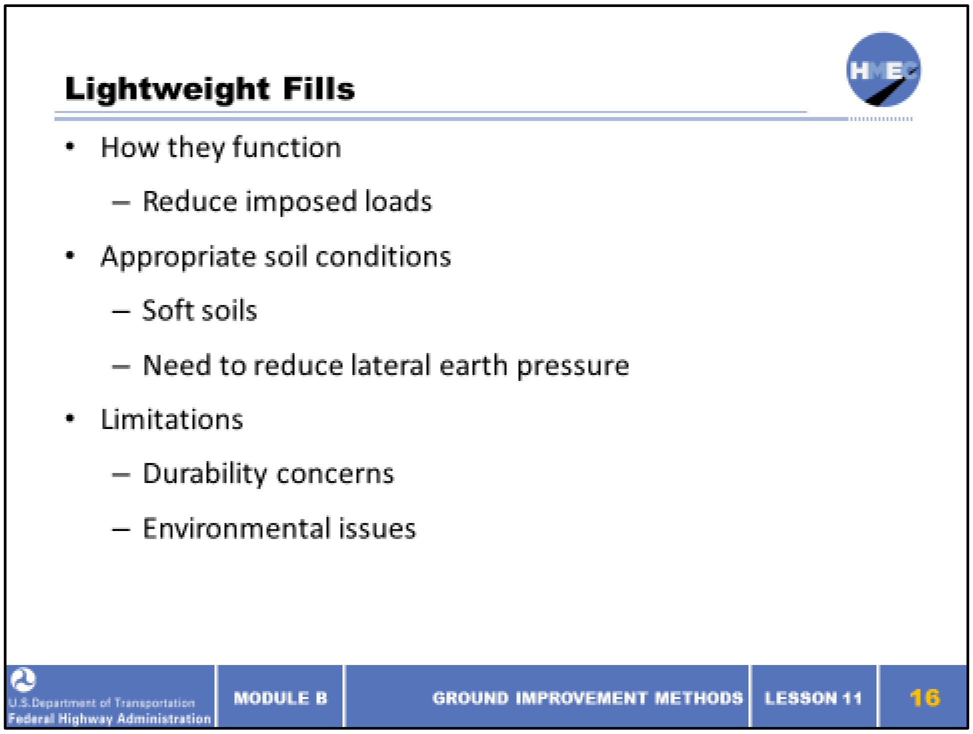
The weight of lightweight fill material vary dramatically from 0.6 #/ft^3 for geofoam to 110 #/ft^3 for boiler slag.

____

____

____

Slide 16



Lightweight fills primarily work by reducing the imposed loads. As we discussed they result in reduced settlement and increased stability. Lightweight fills can be used whenever reducing the load or lateral earth pressure would provide a benefit.

Some lightweight fill materials (e.g., geofoam) must be protected to ensure longevity. Because geofoam is subject to deterioration from hydrocarbon spills, a concrete slab or geomembranes are generally placed over the surface of the blocks. Wood fibers can decay over a long period of time, although recent studies indicate that the deterioration is limited to the outer surface of an embankment. Fly ash deposits need to be protected with a soil surface to minimize or prevent erosion of the side slopes.

Some of the lightweight fill materials generate leachate as water passes through these deposits. Fortunately, design methods have been developed to minimize the amount of leachate, and to date, these measures have worked satisfactorily. However, the additional cost of these measures should be considered during design.

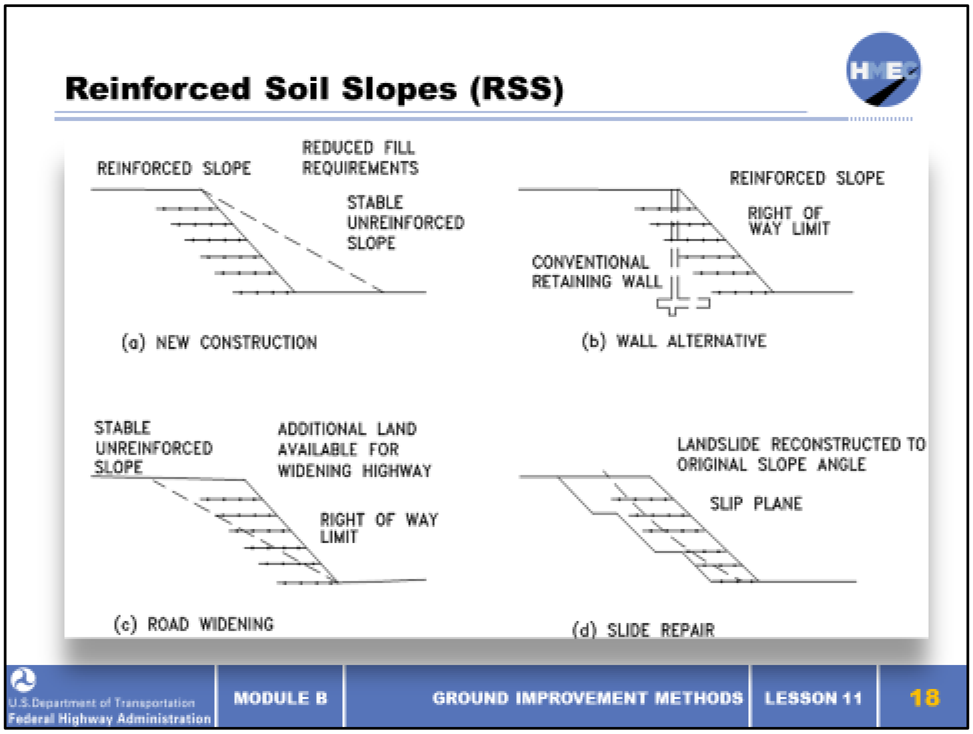
Slide 17

Ground Improvement

- Vertical Drains
- Lightweight Fills
- Reinforced Soil Slopes (RSS)
- Dynamic Compaction
- Deep Mixing Methods
- Stone Columns
- Vibro-compaction
- Grouting
- Column-Supported Embankment

U.S. Department of Transportation Federal Highway Administration | MODULE B | GROUND IMPROVEMENT METHODS | LESSON 11 | 17

Slide 18



The most widespread application for transportation-related projects is in the construction of retaining structures and steepened soil slopes. There are two primary purposes for using reinforcement in engineered slopes.

• To increase the stability of the slope, particularly if a steeper than *safe* unreinforced slope is desirable or after a failure has occurred.
• To provide improved compaction at the edges of a slope, thus decreasing the tendency for surface sloughing.

____

____

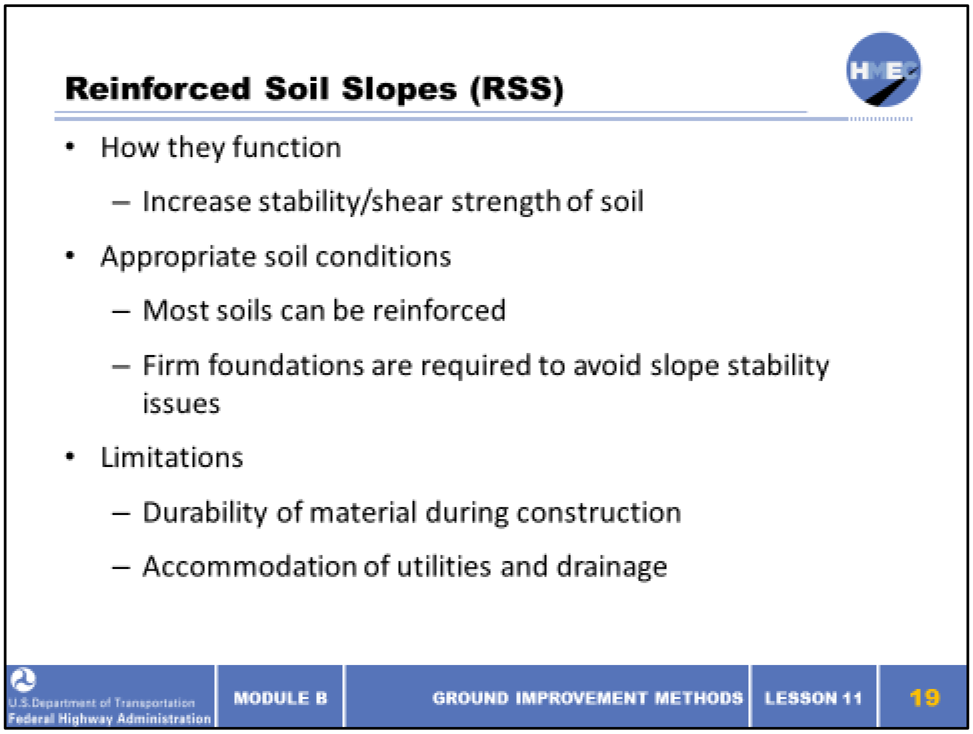
____

____

____

____

Slide 19



The primary function of reinforcements is to restrain soil deformations. In doing so, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

A steeper slope will increase the potential for slope stability issues within the underlying soil. Therefore, if an RSS is built over soft soils that may have stability issues preliminary stability analyses must be made to determine if sufficient shear strength is available to support the weight of the reinforced fill. Where these conditions are not satisfied, ground improvement techniques must be considered to increase the bearing capacity at the foundation level.

Durability issues can be resolved with proper specifications and QC to control the installation process. Accommodating drainage and/or utilities may require cutting the reinforcement, which must be considered in the design.

____

____

____

Slide 20

This approach embankment was built using RSS on 1:1 slope to accommodate ROW constraints between the bridge abutment and the Railroad. The RSS allows space for an access road between the toe of slope and the pier. Support piling for the bridge abutment was driven through the RSS reinforcement.

____

____

____

____

____

____

____

____

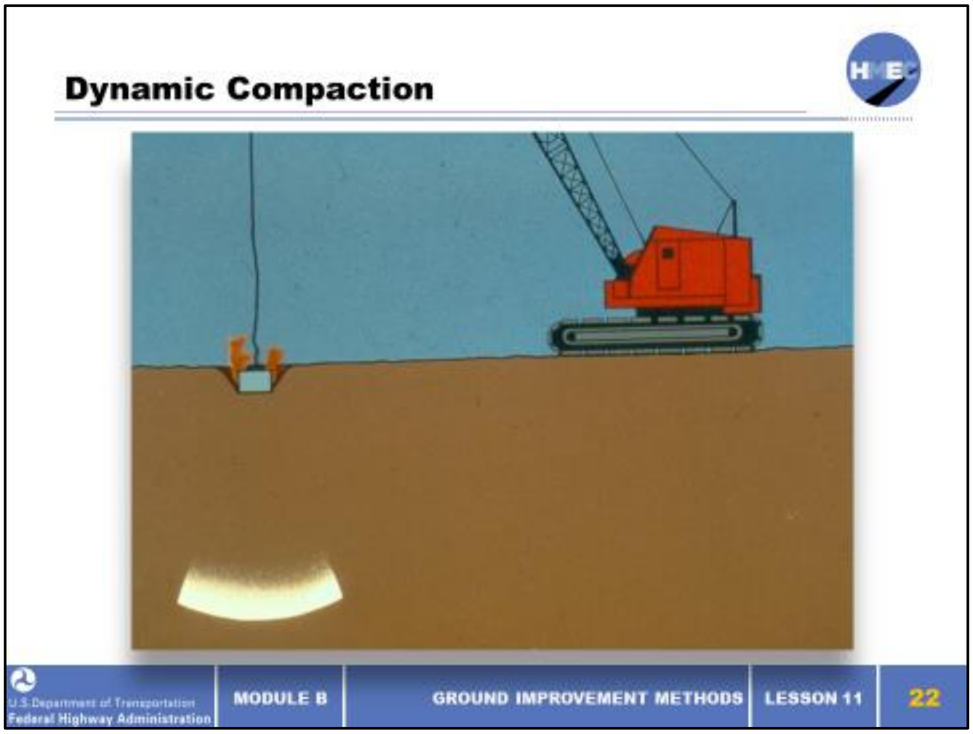
Slide 21

Ground Improvement

- Vertical Drains
- Lightweight Fills
- Reinforced Soil Slopes (RSS)
- **Dynamic Compaction**
- Deep Mixing Methods
- Stone Columns
- Vibro-compaction
- Grouting
- Column-Supported Embankment

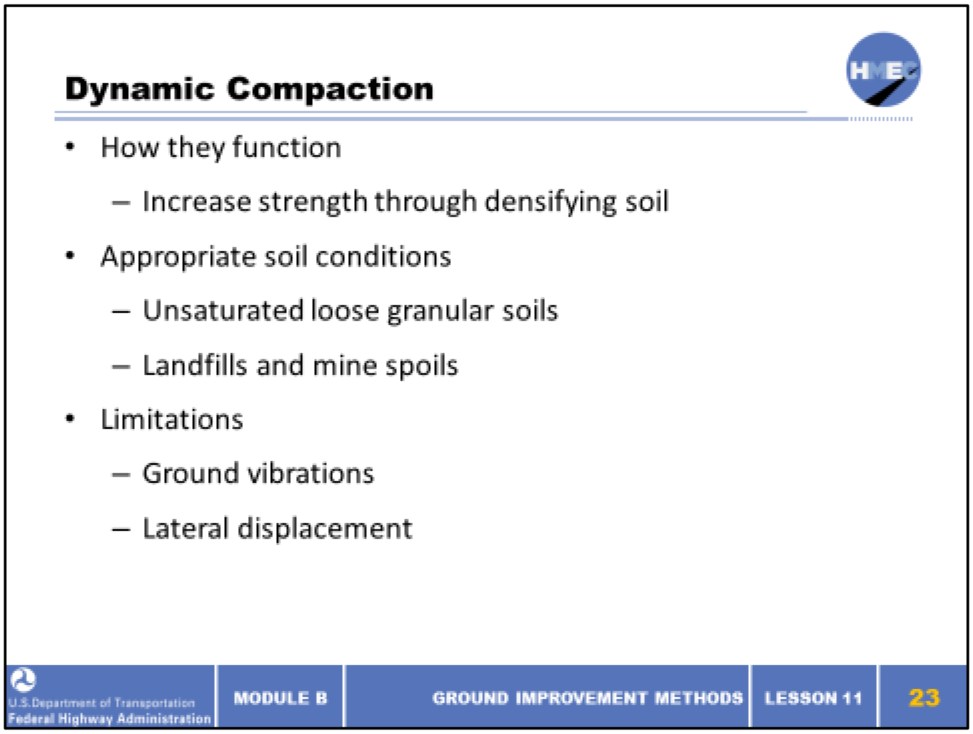
U.S. Department of Transportation Federal Highway Administration | MODULE B | GROUND IMPROVEMENT METHODS | LESSON 11 | 21

Slide 22



Dynamic compaction strengthens the soil through densification. The energy is applied by repeatedly raising and dropping a tamper with a mass ranging from 5–20 tons at heights ranging from 30–100 ft. The tamper is lifted and dropped by a conventional crane with a single cable, plus a winch that has a free spool attachment that allows the single cable to unwind with minimum friction. The tamper's energy of impact at the ground surface results in densification of the deposit to depths that are proportional to the energy applied. The depth of improvement generally ranges from about 10–36 ft. for light- to heavy-energy applications, respectively.

Slide 23

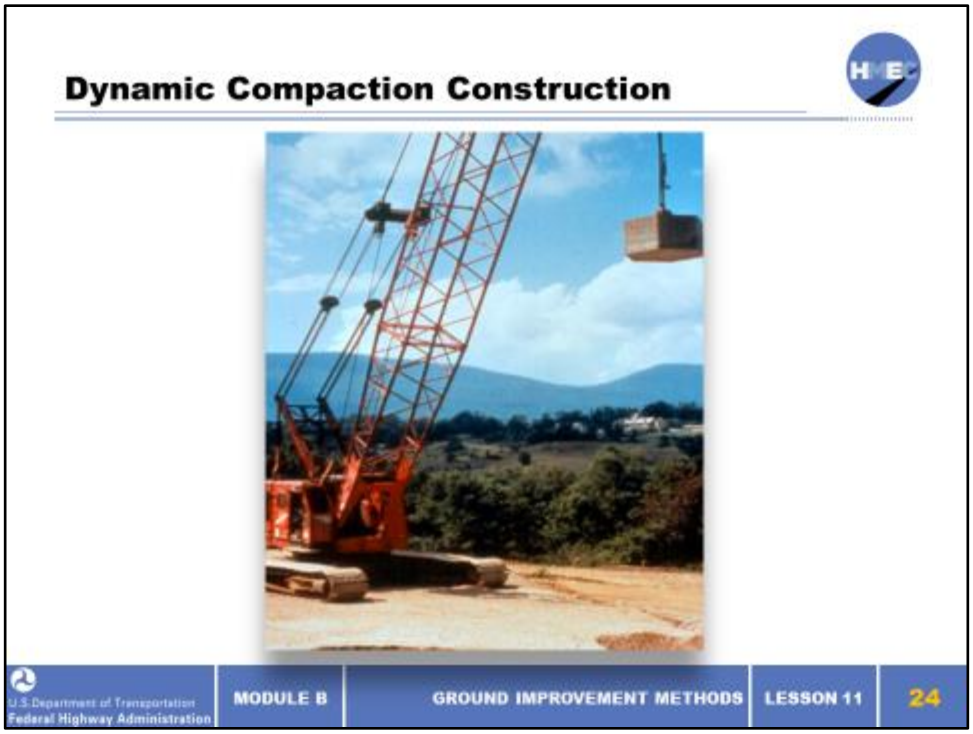


Dynamic compaction densifies the soil mass and this, in turn, improves soil shear strength and reduces compressibility.

Dynamic compaction is most appropriate for unsaturated loose granular soils. It is not appropriate for clays with low permeability. Dynamic compaction can be used to collapse large voids found in landfills and mine spoils.

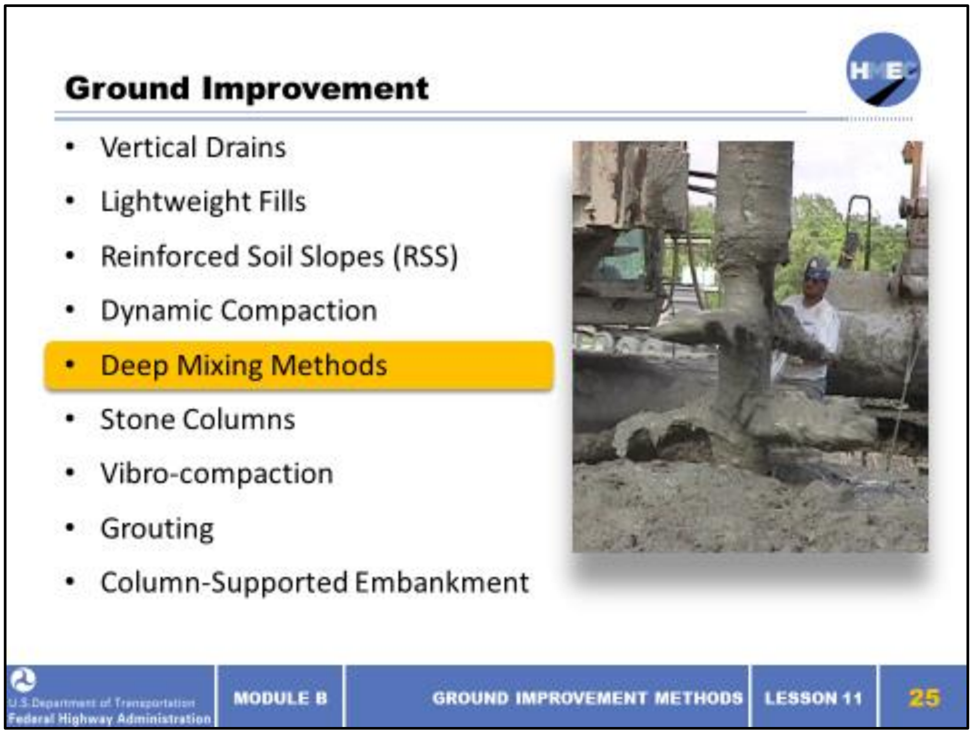
Ground vibration issues may limit the size of tampers (energy) used in urban areas. Lateral displacement can cause damage to underground utilities and drainage pipes.

Slide 24



Following the high-energy level application, the surface of the deposit is in a loose condition to a depth equal to the depth of the craters. This surface is smoothed by filling in the craters and is then compacted on a tight grid basis, with a low-level energy application called an ironing pass.

Slide 25



Ground Improvement
Vertical Drains
Lightweight Fills
Reinforced Soil Slopes (RSS)
Dynamic Compaction
Deep Mixing Methods
Stone Columns
Vibro-compaction
Grouting
Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
25

____

____

____

____

____

____

____

____

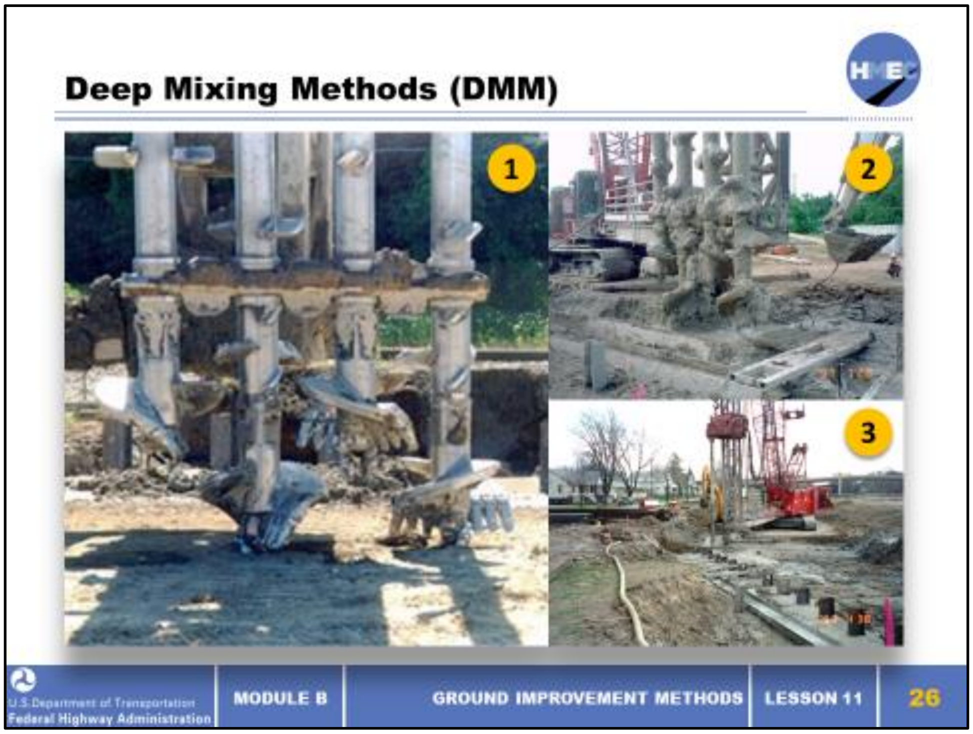
____

____

____

____

Slide 26



DMM can be performed using single or multiple auger systems. DMM can used to construct a cut-off wall with structural beams using the wet mix method as shown in the screen photographs or placed in a grid pattern to reduce settlement or strengthen underlying soil. The photographs clockwise from the top left show: Multiple shaft mixing augers, augers mixing soil with cementing agent using the wet method, and the structural beams inserted into the soil mix columns to anchor tiebacks.

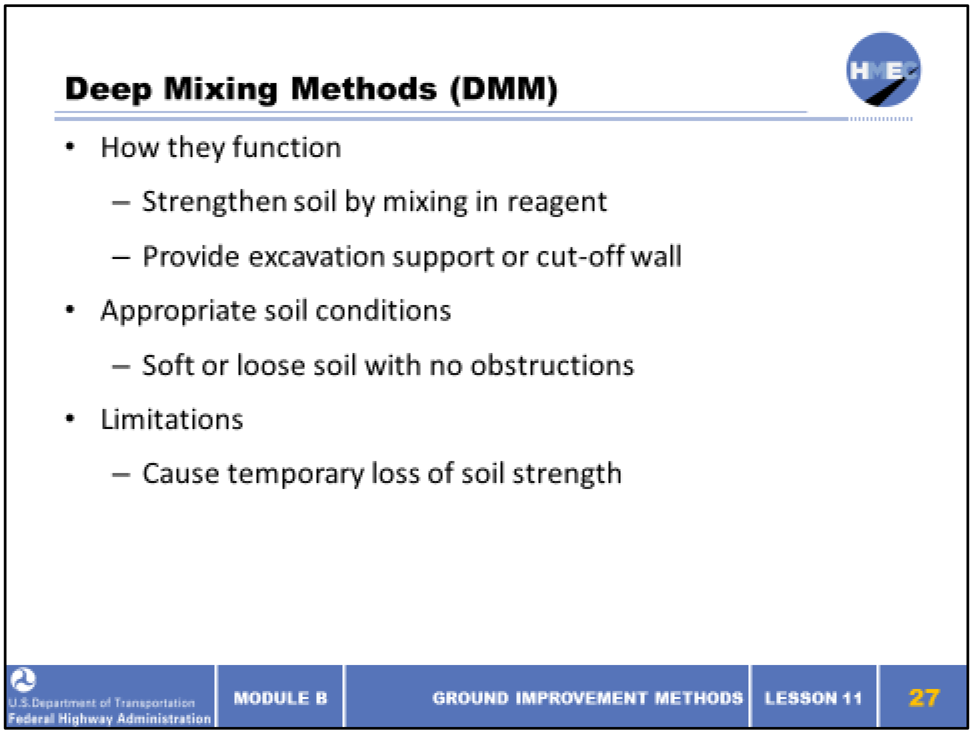
Wet DMM were developed primarily for large-scale structural support and containment, while the dry lime-cement column technique was developed primarily for soil stabilization/reinforcement and settlement reduction. However, as with many ground improvement technologies, the original concepts have been expanded since initial development to encompass a broad range of applications. The use of the dry mixing technique is infrequent in the US.

___

___

___

Slide 27



DMM can be an appropriate ground improvement method for many transportation applications, including embankment stabilization, strengthening foundations soils for bridge abutments and retaining walls, slope failure stabilization, excavation support, retaining walls, and others.

Instruction: The most significant typical usage is for settlement control and/or shear strength improvement under embankments. Under the latter usage, DMM columns are typically constructed in grid or lattice geometry.

DMM walls have the capability to form continuous, low permeability support walls for deep excavations. These walls can accommodate large, steel H-beams placed in every auger hole or in any sequence, as required by wall loadings. The soil-cement in the cylinder acts like lagging between the beams and forms a vertical barrier to groundwater.

DMM wall configurations can be constructed to depths in excess of 100 ft. to serve as groundwater cut-off or containment barriers.

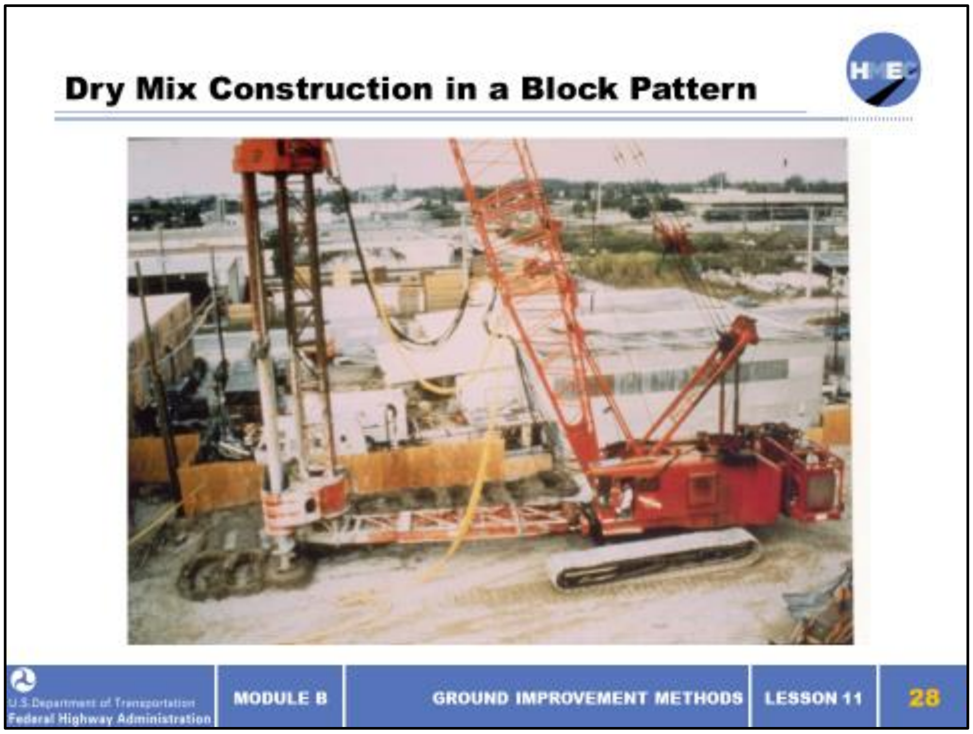
DMM can be used to improve mass shear strength and contain liquefaction propagation.

In general, a soil mixing solution may be most attractive where the site investigation has a soft-to-loose soil profile with no obstructions to depths no greater than 100 ft.

It should be understood that DMM methods cause a temporary loss of shear strength, which might adversely affect existing slopes with marginal stability or cause lateral ground movement to immediately adjacent structures.

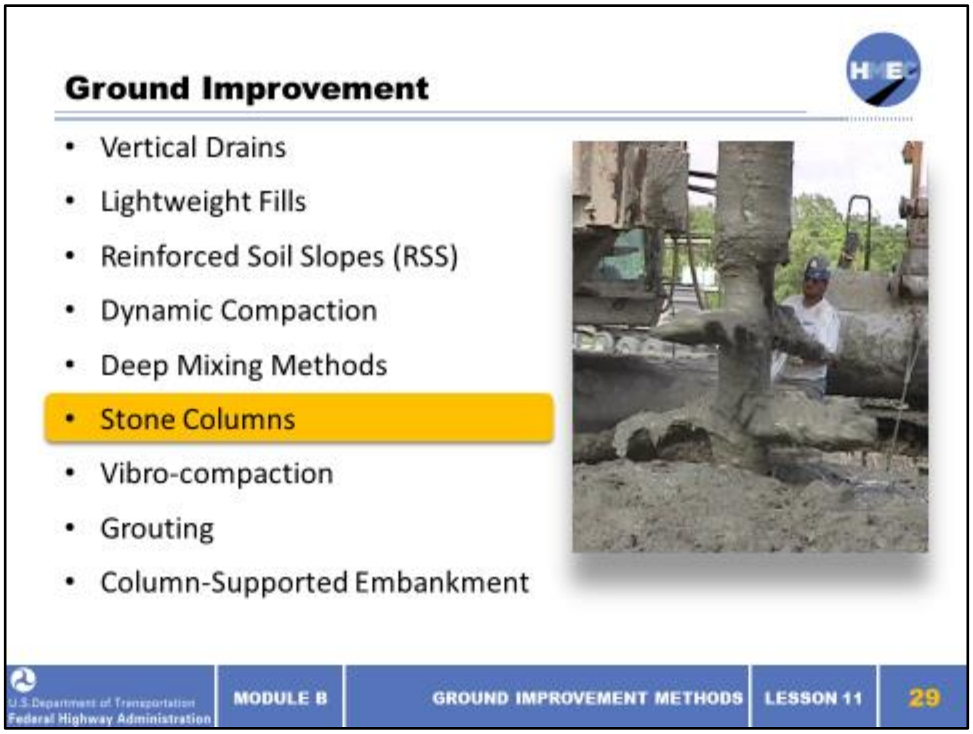
Relatively extensive field and laboratory investigations are normally required to develop design parameters. A recent FHWA publication provides detailed design and analysis methods**.** This publication also provides detailed quality control testing guidance for design verification. Note that the wide range of reported improved properties is a function of the in-situ soils, reagent content, the equipment used for mixing, the in-situ water content, and organic content. Strength has a high degree of variability, but a minimum specified strength should be readily achievable.

Slide 28

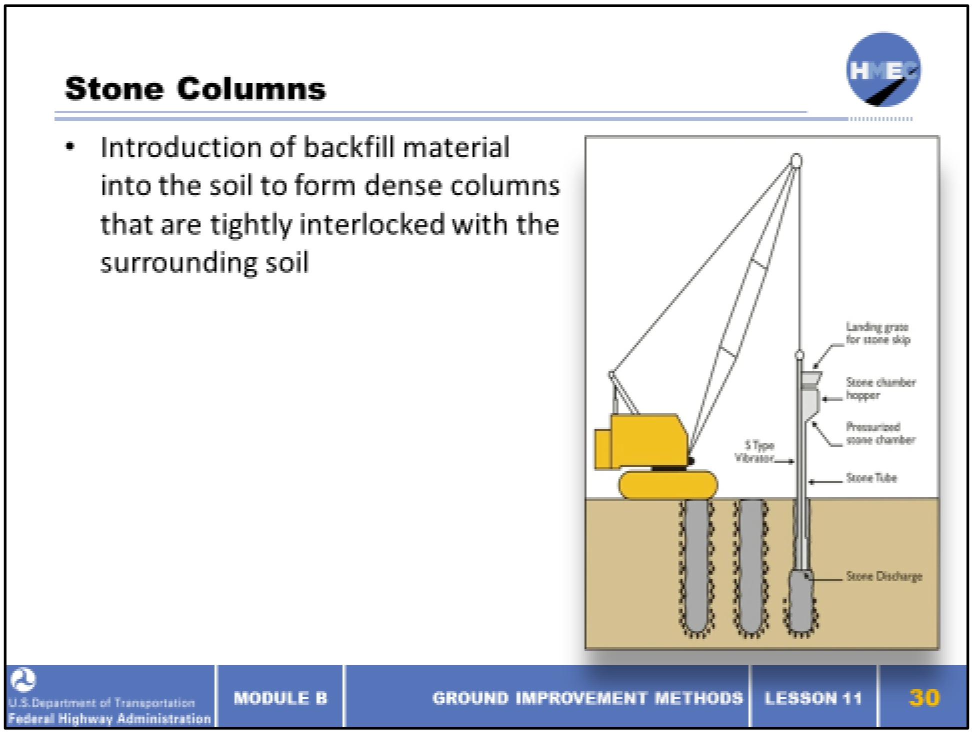


The reagent used is typically a mix of lime and cement. The DMM is performed on a pattern to provide the improvement required. The patterns typically used are block, grid (square or triangular), wall, or areas. In the US, the dry method is used infrequently. However, the wet method can be performed in the same typical patterns to accomplish foundation improvement of soft soil.

Slide 29


Ground Improvement
• Vertical Drains
• Lightweight Fills
• Reinforced Soil Slopes (RSS)
• Dynamic Compaction
• Deep Mixing Methods
• Stone Columns
• Vibro-compaction
• Grouting
• Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
29

Slide 30



The original stone column installation technique, called vibro-replacement or the wet process, uses a high-pressure jet of water to open a hole that the probe follows into the ground. The probe is then retracted in increments, and stone is introduced into the void from the surface. After every increment, the probe is lowered into the new column material, thereby densifying and compacting the stone column and, potentially, the surrounding soil (depending on percent fine content). This method is best suited for sites with soft to firm soils with undrained shear strengths of 200-1000 psf and a high groundwater table.

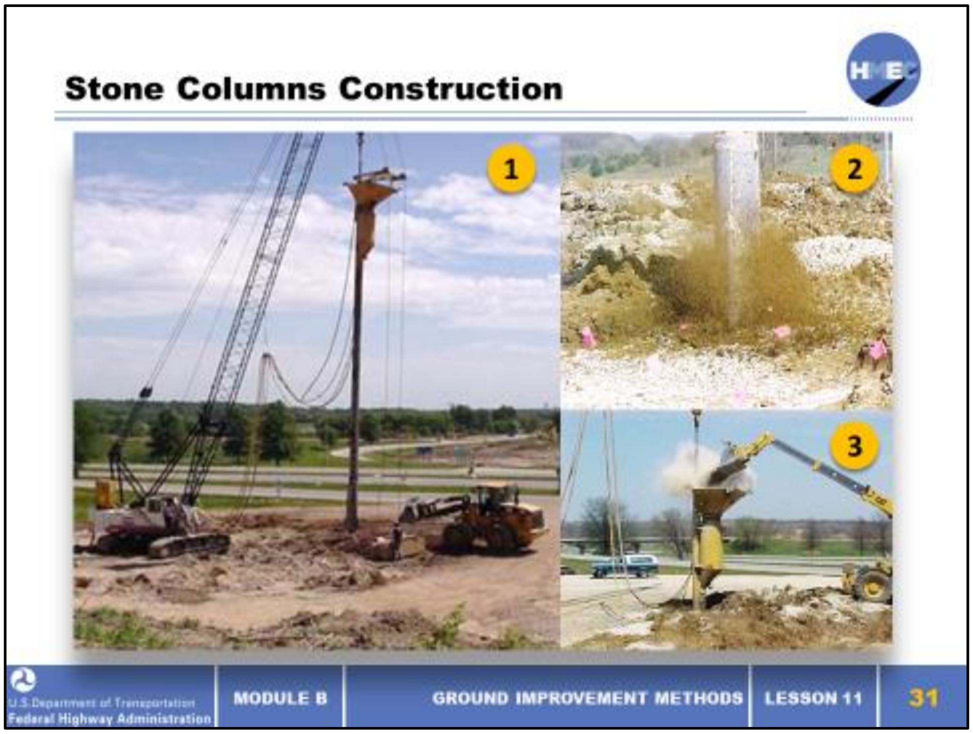
If time is critical to project start-up, site improvement by stone column installation can be achieved quicker than by pre-loading the soils. Where the infrastructure precludes high-vibration techniques, such as dynamic compaction, deep blasting, or piling, the low-vibration stone column technique is often viable. In seismic areas, stone columns can densify the soils beyond the threshold of liquefaction. Stone columns also provide a vertical drainage path for excess pore water pressure dissipation, as well as densifying the soils.

____

____

____

Slide 31



Crane supported vibrator with aggregate hopper on top beginning penetration. Vibrator penetrating ground surface. Filling aggregate hopper with vibrator at full depth.

This project included a section that was improved with stone columns and a section that was improved with Geopiers. Mention that Geopiers are a proprietary rammed aggregate column ground improvement. We won't discuss that method here; however, it is discussed on the GeoTechTools Web site.

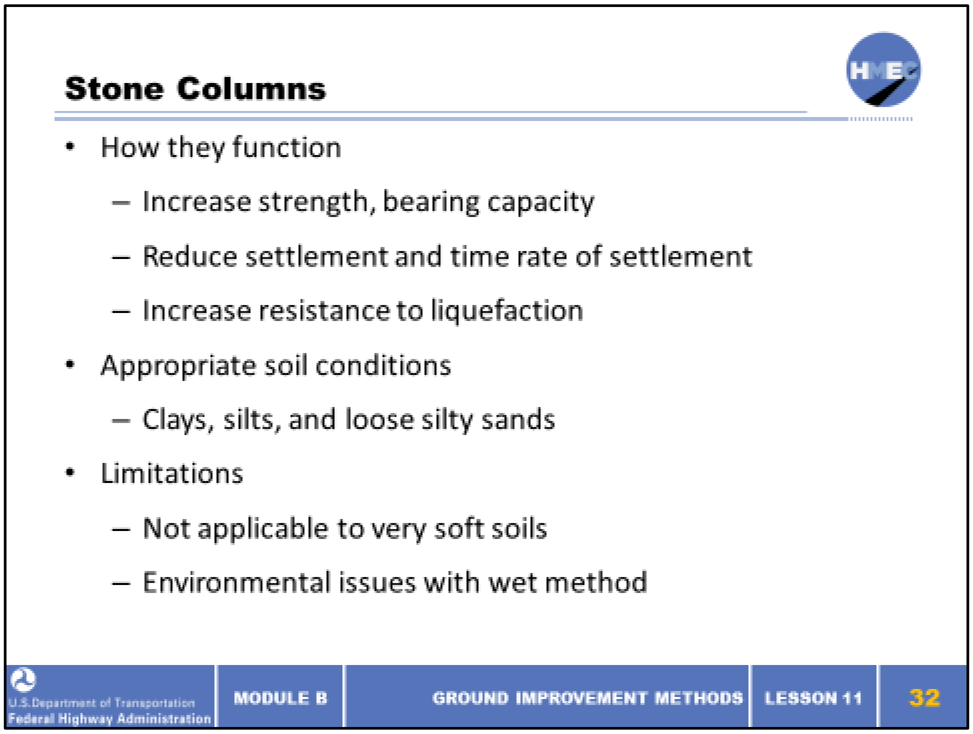
Case Study Notes: Photos and note excerpts are from Highway Applications for Rammed Aggregate Piers in Iowa Soils
Sponsored by the Iowa Department of Transportation and the Iowa Highway Research Board
Final Report, April 2003 Iowa DOT Project TR-443 CTRE Project 00-60

For the embankment foundation project, Geopier elements were installed within and around an abutment footprint for the new I-35 overpass at the US Highway 5/Interstate 35 interchange in Des Moines, Iowa. Although the main focus of this investigation was to evaluate embankment foundation reinforcement using Geopier elements, a stone column reinforced soil provided an opportunity to compare systems. In situ testing included cone penetration tests (CPTs), pressure meter tests (PMTs), Ko stepped blade tests, and borehole shear tests (BSTs), as well as laboratory material testing. Comparative stiffness and densities of Geopier elements and stone columns were evaluated based on full-scale modulus load tests and standard penetration tests. Vibrating wire settlement cells and total stress cells were installed to monitor settlement and stress concentration on the reinforcing elements and matrix soil. Settlement plates were also monitored by conventional optical survey methods. Results show that the Geopier system and the stone columns performed their intended functions.

For the embankment foundation project, Geopier elements were installed within and around an abutment footprint for the new I-35 overpass at the US Highway 5/Interstate 35 interchange in Des Moines, Iowa. The Geopier elements were installed to reduce total settlement and increase the rate of settlement of a 5–6 m alluvial clay layer beneath the proposed 8 m fill. The goal was to reduce construction delay between embankment and abutment construction from 120 days to less than 30 days.

At an adjacent embankment foundation site, stone columns were installed in the foundation soils to increase the factor of safety against global slope instability at the interface between a silty alluvium layer and weathered shale bedrock. At the test site, stone columns were installed to depths ranging from 3–14 m to reduce settlement and increase the factor of safety against global slope instability prior to construction of a 9 m high bridge approach embankment.

Slide 32



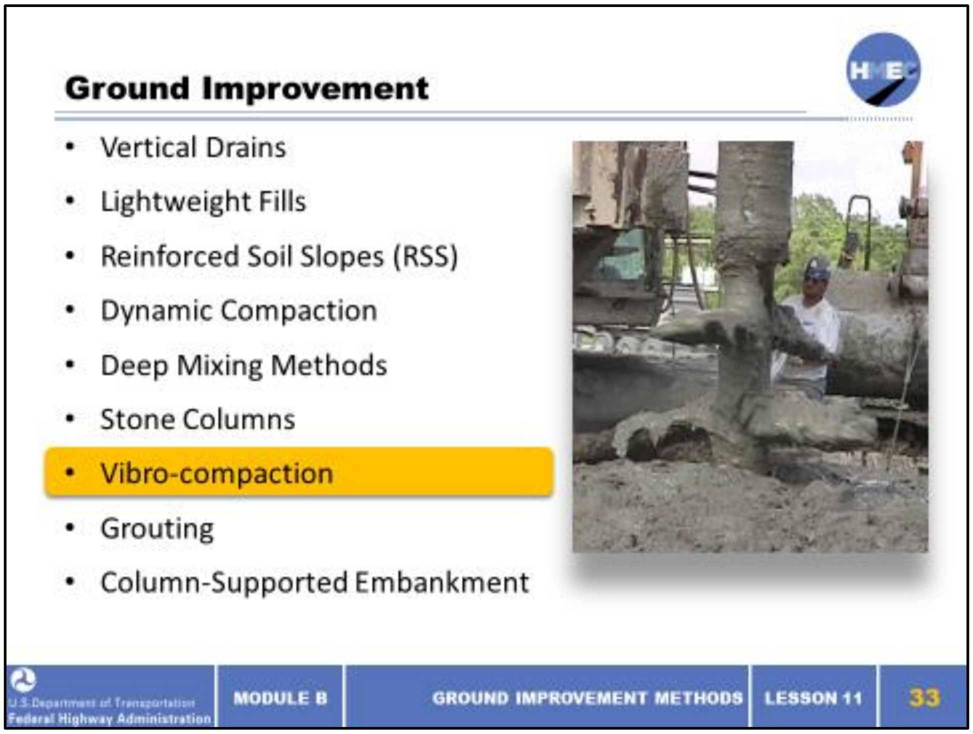
The technique can be applied to improving slope stability, increasing bearing capacity, reducing total and differential settlements, reducing the liquefaction potential of soil, and accelerating the time rate of settlement. Typical applications include foundation improvement for the construction of highways, embankments, warehouses, and light industrial buildings.

Stone columns have a proven record of experience and are ideally suited for improving clays, silts, and loose silty sands.

Stone columns are not a solution for all soft soil problems. Strata of peat and other organic materials, and very soft clays with a thickness greater than the diameter of the stone column can be inappropriate for stone column construction, as they offer inadequate lateral support.

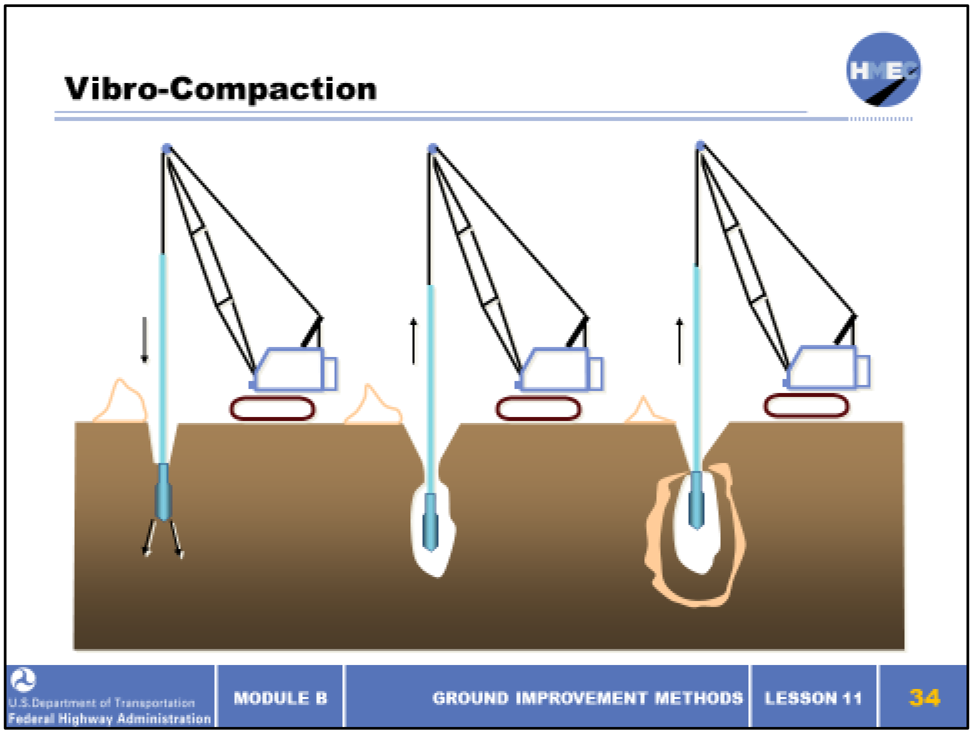
Stone columns are traditionally jetted in place, thus removing the finer portions of the influenced soil. The resulting fines-laden jetted water has to be temporarily contained to allow for sediment deposition and disposal. Jurisdictions have varying regulations regarding the processes for these operations. Also, unknown contaminants may be removed and transferred to the environment by the jetting water.

Slide 33



Ground Improvement
HMEC
- Vertical Drains
- Lightweight Fills
- Reinforced Soil Slopes (RSS)
- Dynamic Compaction
- Deep Mixing Methods
- Stone Columns
- Vibro-compaction
- Grouting
- Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
33

Slide 34



Vibro-compaction is a ground improvement technique that uses specially designed probe-type, depth vibrators for in-situ densification of loose sands and gravels. Originally called Vibroflotation, the process was accomplished with water jetting, hence the name. Subsequent equipment development featuring higher horsepower and amperage made a dry operation possible. The majority of vibro-compaction projects, however, are accomplished by the jetting water (wet) method.

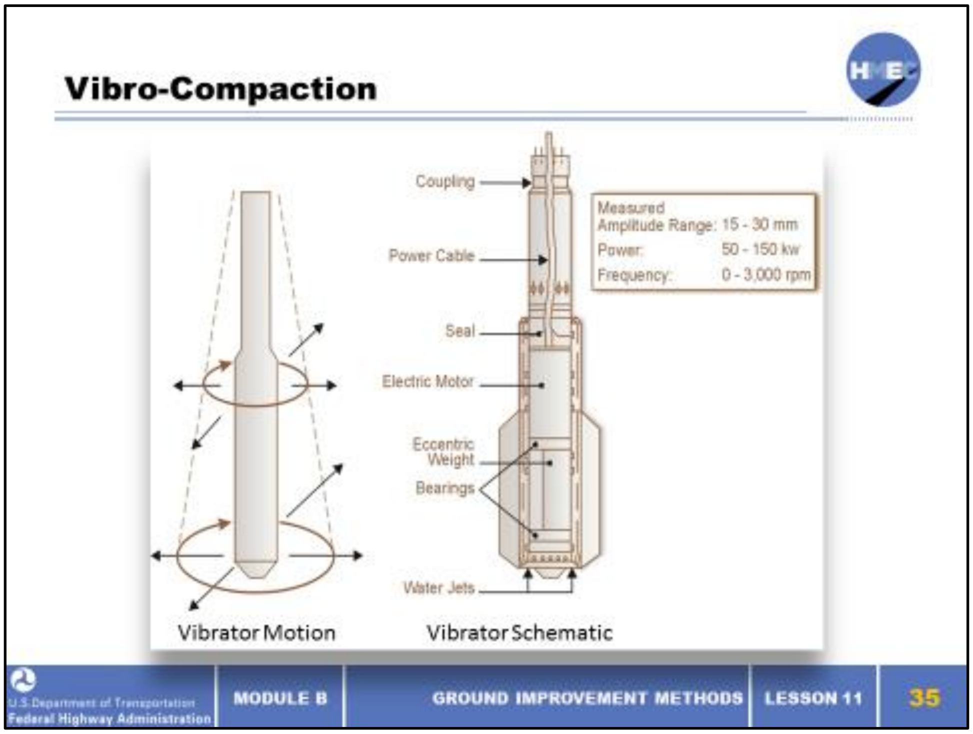
The mechanism of densifying granular, cohesionless soils with vibrators can be briefly described as follows: Mechanical vibrations and simultaneous application of water nullify the effective stresses between the soil grains, which are rearranged, unconstrained, and unstressed under the action of gravity to the densest possible state, thus providing permanent compaction. In the immediate vicinity of the vibrator, the soil is saturated, liquefies locally and temporarily under the influence of the vibrations.

____

____

____

Slide 35



A set of rotating eccentric weights housed inside the probe is mounted on a vertical shaft. Vibrations (induced by rotating these weights) are produced close to the bottom of the unit. The vibrations produced by these units are generated at the nose of the unit and, as a result of the rotation of the weights, emanate radially in the horizontal plane away from the unit.

The vibro-compaction operation necessitates the use of water- or air-jetting to facilitate the penetration of the vibrator and to densify the soil. Therefore, water- or air-feed hoses, as well as water or air pumps, are also required.

____

____

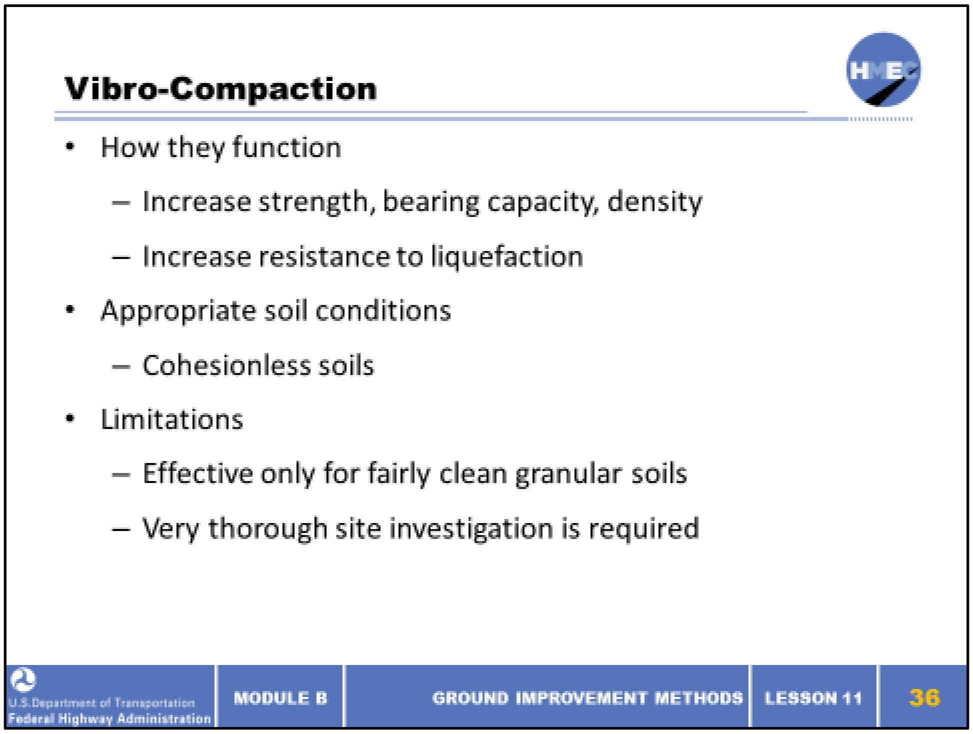
____

____

____

____

Slide 36



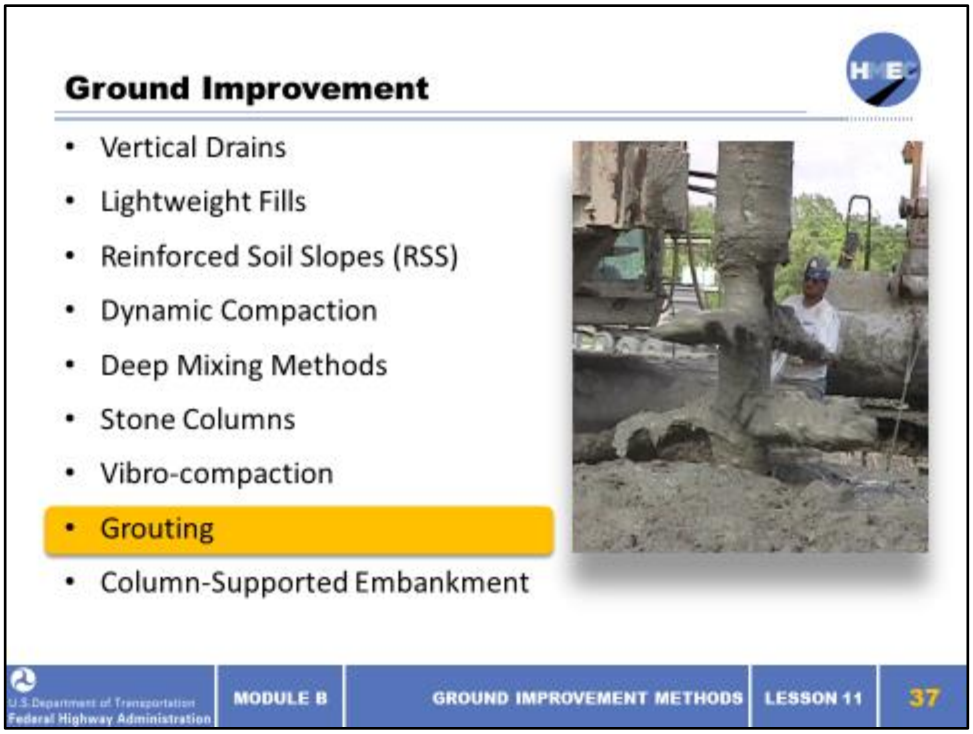
The vibrations of the probe cause the grain structure to collapse thereby densifying the soil surrounding the probe. The void ratio and compressibility of the soil treated by vibratory means will be decreased, and the angle of shearing resistance increased. Liquefaction potential can be reduced by vibro-compacting loose, granular soil to a density beyond the threshold density triggering liquefaction.

As with any ground improvement technique, vibro-compaction has its limitations. The improved in-situ soil characteristics depend on the in-situ soil type and its gradation, the spacing of the compaction points, the characteristics of the equipment used, and the compaction duration. All of these factors affect the outcome of the project.

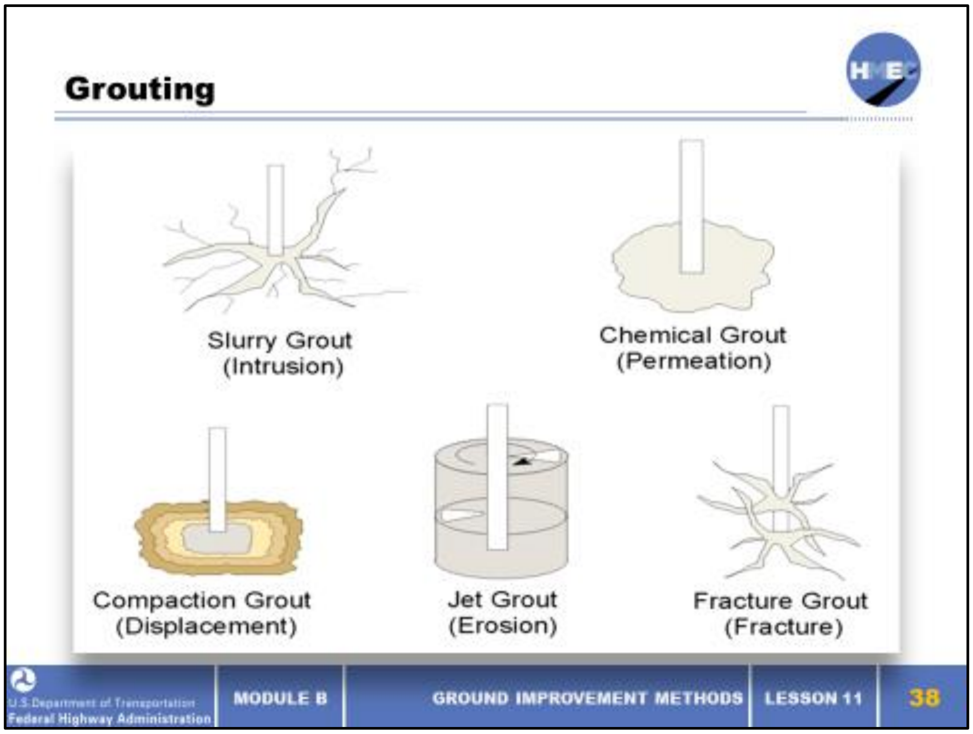
The major disadvantage of vibro-compaction is that it is effective only in granular, cohesionless soils. The realignment of the sand grains and, therefore, proper densification generally cannot be achieved when the granular soil contains more than 12-15% silt or more than 2% clay. The maximum depth of 165 ft. may be considered a disadvantage, but there are very few construction projects that will require densification to a greater depth.

A comprehensive understanding of the total soil profile is therefore necessary. A vibro-compaction investigation will require continuous standard penetration tests (SPT), and/or cone penetrometer (CPT), as well as gradation tests to verify that the soils are suitable for vibro-compaction

Slide 37


Ground Improvement
• Vertical Drains
• Lightweight Fills
• Reinforced Soil Slopes (RSS)
• Dynamic Compaction
• Deep Mixing Methods
• Stone Columns
• Vibro-compaction
• Grouting
• Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
37

Slide 38



It must be realized that the field of grouting is extremely large, and continues to grow at an accelerating pace. Engineers considering a grouting project are strongly encouraged to seek expert advice.

Rock grouting (a type of intrusion grouting): Uses primarily cement grout to alter in-situ characteristics of weak or permeable rock. Injected to displace water and air in fissures and cracks. When the grout sets, the rock is relatively impermeable and is stronger.

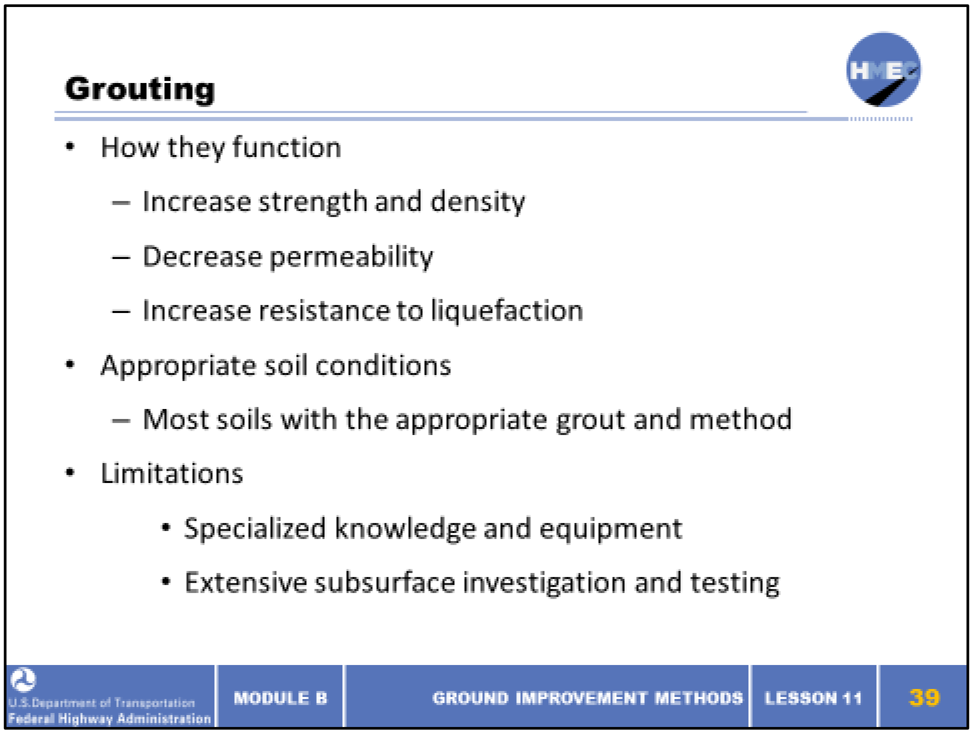
Permeation grouting either with particulate or chemical grouts is utilized to give cohesion and/or to reduce the permeability of the soils by changing the structure or volume of the virgin soil mass. The type of grout utilized will depend on the grain size of the in-situ soil and the results derived from the grouting operation.

Compaction grouting: Injection of very stiff, “zero-slump” mortar grout to displace and compact soils in place.

Jet grouting: The different types of jet grouting are intended to transform soils into a mixture of soil and cement, typically referred to as “soilcrete.” Jet grouting permits the shape, size, and properties of these treated masses, usually circular columns, to be engineered in advance, with an increasingly high degree of precision.

Soil fracture grouting: Injection of a fluid, cement-based grout to deliberately fracture coarse and fine grained soils to strengthen them and, if desired, to raise settled structures

Slide 39

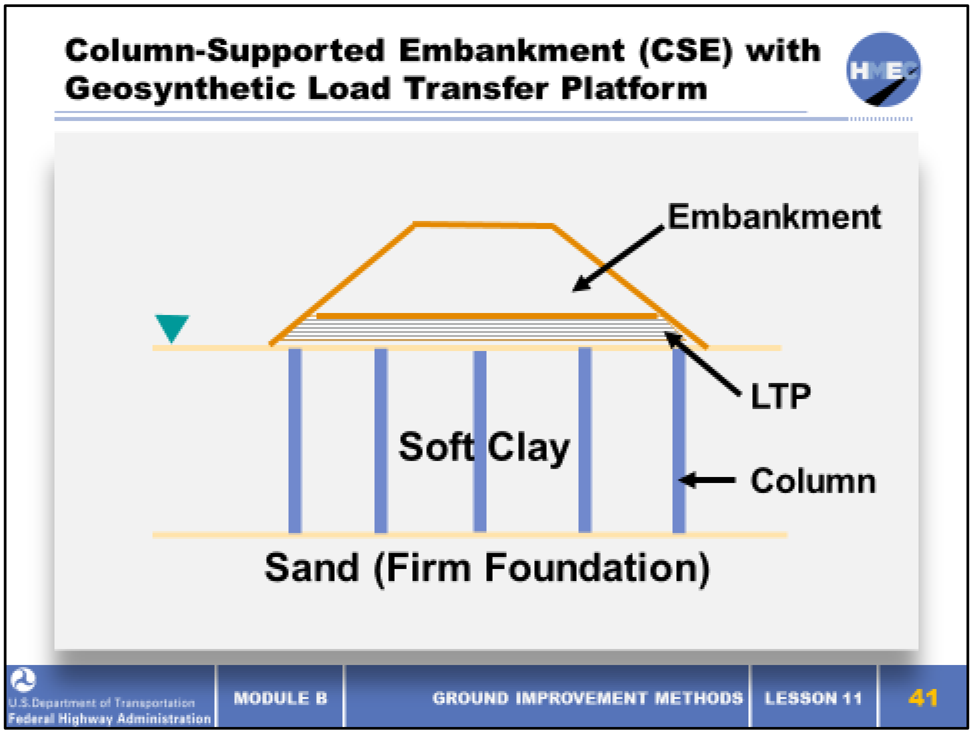


By selecting the correct method and grout components, grouting can be used to increase strength and density of the soil, reduce permeability, and/or increase resistance to liquefaction. Grouting can be used to improve almost any soil type. The only drawback is the highly specialized equipment and knowledge that is required to achieve a quality grouting project.

Slide 40


Ground Improvement
• Vertical Drains
• Lightweight Fills
• Reinforced Soil Slopes (RSS)
• Dynamic Compaction
• Deep Mixing Methods
• Stone Columns
• Vibro-compaction
• Grouting
• Column-Supported Embankment
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
40

Slide 41



Column-supported embankments consist of vertical columns that are designed to transfer the load of the embankment through the soft compressible soil layer to a firm foundation. The selection of the type of column used for the CSE will depend on the design loads, constructability of the column, cost, etc. There is a wide range of columns that may be used for CSEs. Conventional (i.e., timber, steel H, steel pipe, pre-cast concrete, and cast in-place concrete shell) piles may be used for pile supported embankments. However, conventional piles, with the exception of timber piles, have a rather high structural capacity 45–225 tons that is seldom required for CSE and are, therefore, economically not as attractive as non-traditional columns. Additional elements that have been used for columns in CSEs include soil mix columns, stone columns, geotextile encased columns (GEC), geopier rammed aggregate piers, and vibro-concrete columns (VCC).

In order to minimize the number of columns required to support the embankment and increase the efficiency of the design, a geosynthetic reinforced load transfer platform (LTP) may be used. The load transfer platform consists of one or more layers of geosynthetic reinforcement placed between the top of the columns and the bottom of the embankment.

Slide 42

The new project required an embankment to be built adjacent to an existing bridge that was supported on piles. The owner agency was concerned that consolidation of a soft layer under the planned embankment would overload the existing piling because of down drag. The embankment was vertical next to the existing bridge abutment.

____

____

____

____

____

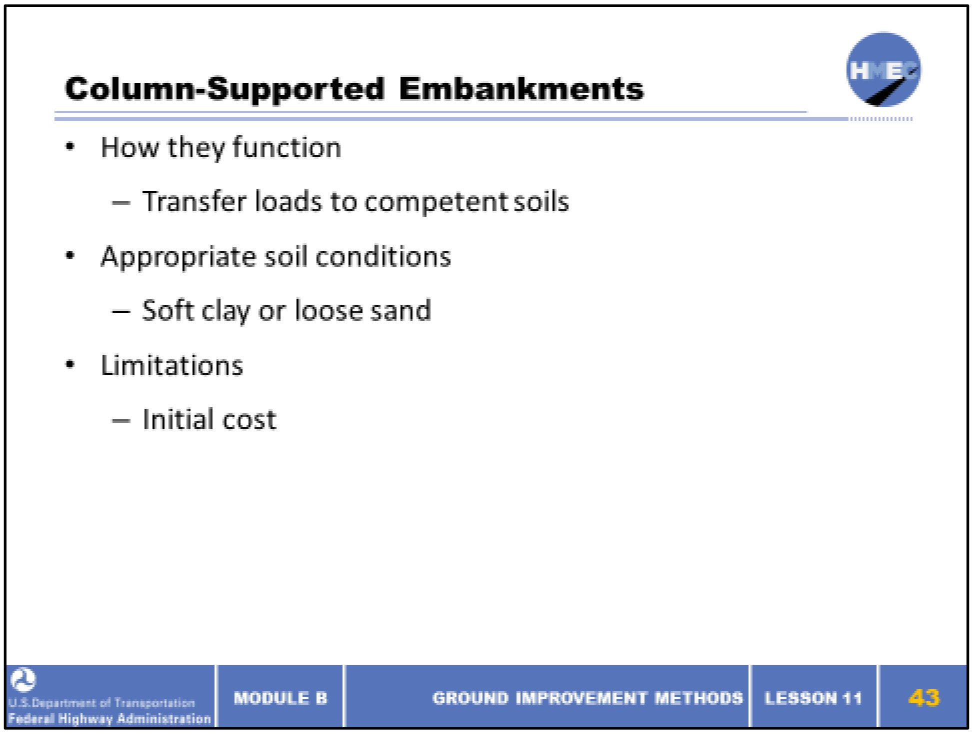
____

____

____

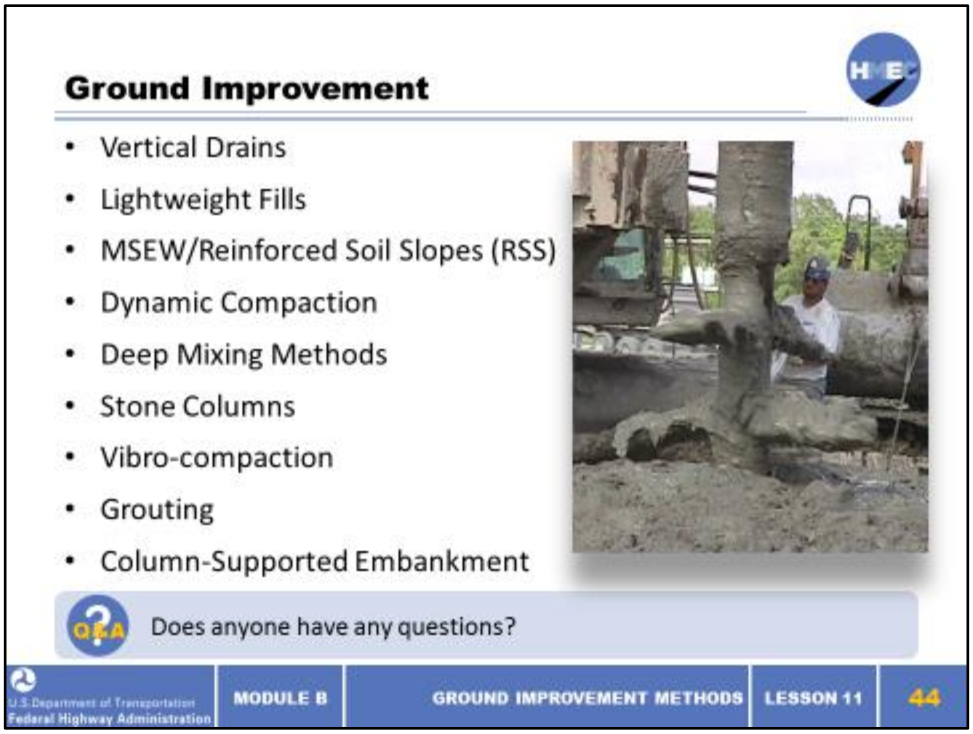
____

Slide 43

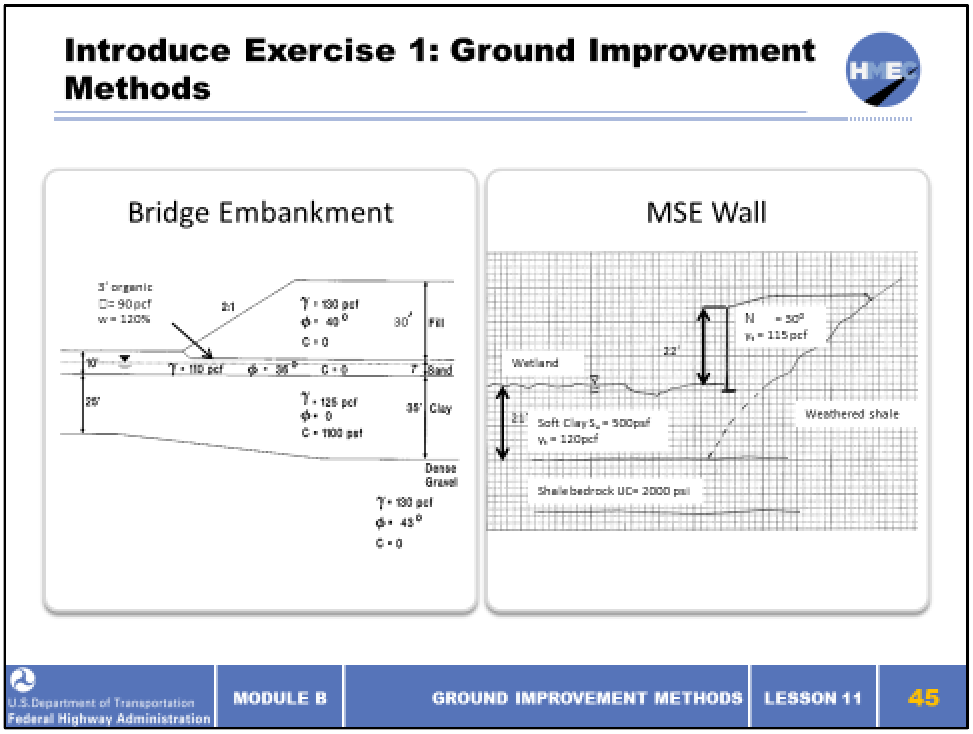


CSE functions by transferring the embankment load to a competent foundation soil below the compressible layer. CSE may be an appropriate solution for compressible clay layers that will take too long to consolidate. CSE may also be appropriate for loose sand layers where settlement will cause down drag loads on existing structures. Although the sand layer will settle immediately upon placement of the embankment, the settlement may cause intolerable down drag forces on existing piling. CSE can be used to decrease the amount of settlement to a tolerable level. One disadvantage of the CSE method is high initial cost. Although if the time saved by not having to wait for consolidation to take place is critical, CSE may be the appropriate selection

Slide 44


Ground Improvement
Vertical Drains
Lightweight Fills
MSEW/Reinforced Soil Slopes (RSS)
Dynamic Compaction
Deep Mixing Methods
Stone Columns
Vibro-compaction
Grouting
Column-Supported Embankment
Does anyone have any questions?
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
44

Slide 45


Introduce Exercise 1: Ground Improvement Methods
HMEC
Bridge Embankment
3' organic
= 90pcf
w = 120%
2:1
γ = 130 pcf
ϕ = 40°
C = 0
30'
Fill
10'
γ = 110 pcf
ϕ = 36°
C = 0
7'
Sand
25'
γ = 125 pcf
ϕ = 0
C = 1100 psf
35'
Clay
Dense Gravel
γ = 130 pcf
ϕ = 43°
C = 0
MSE Wall
N = 30°
γt = 115pcf
22'
Wetland
21'
Soft Clay Su = 500psf
γt = 120pcf
Weathered shale
Shalebedrock UC= 2000 psi
U.S.Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
45

____

____

____

____

____

____

____

____

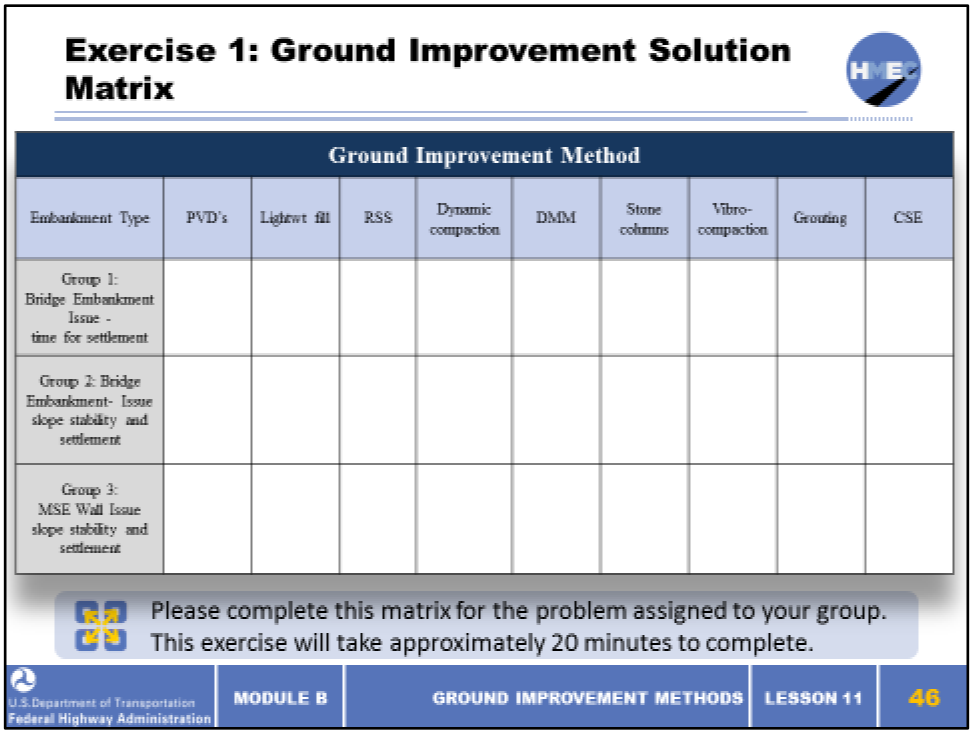
____

____

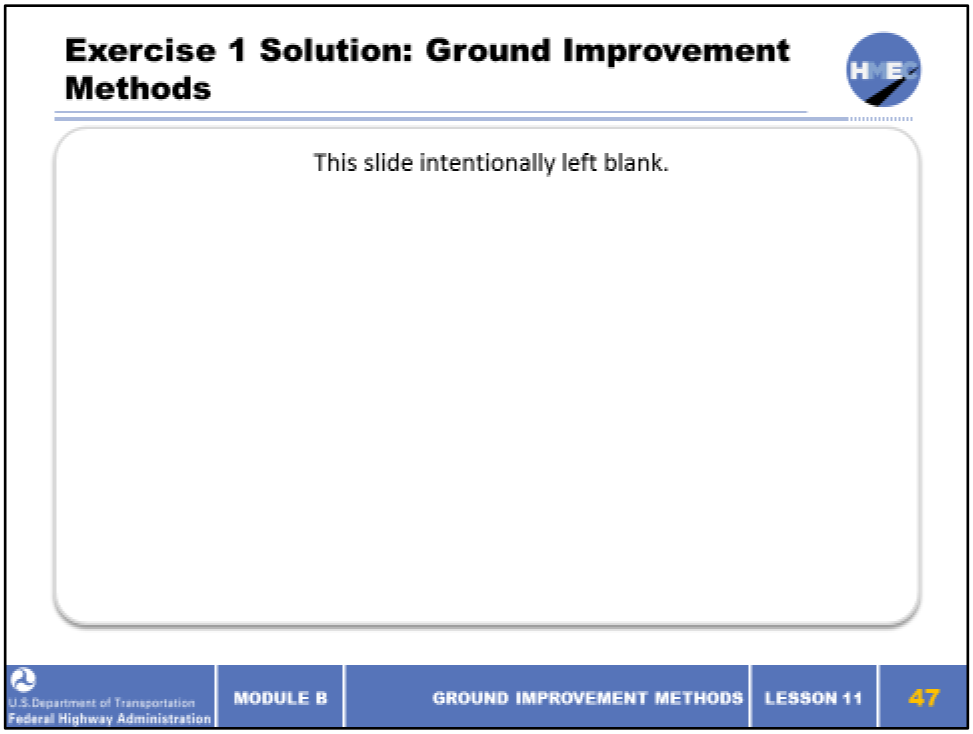
____

____

Slide 46


Exercise 1: Ground Improvement Solution Matrix
HMEC
Ground Improvement Method
Embankment Type
PVD's
Lightwt fill
RSS
Dynamic compaction
DMM
Stone columns
Vibro-compaction
Grouting
CSE
Group 1: Bridge Embankment Issue - time for settlement
Group 2: Bridge Embankment- Issue slope stability and settlement
Group 3: MSE Wall Issue slope stability and settlement
Please complete this matrix for the problem assigned to your group.
This exercise will take approximately 20 minutes to complete.
U.S.Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
46

Slide 47


Exercise 1 Solution: Ground Improvement Methods
This slide intentionally left blank.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GROUND IMPROVEMENT METHODS
LESSON 11
47

Slide 48

Learning Outcomes Review

You are now able to:

- Describe available ground improvement methods.
- Describe how to select appropriate ground improvement methods.

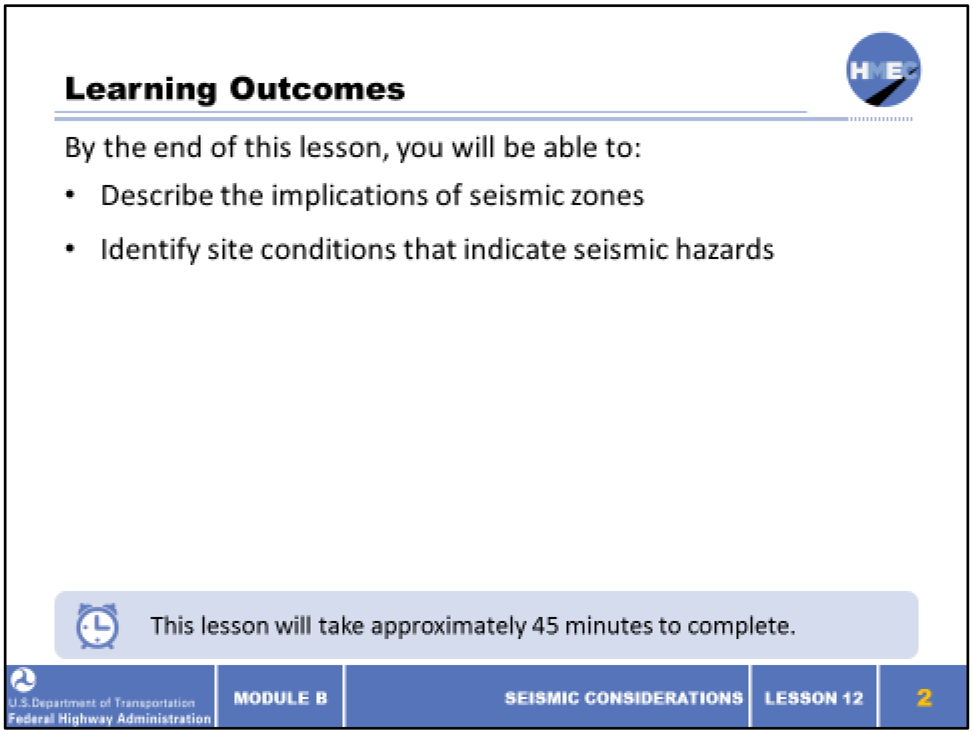
U.S. Department of Transportation Federal Highway Administration | MODULE B | GROUND IMPROVEMENT METHODS | LESSON 11 | 48

This page is intentionally blank.

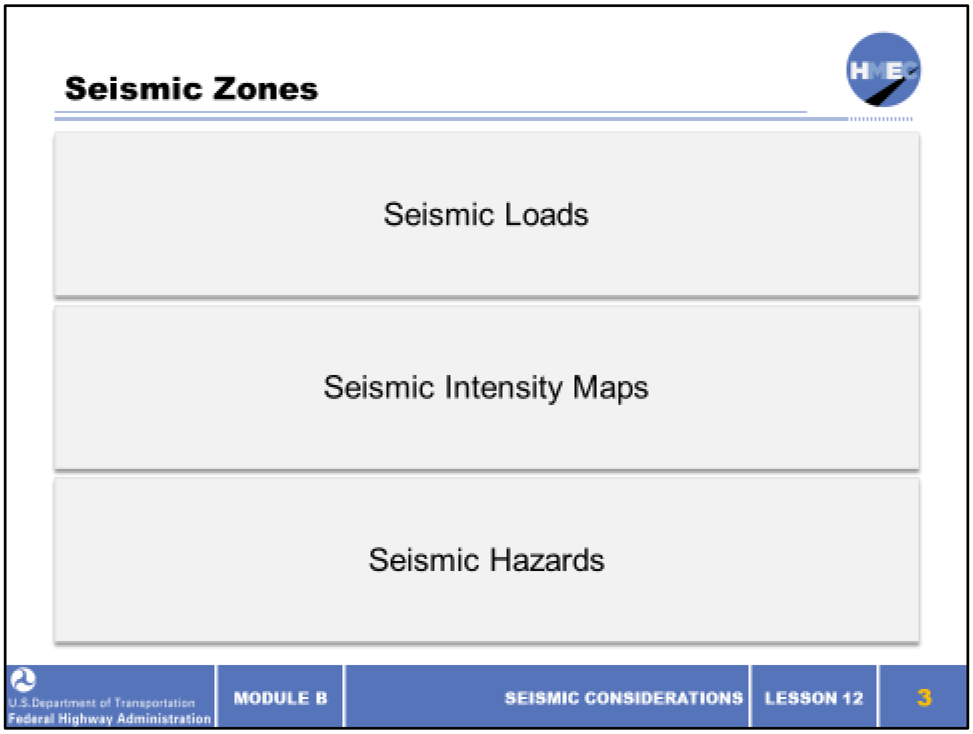
Slide 1


HMEC
Highway Materials Engineering Course
Lesson 12: Seismic Considerations
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

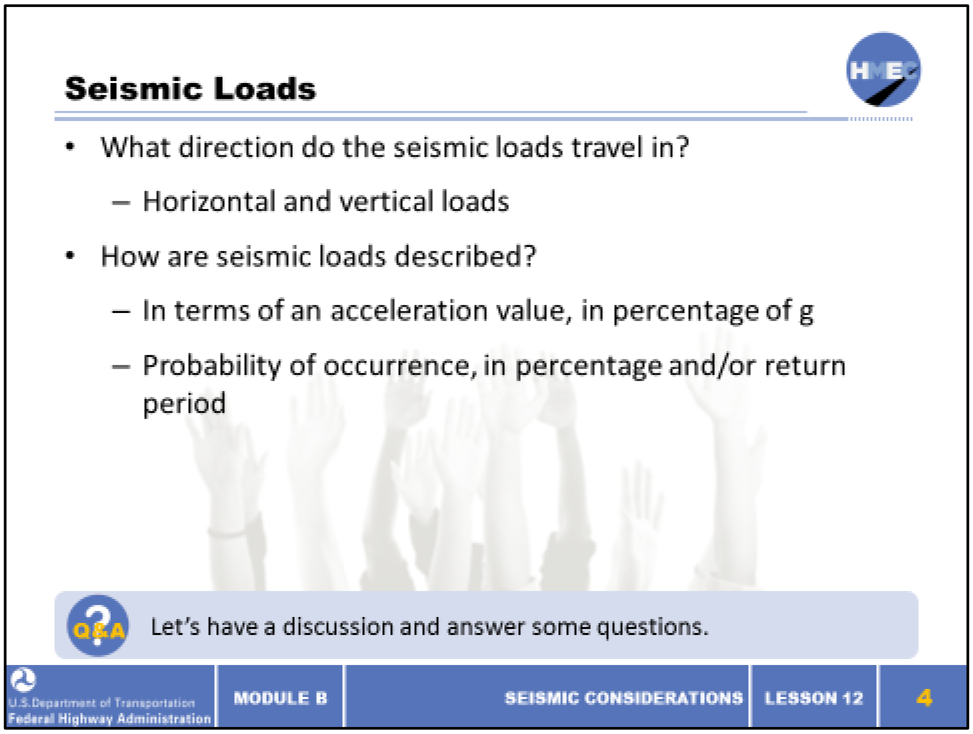
Slide 2


Learning Outcomes
HMEC
By the end of this lesson, you will be able to:
Describe the implications of seismic zones
Identify site conditions that indicate seismic hazards
This lesson will take approximately 45 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
SEISMIC CONSIDERATIONS
LESSON 12
2

Slide 3

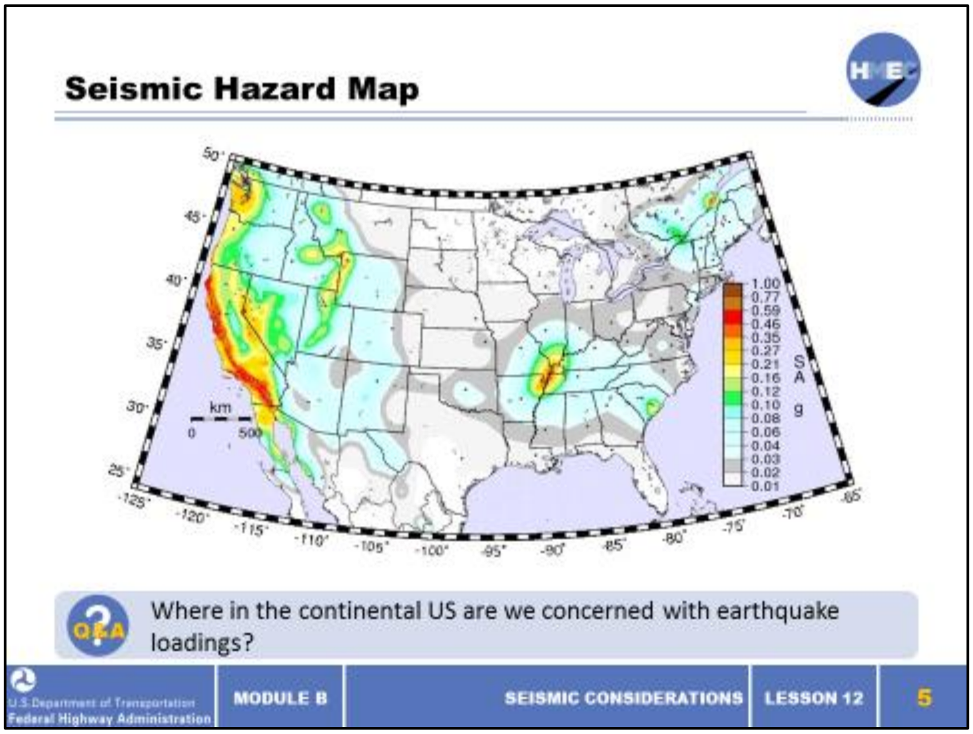

Seismic Zones
HMEC
Seismic Loads
Seismic Intensity Maps
Seismic Hazards
U.S. Department of Transportation
Federal Highway Administration
MODULE B
SEISMIC CONSIDERATIONS
LESSON 12
3

Slide 4



What is a seismic load? How do we define or quantify it?

Slide 5



While earthquakes are sometimes considered primarily a California or west coast problem in the continental US, damaging earthquakes are not limited to the western US. In fact, some of the strongest earthquakes in US history have occurred in the central and eastern US. The Charleston, South Carolina earthquake of 1886 is believed to have been as strong, if not stronger, than the 1971 San Fernando and 1994 Northridge earthquakes. And there were three large magnitude earthquakes, including at least one believed to have been as strong as the 1906 San Francisco earthquake, in the New Madrid seismic zone in the central United States in 1811 and 1812.

This map is from http://earthquake.usgs.gov/hazards/products/conterminous/2008/maps

____

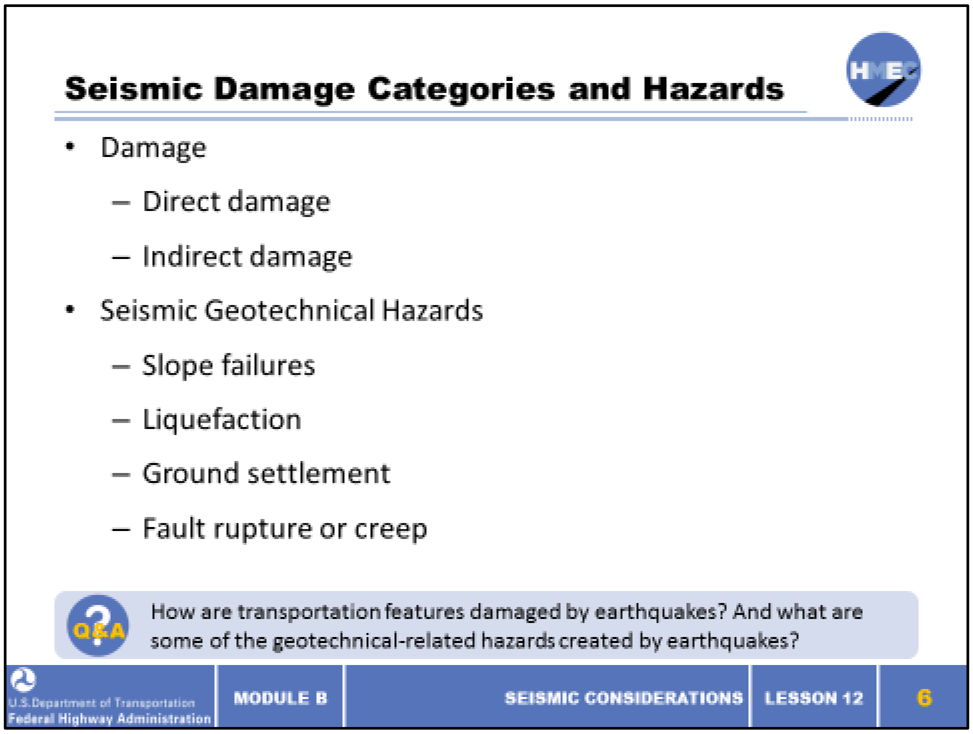
____

____

____

____

Slide 6



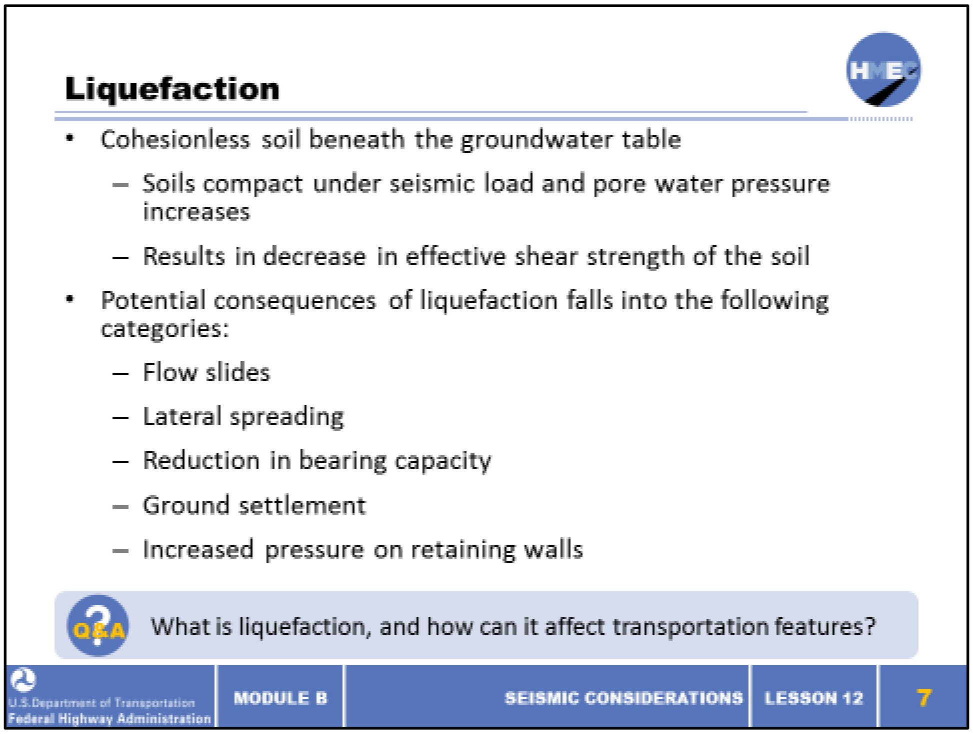
Damage from earthquakes can be subdivided into direct damage and indirect damage. Direct damage is the physical damage due to the earthquake. Direct damage includes primary damage due to strong shaking and fault rupture, and secondary damage due to the effects of strong shaking or fault rupture. Indirect damage refers to the socio-economic impacts of an earthquake. Primary damage due to strong shaking includes partial or total collapse of structures, landslides, and liquefaction—as illustrated in some of the following slides. Indirect damage is, for example, the impact of a slope failure or impact liquefaction. The economic component of indirect damage often exceeds the economic consequences of direct damage from a major earthquake.

Geotechnical seismic hazards are related to slope stability, liquefaction, ground settlement, and fault rupture. Earthquake-induced slope stability and bearing failures may damage bridge foundations or superstructures, block highways, or rupture pipelines or culverts. Earthquake-induced liquefaction has been a major source of damage to bridge structures and non-bridge transportation facilities in past earthquakes.

____

____

Slide 7



Soil liquefaction is a phenomenon in which a cohesionless soil deposit below the groundwater table loses a substantial amount of strength due to pore pressure generation resulting from strong earthquake ground shaking. Loose sands and silty sands are particularly susceptible to liquefaction.

Liquefaction has been perhaps the single most significant cause of damage to bridges during past earthquakes. Most of the damage has been related to liquefaction-induced lateral movement of soil at bridge abutments.

Flexible retaining walls have demonstrated more survivability during seismic events than rigid retaining walls.

The potential consequences of liquefaction can be grouped into the following categories:

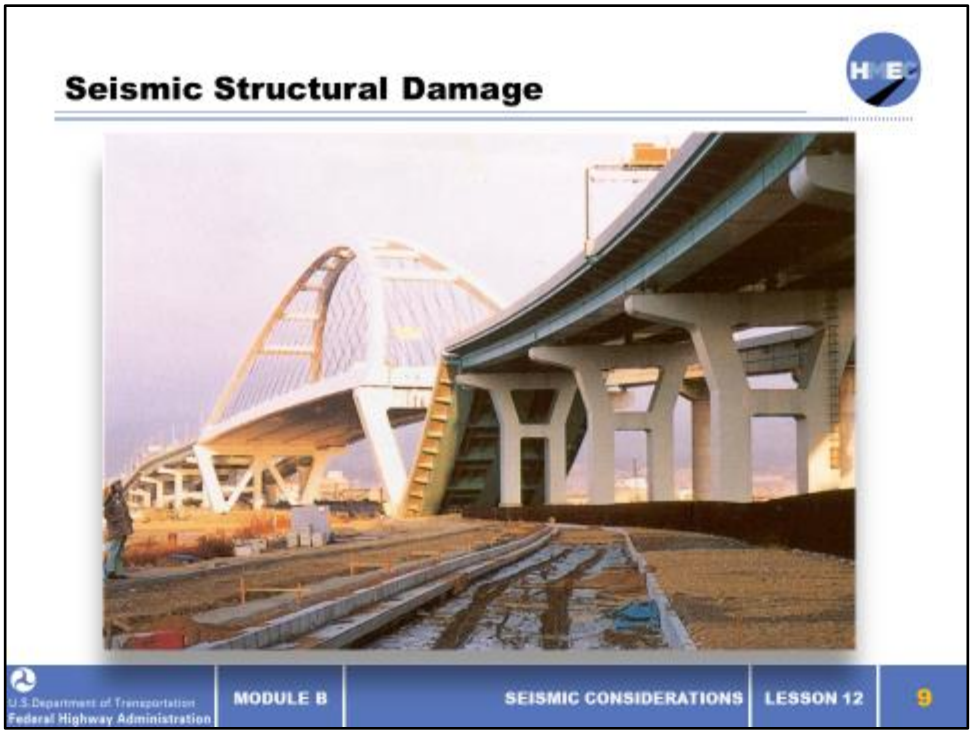
• Flow slides. These large slides occur when the down slope static loads exceed the resistance provided by low shear strengths of liquefied soils. These slides can occur even after the ground stops shaking, and commonly results in tens of feet of displacement.

• Lateral spreading. It can occur on very gently sloping ground or under embankments underlain by liquefied soil due to combined static gravity and seismically induced inertia forces in the soil mass. The resulting lateral movements can range in magnitude from inches to several feet and are typically accompanied by ground cracking with horizontal and vertical offsets.
• Reduction in foundation bearing capacity. The occurrence of liquefaction beneath, and/or laterally adjacent to, bridge foundations can greatly reduce foundation vertical and/or lateral capacity, resulting in unacceptable foundation settlements and or lateral movements.
• Ground settlement. Ground settlements due to soil consolidation can occur as liquefaction-induced; excess pore water pressures in the soil dissipate. In some cases, this consolidation may take a substantial period of time to occur, perhaps hours or, in extreme cases, days after the earthquake, and may result in unacceptable total and/or differential settlement of foundations located above the liquefied layer.
• Increased pressure on retaining walls. Liquefaction in the backfill behind a retaining wall will increase lateral earth pressures on the wall, potentially leading to wall failure or excessive deformations.

Slide 8

This photo shows a buckling of a column due to lack of sufficient confinement steel.

Slide 9

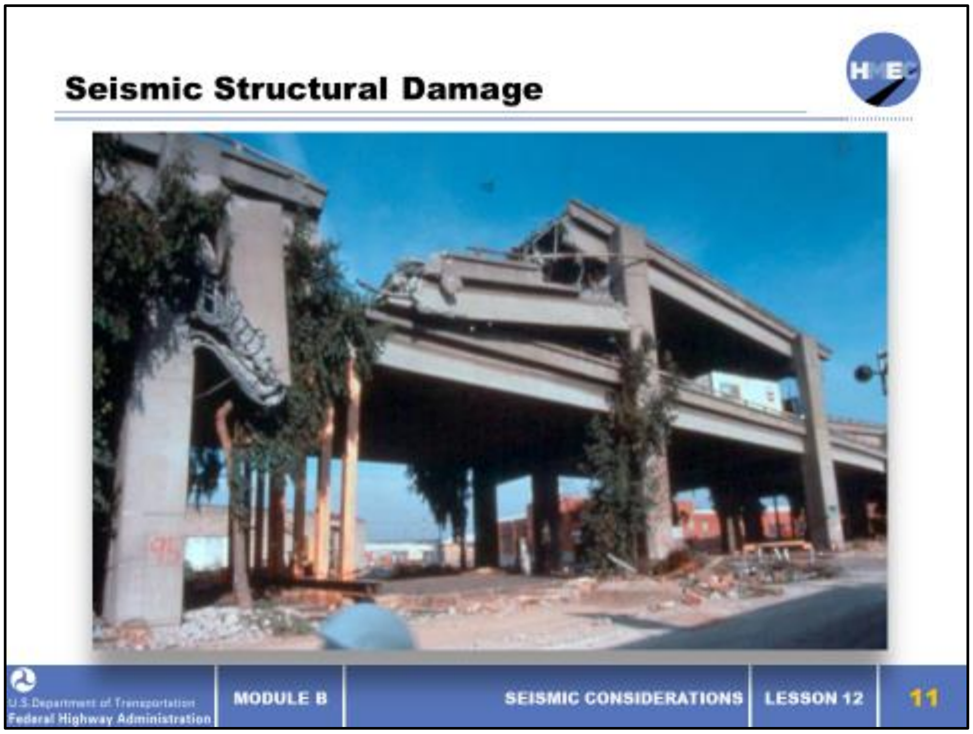


This photo shows a bridge span that fell from girder seat.

Slide 10

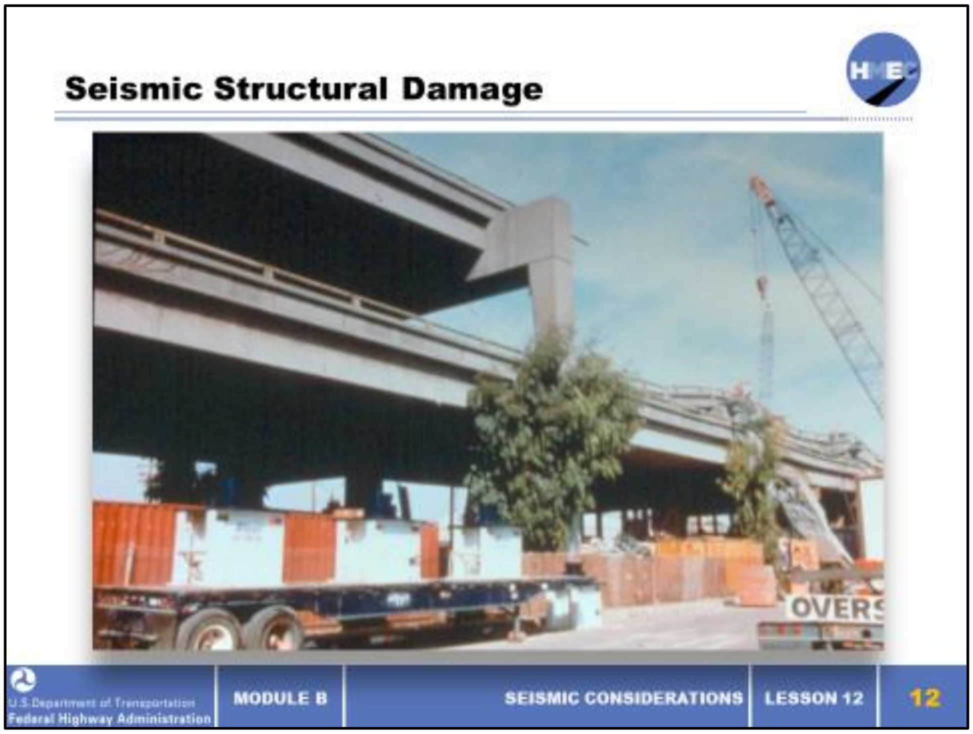
This shows the collapse of the Cypress Viaduct due to the Loma Prieta Earthquake.

Slide 11



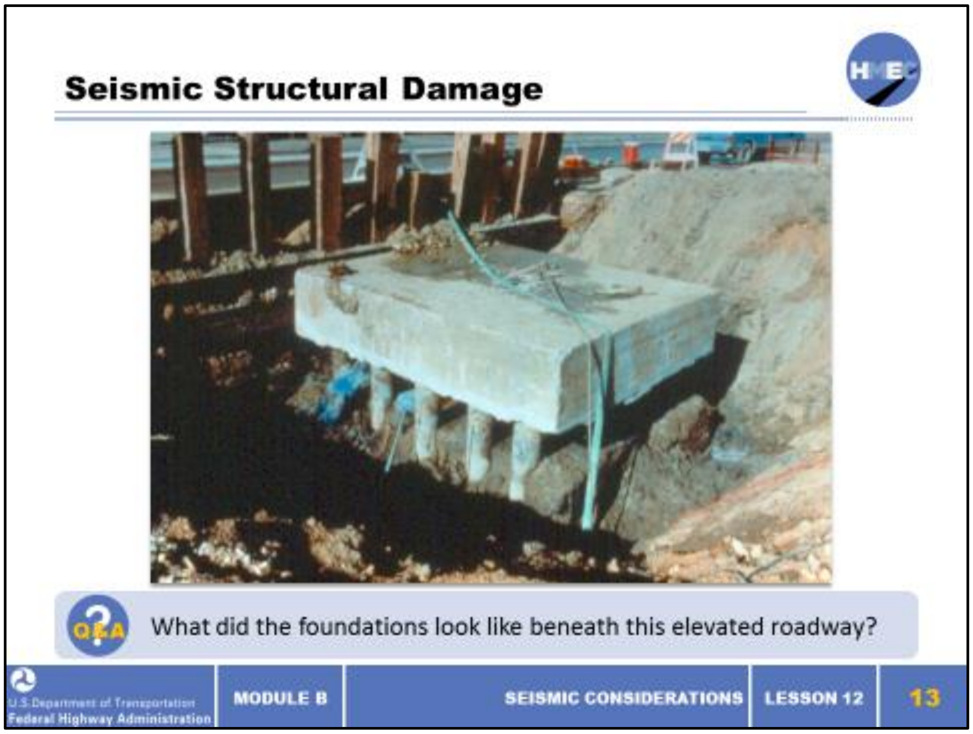
This shows a column collapse due to lack of confining steel.

Slide 12



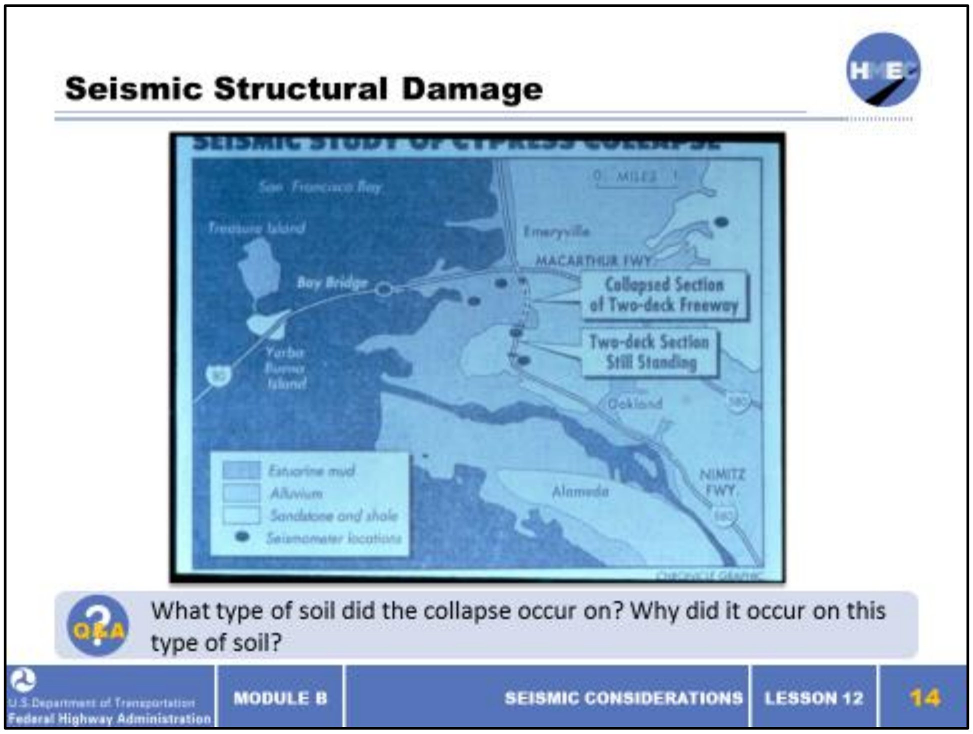
Demolition and removal of fallen sections of the Cypress structure.

Slide 13



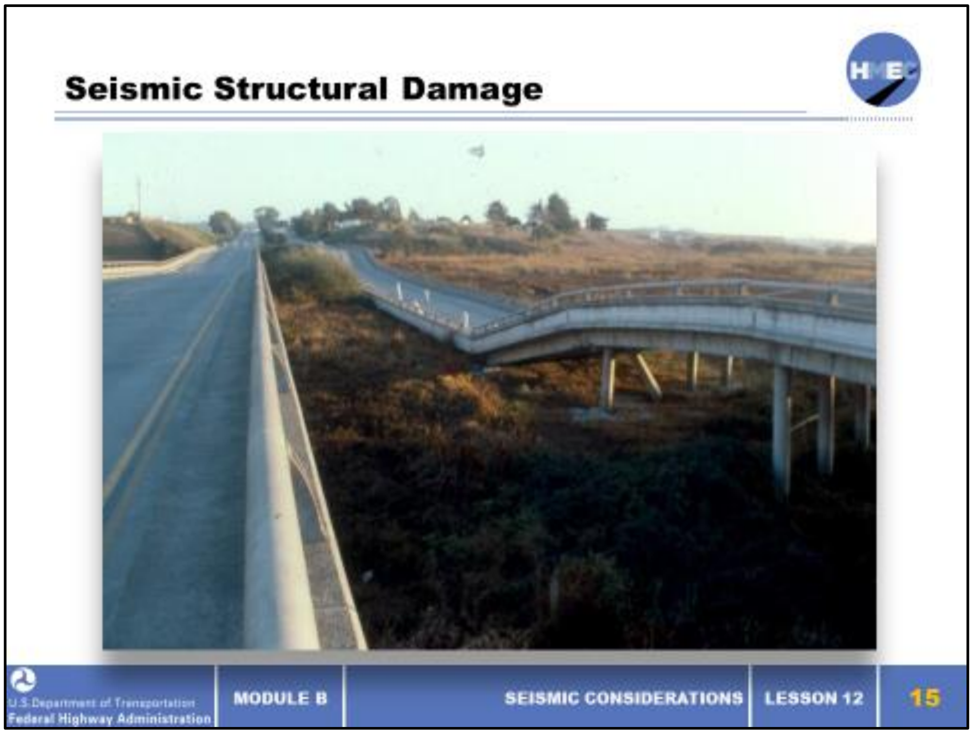
Pile supported footing exhumed during the demolition of the Cypress structure. No damage to the pile observed.

Slide 14



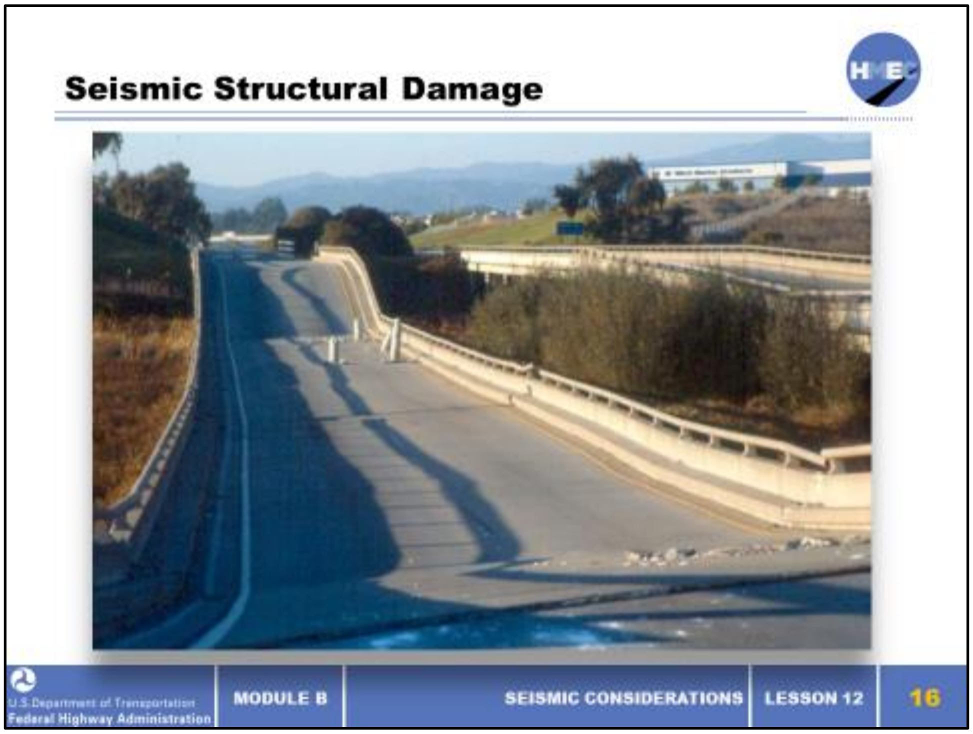
Yes. The correlation of the collapsed structure location and subsurface soil types is shown on this map.

Slide 15



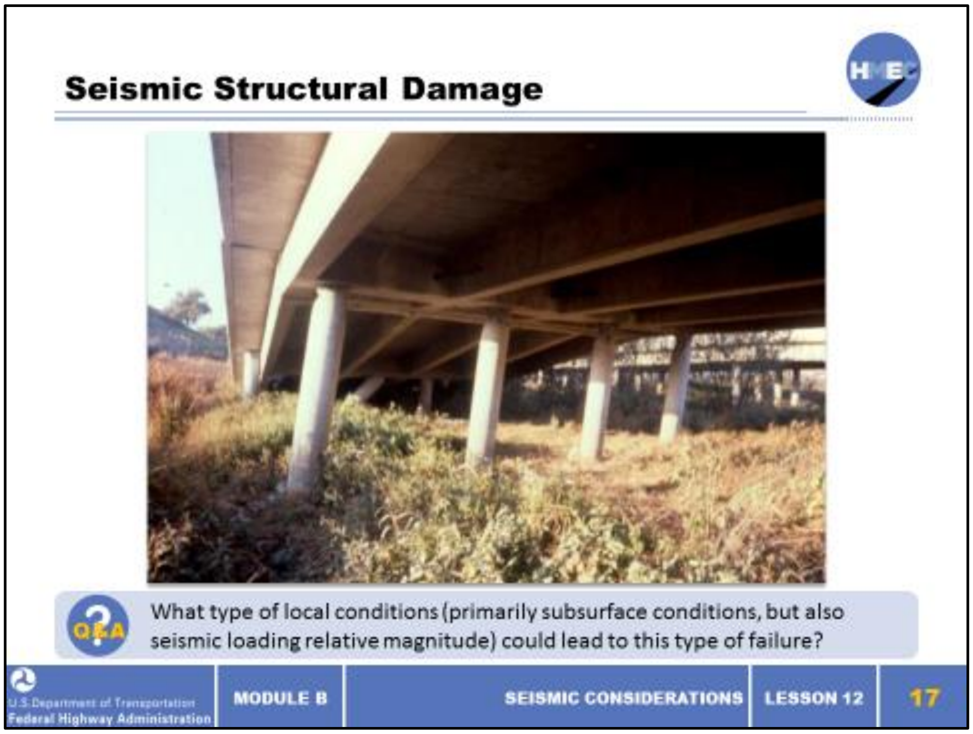
Collapsed structure due to failure of the pile-to-superstructure connections.

Slide 16



Piles punching through the deck.

Slide 17



This photo shows piles leaning.

____

____

____

____

____

____

____

____

____

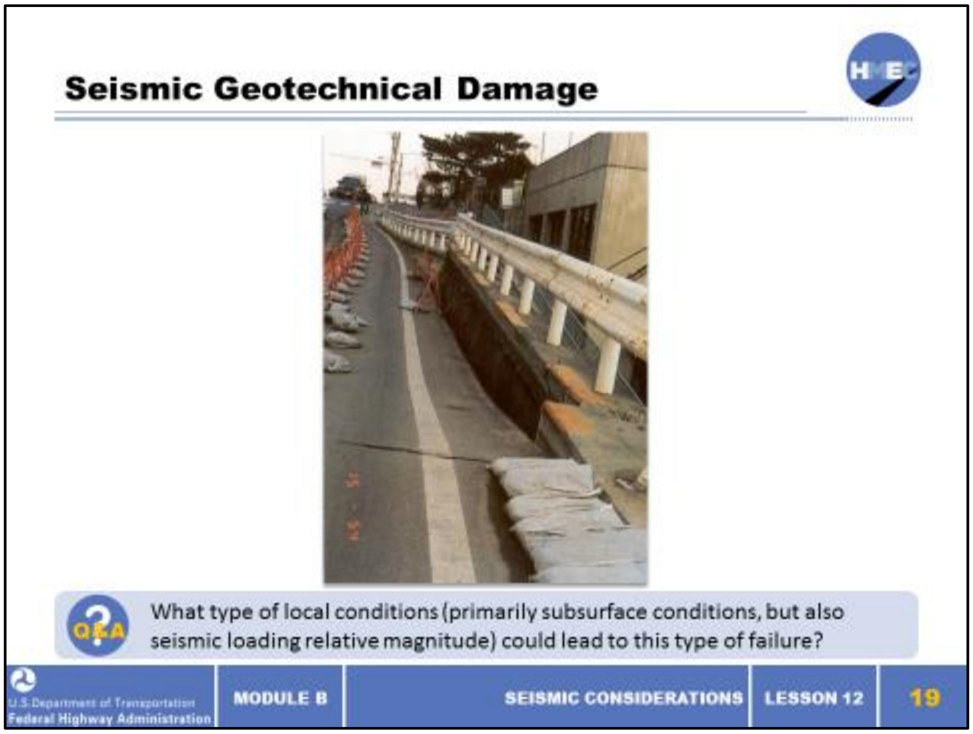
____

____

Slide 18

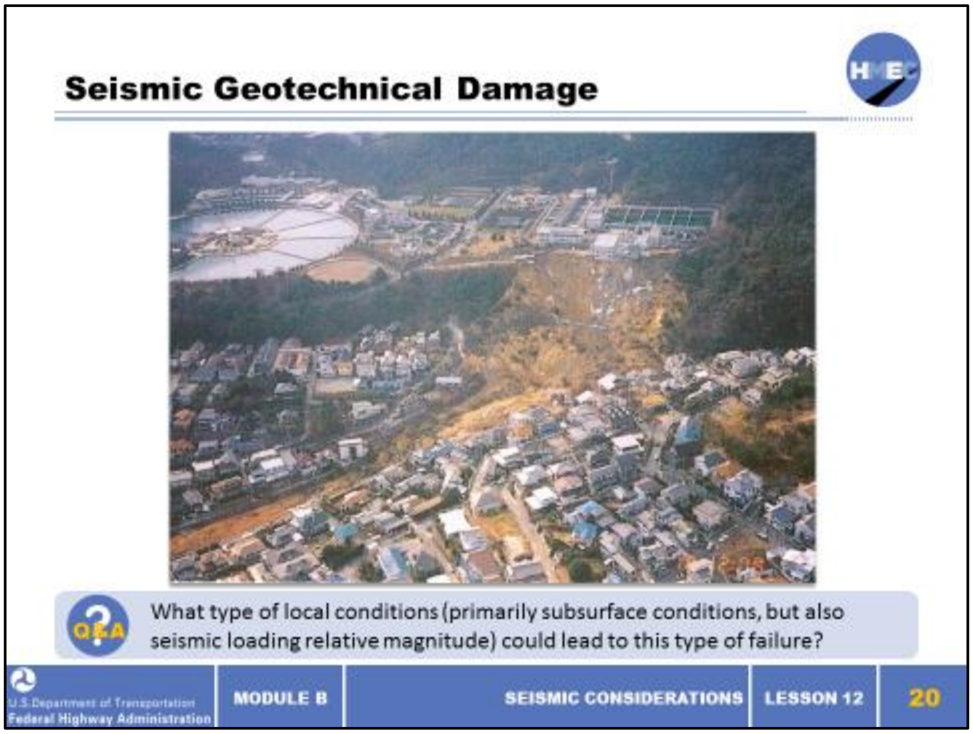
Severe gapping of soil around piles due to structure swaying.

Slide 19



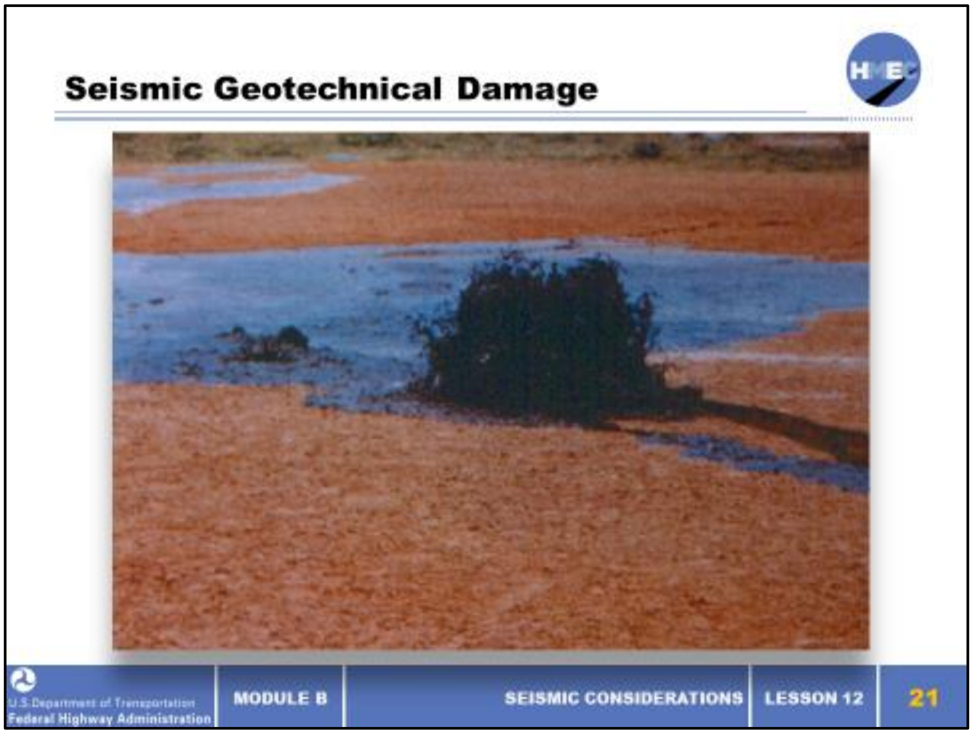
Subsidence of roadway.

Slide 20



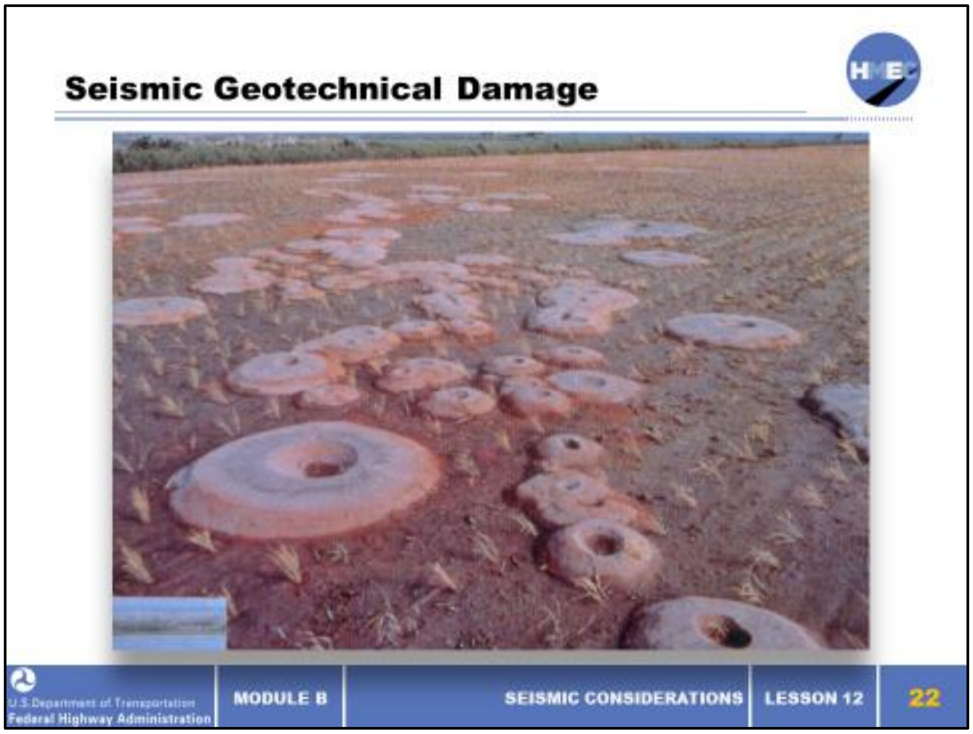
Seismic hazards include a landslide, which is direct damage. The consequences of destroying homes and businesses is an indirect damage.

Slide 21



This image shows the flow of water from the ground due to pore pressures induced by liquefaction. Obviously the underlying soil shear strength is greatly reduced by this excess pore water pressure.

Slide 22



Sand boil features after liquefaction.

___

___

___

___

___

___

___

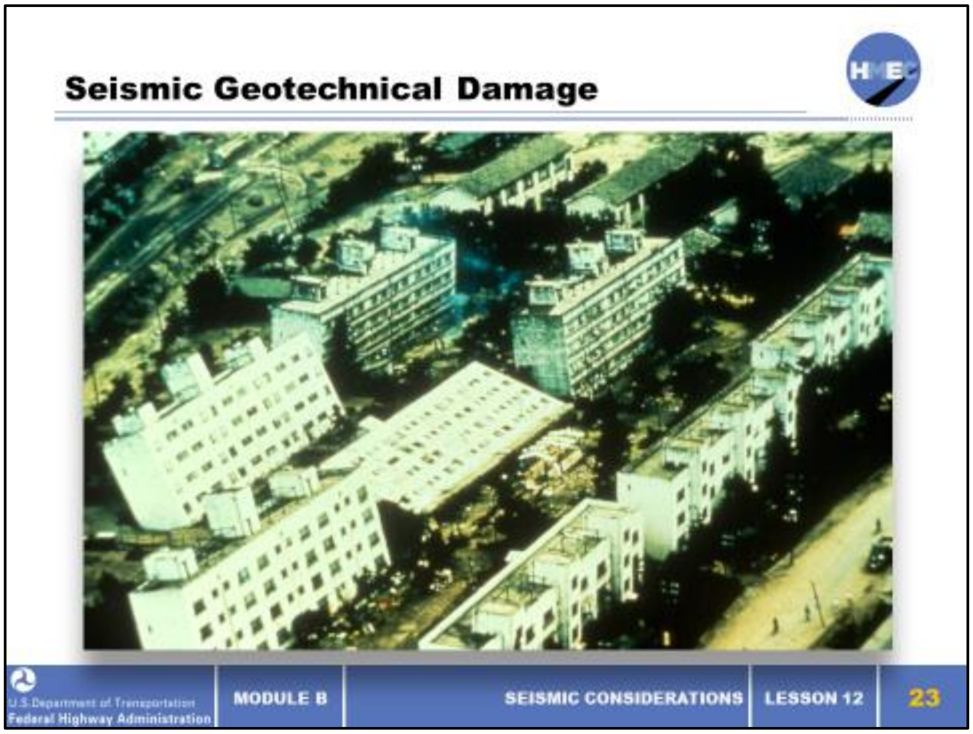
___

___

___

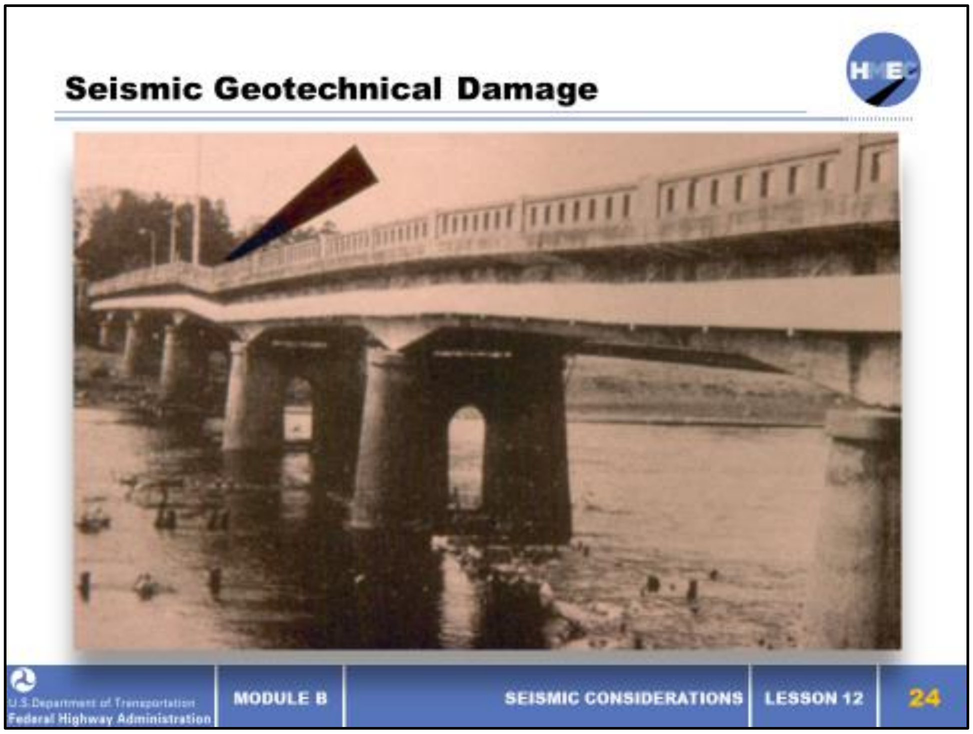
___

Slide 23



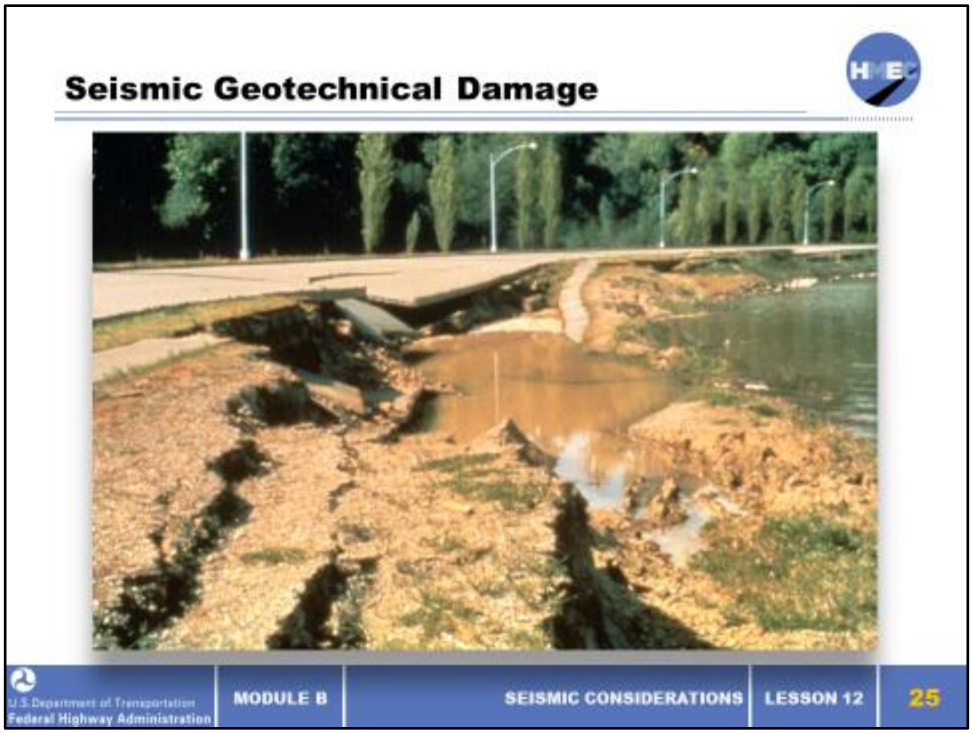
This image shows bearing capacity failures of building foundations due to liquefaction and loss of shear strength of foundation soils.

Slide 24



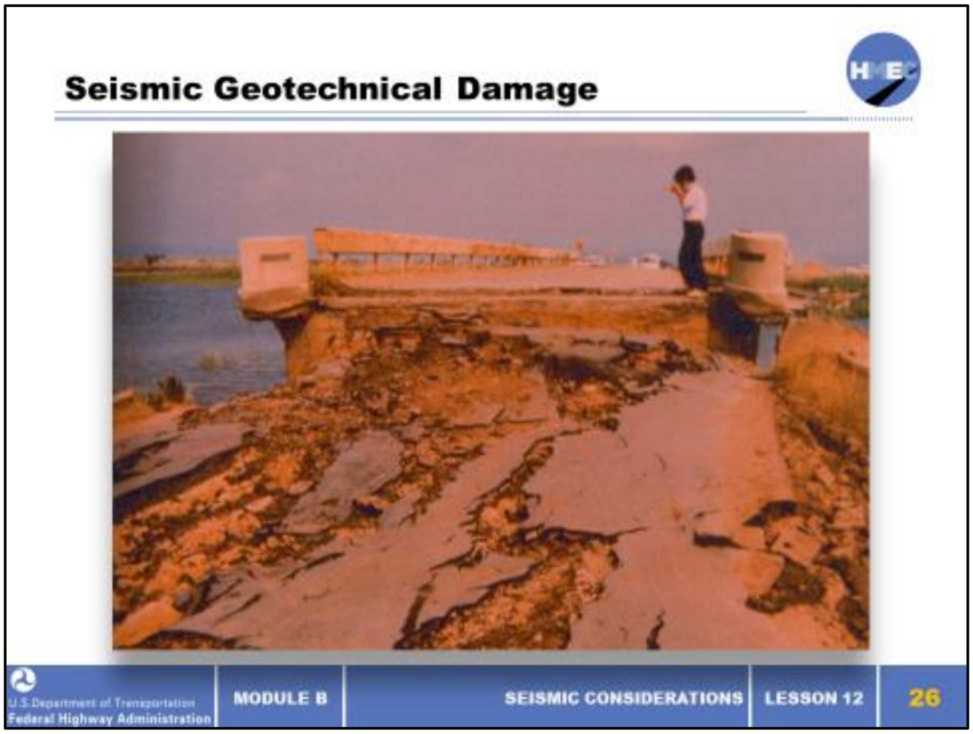
This image shows differential pier movement due to seismic load. This is likely due to lateral spreading of underlying soils, due to liquefaction.

Slide 25



Lateral spreading at Capitol Lake in Olympia, Washington, in 1949.

Slide 26



This image shows the failure of a bridge abutment approach fill.

___

___

___

___

___

___

___

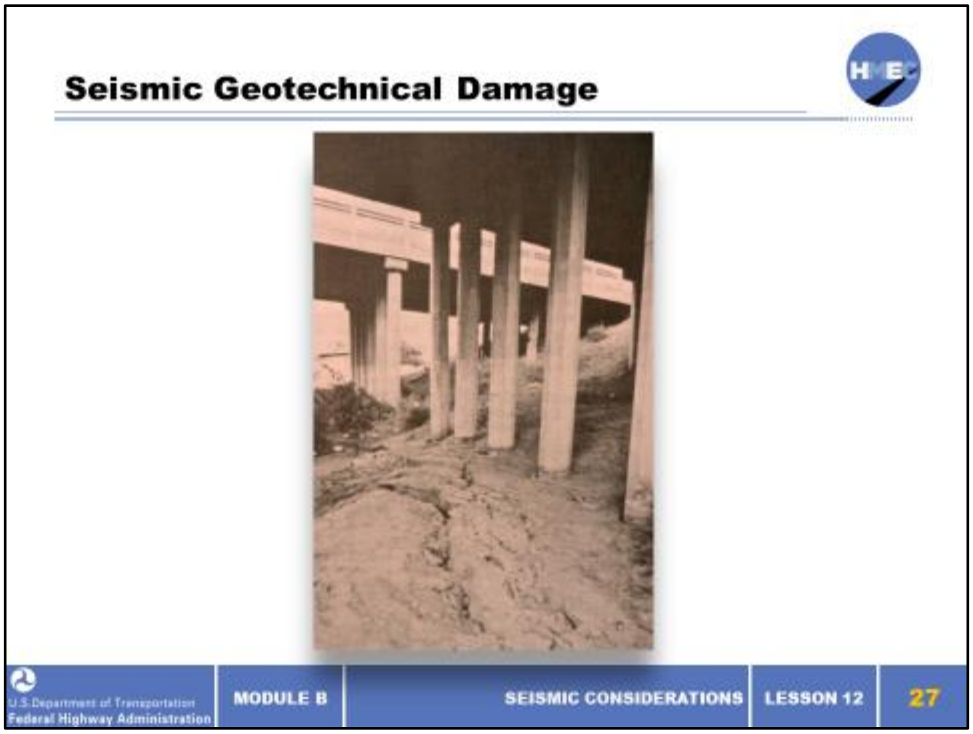
___

___

___

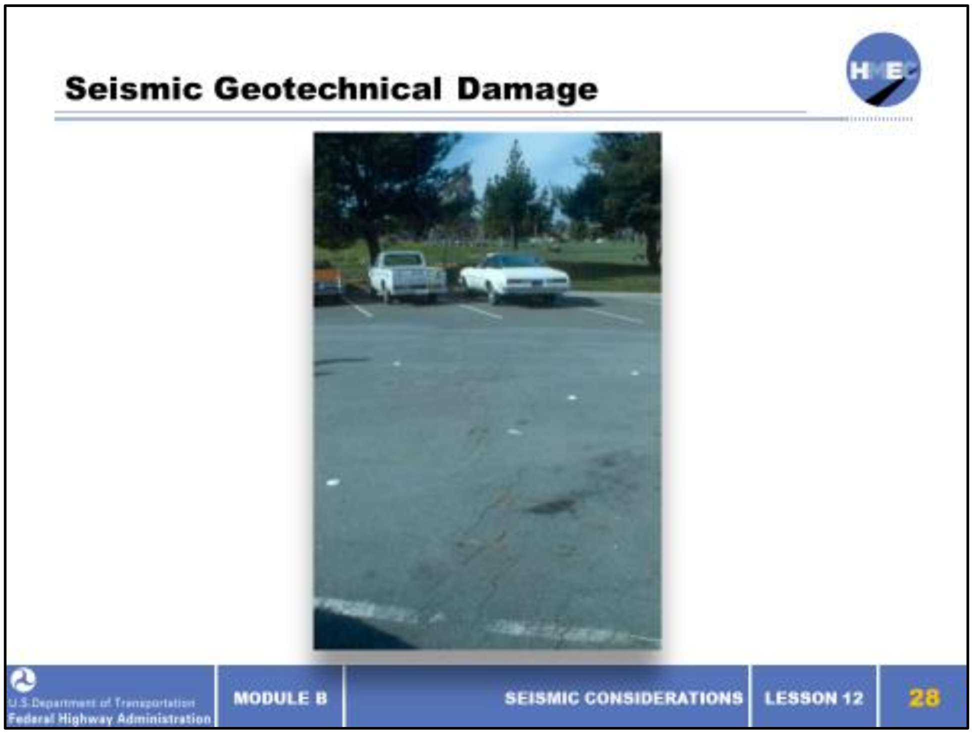
___

Slide 27



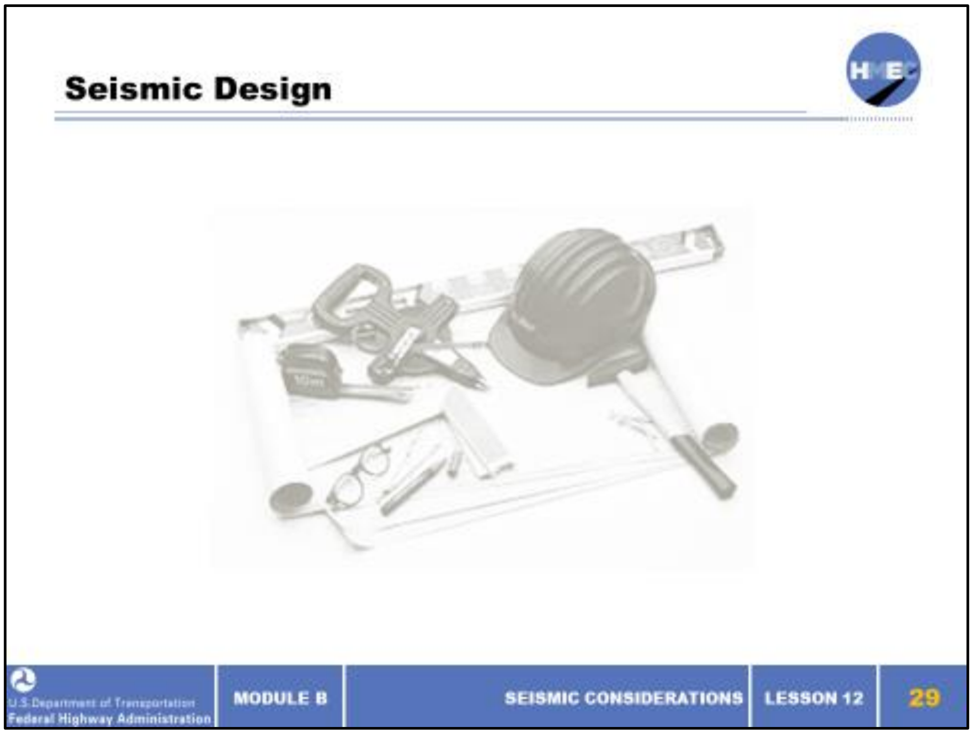
This image shows gaps around bridge columns due to shaking of the structure, which can potentially lead to a loss of support capacity for this bent.

Slide 28



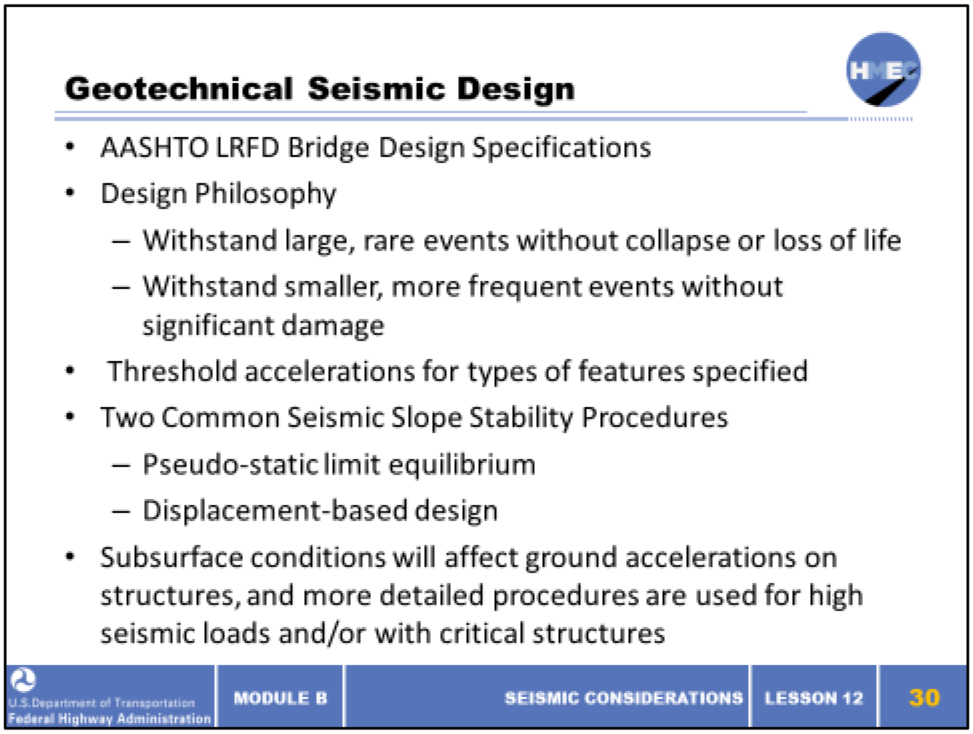
Here is an example of fault creep, or movement along a fault line.

Slide 29



Design codes often change due to lessons learned from the types damage due to seismic events we just reviewed.

Slide 30



Design code is the AASHTO LRFD Bridge Specifications. The design philosophy is that some movements are okay and repairable.

AASHTO code specifies when seismic designs should be performed, based upon seismic rating and type of structure. That is, below certain levels of acceleration, seismic design is not required.

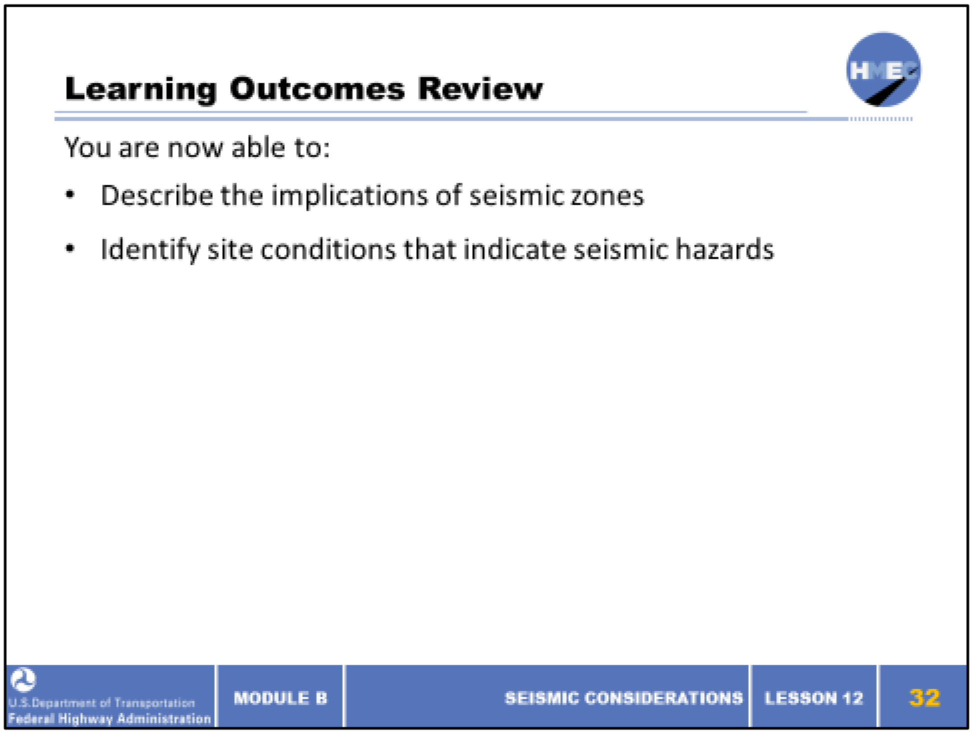
Slope stability can be analyzed with limit equilibrium methods, or with larger seismic loads, a displacement based design may be used. This allows some movement but overall stability is maintained.

There are general maps of seismic loads. These loads can be modified based upon subsurface conditions. For example, seismic loads on a structure will vary if founded on rock or if founded in a deep deposit of soils

Slide 31

Primary resources on seismic design of geotechnical features are AASHTO Bridge Specifications and the FHWA-NHI-11-032 reference manual. There is also a 5-day training course associated with the FHWA manual.

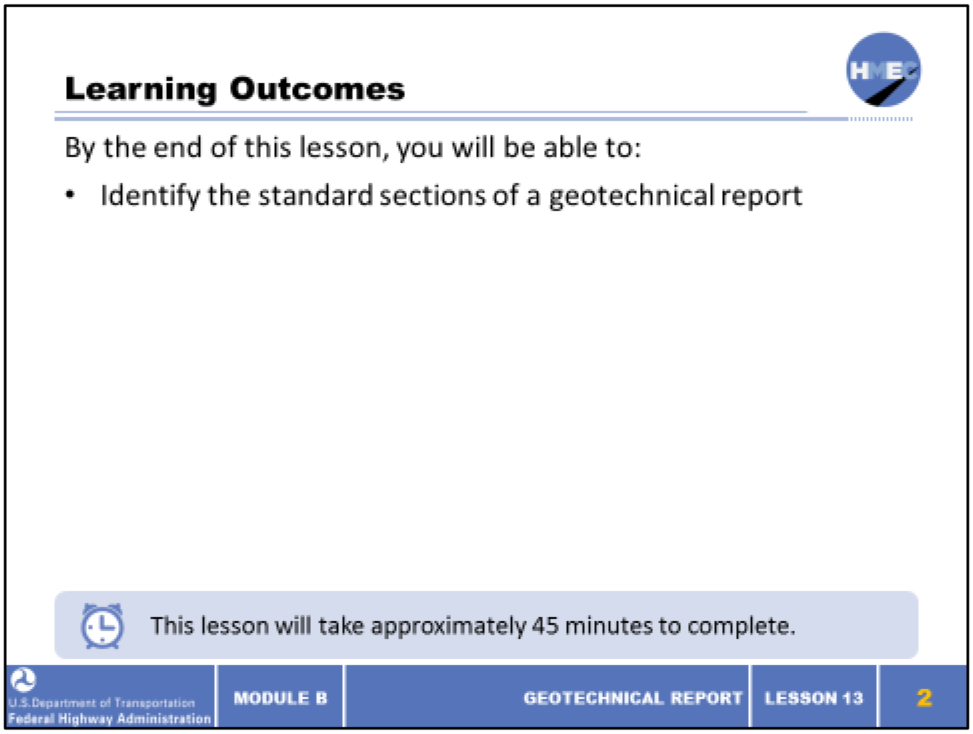
Slide 32


Learning Outcomes Review
HMEC
You are now able to:
• Describe the implications of seismic zones
• Identify site conditions that indicate seismic hazards
U.S.Department of Transportation
Federal Highway Administration
MODULE B
SEISMIC CONSIDERATIONS
LESSON 12
32

Slide 1

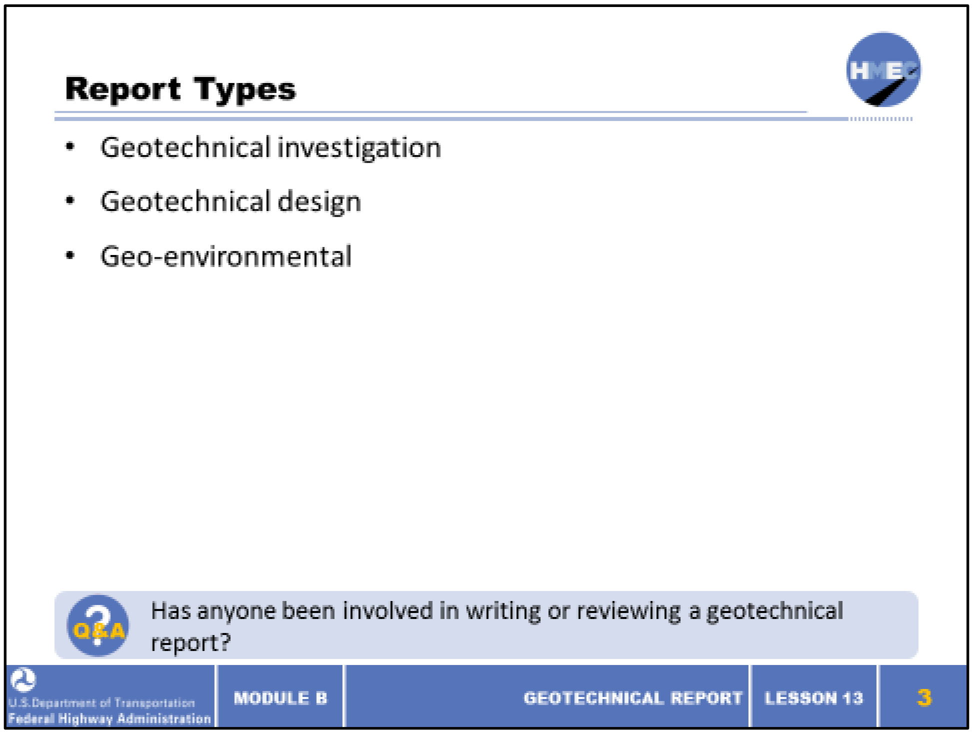

HMEC
Highway Materials Engineering Course
Lesson 13: Geotechnical Report
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

Slide 2


Learning Outcomes
By the end of this lesson, you will be able to:
• Identify the standard sections of a geotechnical report
This lesson will take approximately 45 minutes to complete.
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL REPORT
LESSON 13
2

____
____
____
____
____
____
____
____
____
____
____
____

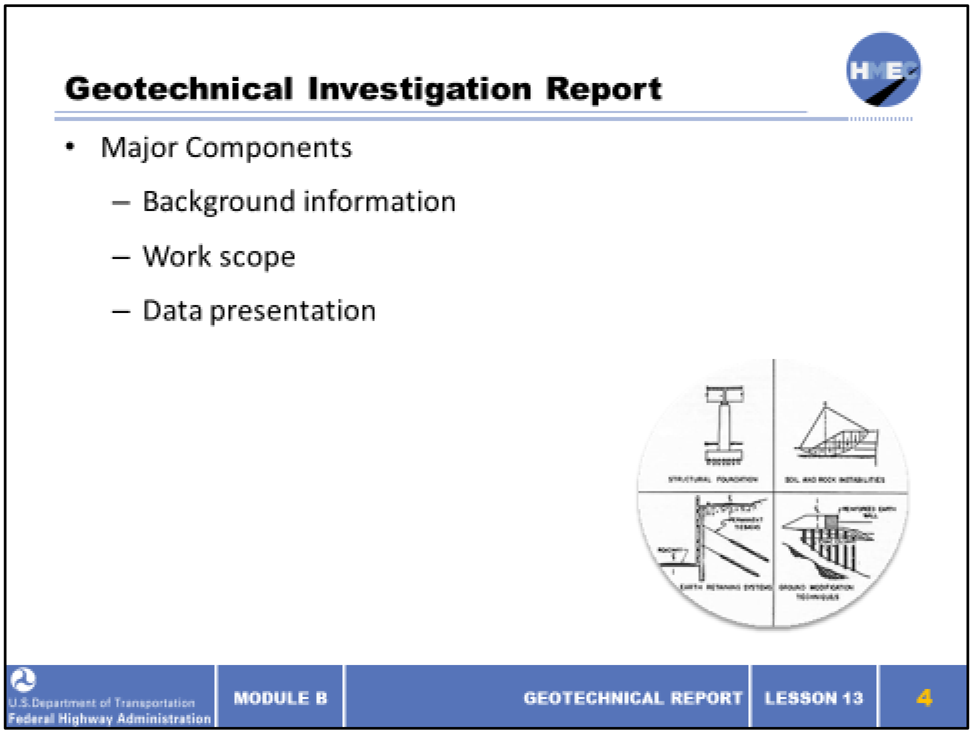
Slide 3



The geotechnical report is the tool used to communicate the site conditions and design and construction recommendations to the roadway design, bridge design, and construction personnel (including the contractor). Site investigations for transportation projects have the objective of providing specific information on subsurface soil, rock, and water conditions. Interpretation of the site investigation information, by a geotechnical engineer, results in design and construction recommendations that should be presented in a project geotechnical report.

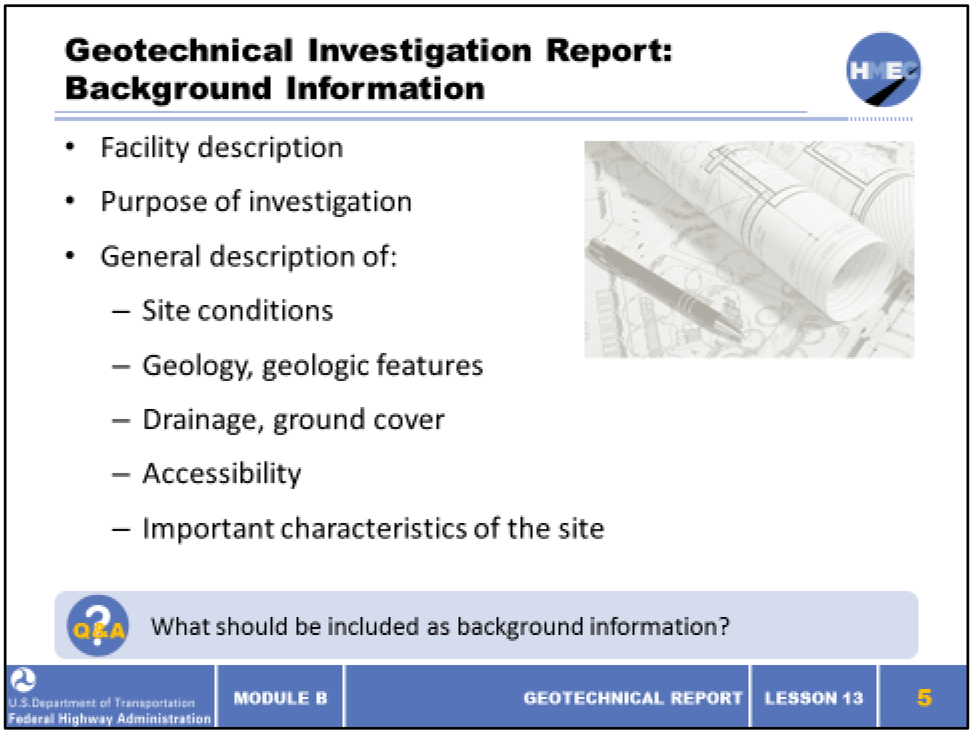
A single report may combine the elements of the three report types listed here or individual reports may be written for each type of report listed. The report may be a few pages for a simple project to multiple volumes for a major complex project.

Slide 4



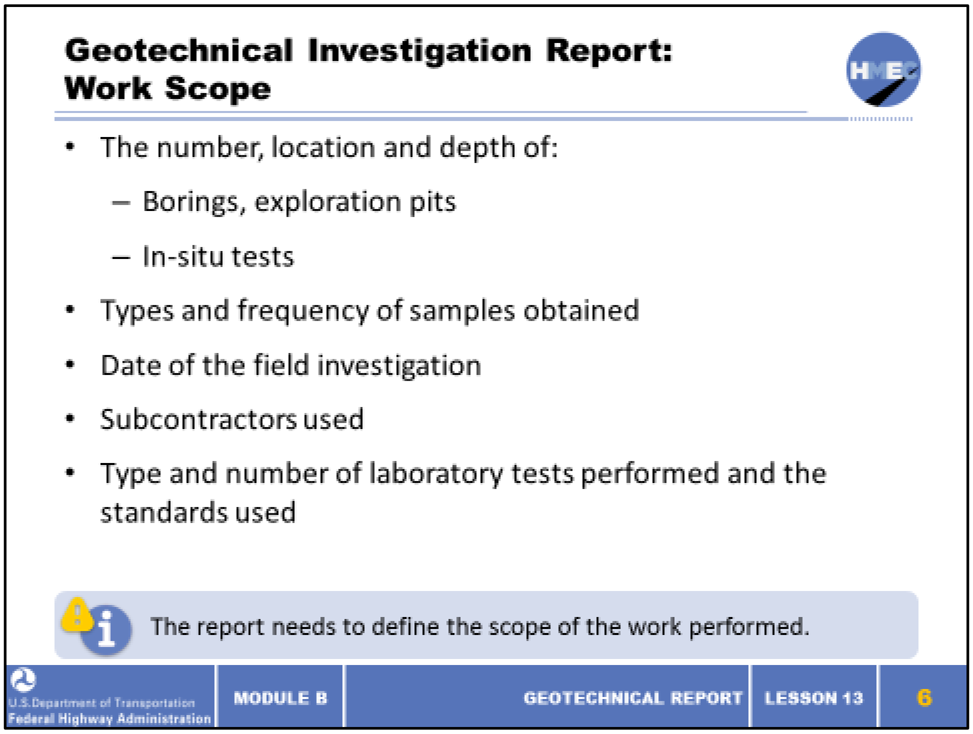
Several of the elements discussed for this report will be common to the other two report types. The reports typically have three major components: background information, work scope, and data presentation.

Slide 5



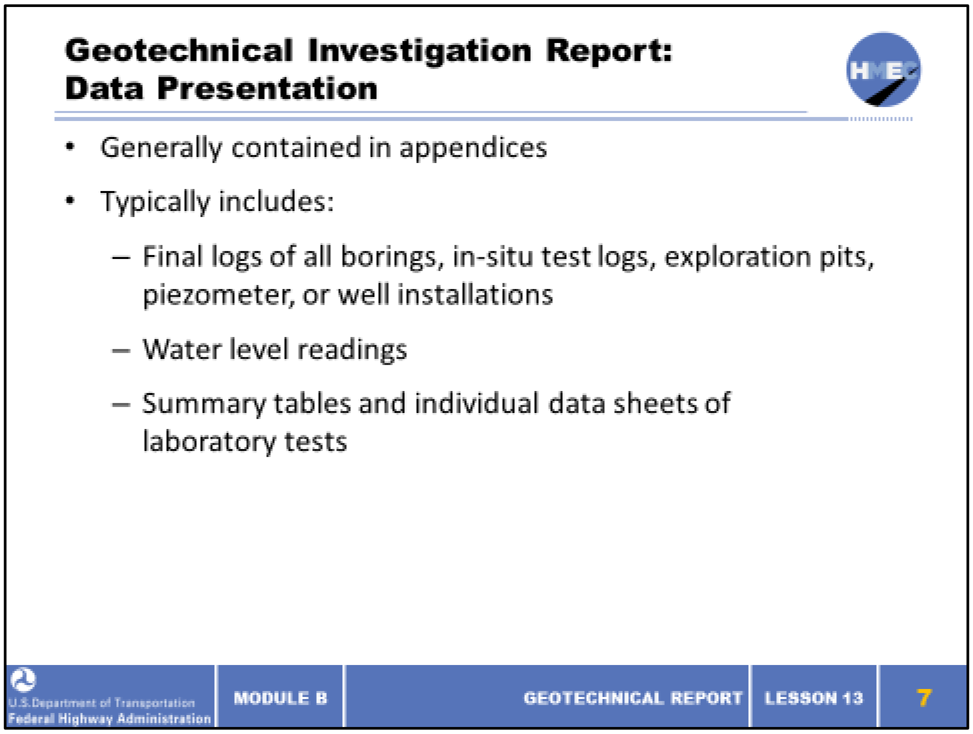
The initial sections of the report summarize the geotechnical specialist's understanding of the facility for which the report is being prepared and the purposes of the subsurface exploration. This section includes information on loads, deformations, and additional performance requirements. This section also presents a general description of site conditions, geology and geologic features, drainage, ground cover and accessibility, and any important characteristics of the site that may affect the design and/or construction.

Slide 6



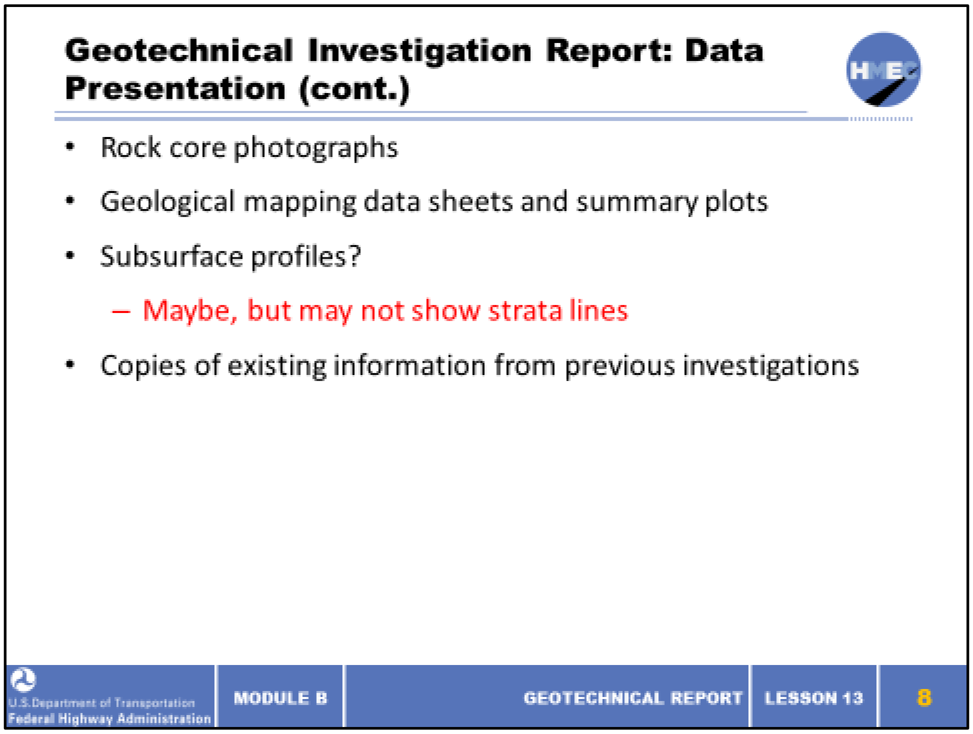
The second part of the investigation report documents the scope of the exploration program and the specific procedures used to perform this work. These sections identify the types of exploration methods used; the number, location, and depths of borings, exploration pits, and in-situ tests; the types and frequency of samples obtained; the dates of subsurface exploration; the subcontractors used to perform the work; the types and number of laboratory tests performed; the testing standards used; and any variations from conventional procedures.

Slide 7



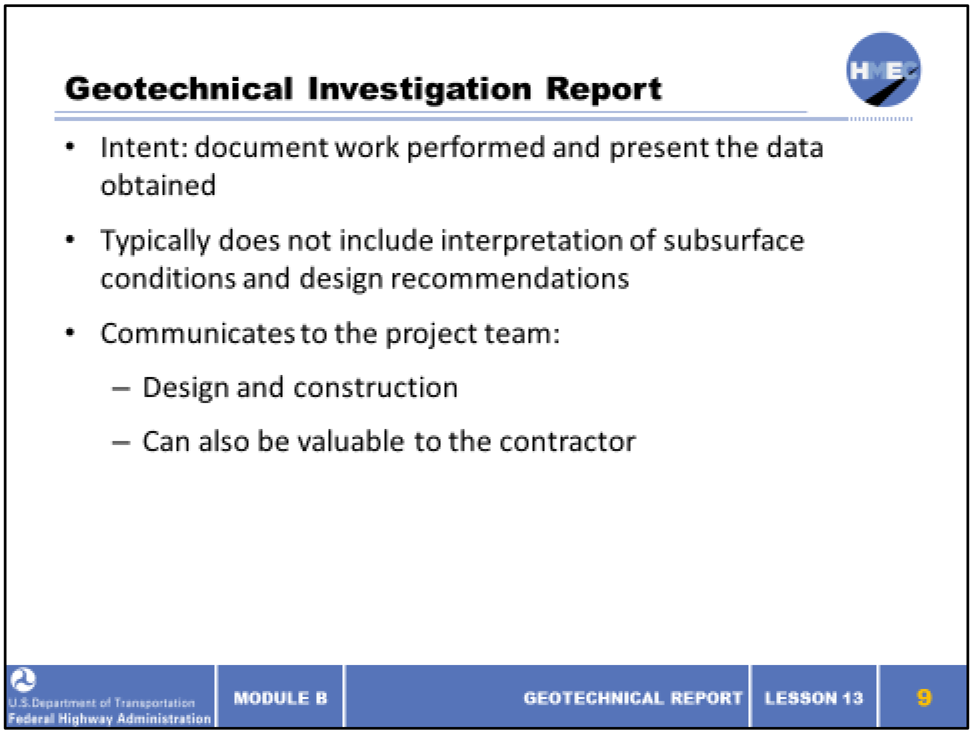
This portion of the report, generally contained in appendices with a complementary narrative of explanation, presents the data obtained from the field and laboratory exploration program. The appendices typically include final logs of all borings, in-situ test logs, exploration pits, and piezometer or well installations, water level readings, data plots from each in-situ bore hole, and summary tables and individual data sheets for all laboratory tests performed.

Slide 8



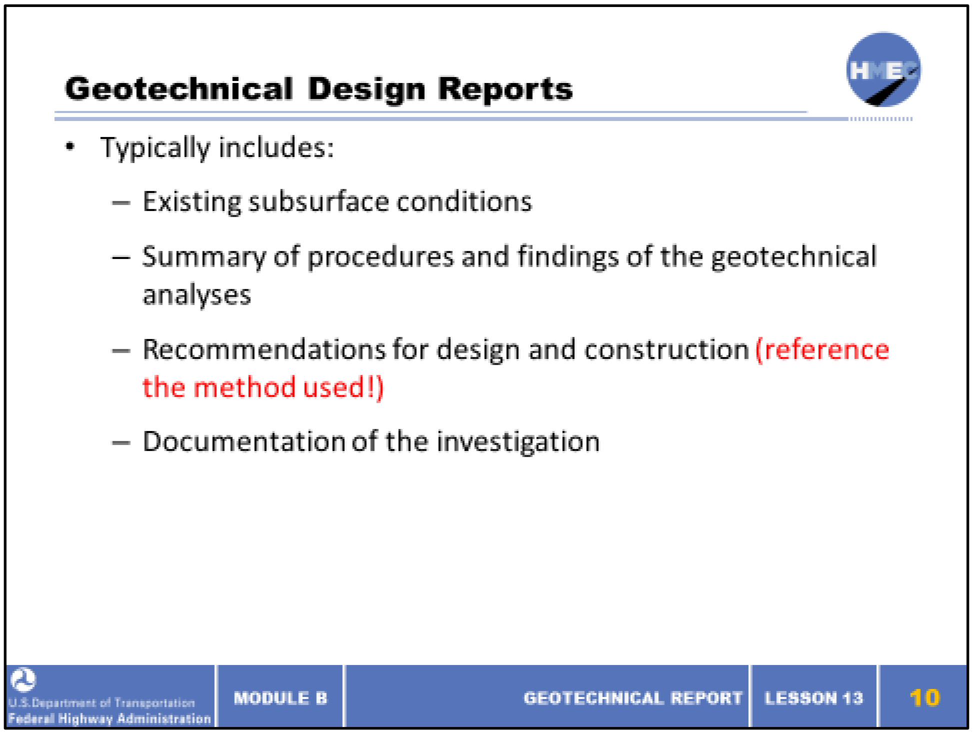
Also included are: rock core photographs, geologic mapping data sheets, and summary plots, subsurface profiles developed from the field and laboratory test data, as well as statistical summaries. The geotechnical investigation report often includes copies of existing information such as boring logs or laboratory test data from previous investigations at the project site.

Slide 9



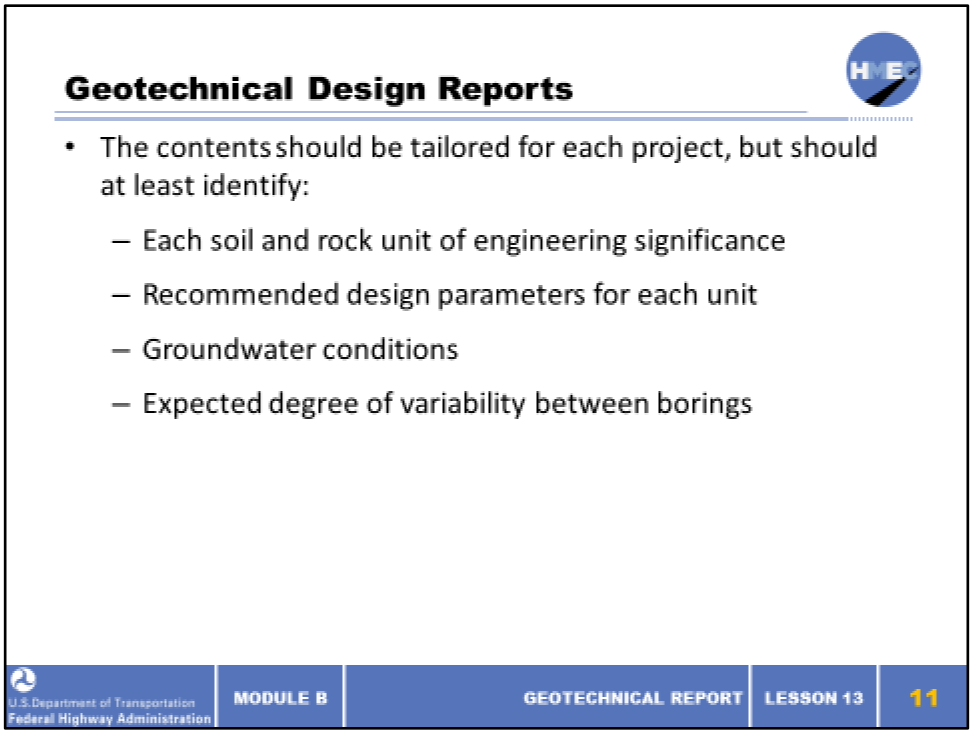
In summary, the geotechnical investigation report is intended to document the work performed and to present the data. It does not typically include data interpretation or design recommendations. The report should be distributed to the project team, both design and construction. As we will discuss later, the report is also valuable to the contractor.

Slide 10



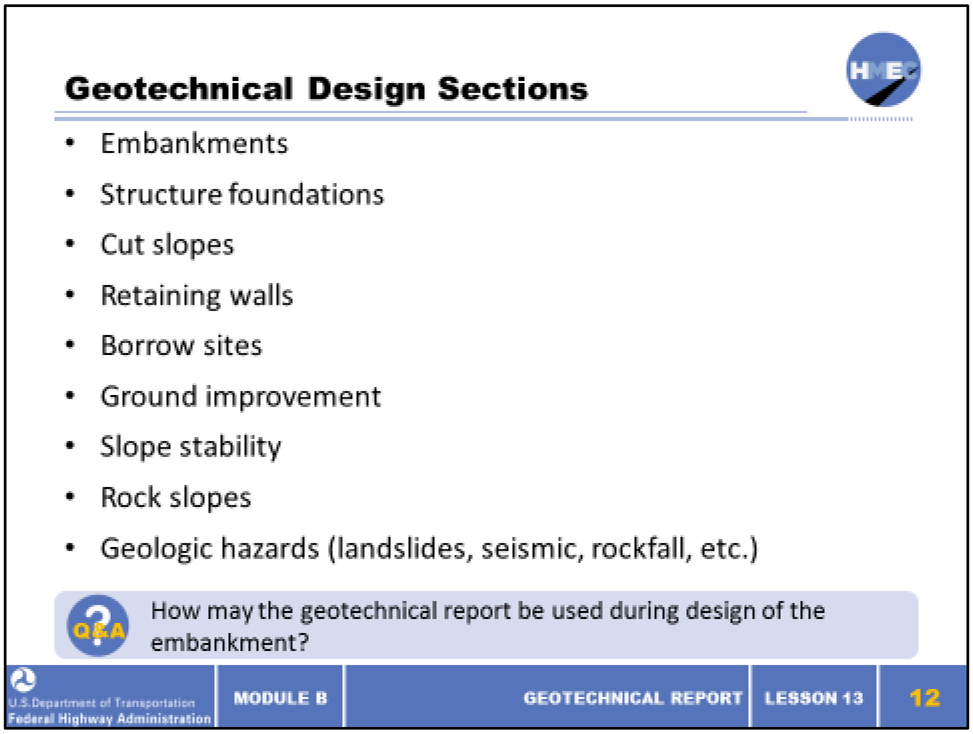
A geotechnical design report typically provides an assessment of existing subsurface conditions at a project site; presents, describes, and summarizes the procedures and findings of all geotechnical analyses performed and provides appropriate recommendations for design and construction of foundations, earth retaining structures, embankments, cuts, and other required facilities. Unless a separate geotechnical investigation report was developed previously, the geotechnical design report will also include documentation of any subsurface explorations and laboratory investigations performed and a presentation of the results of those investigations.

Slide 11



Since the scope, site conditions, and design/construction requirements of each project are unique, the specific contents of a geotechnical design report must be tailored for each project. In order to develop this report, the author must possess detailed knowledge of the facility. The report must identify each soil and rock unit of engineering significance, and must provide recommended design parameters for each of these units. To this end, all factual data must be synthesized and analyzed to justify the recommended index and design properties. Groundwater conditions are particularly important for both design and construction and, accordingly, they need to be carefully assessed and described. For every project, the subsurface conditions encountered in the site investigation need to be compared with the geologic setting in order to understand the nature of the deposits better and to predict the degree of variability between exploration locations.

Slide 12



Embankment design recommendations such as the slope required for stability, any other measures that need to be taken to provide a stable embankment (e.g., geosynthetic reinforcement, wick drains, controlled rate of embankment construction, lightweight materials, etc.), embankment settlement magnitude and rate, and the need and extent of removal of any unsuitable materials beneath the proposed fills.

Geotechnical recommendations for bridges, tunnels, hydraulic structures, and other structures.

Cut design recommendations such as the slope required for stability, seepage and piping control, erosion control measures needed, and any special measures required to provide a stable slope.

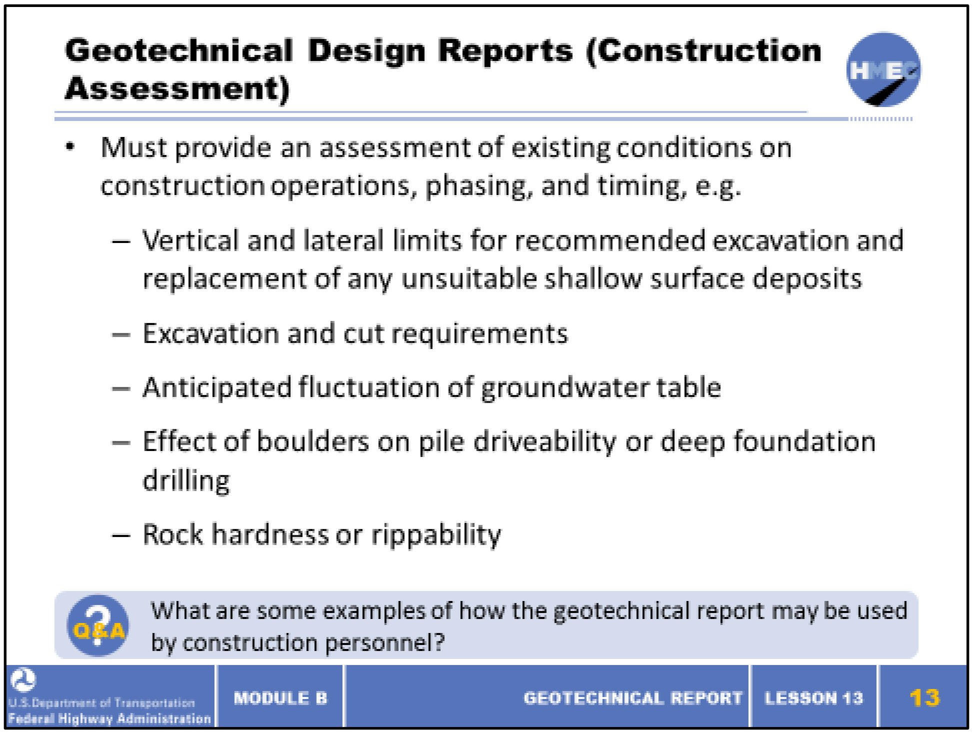
Geotechnical recommendations for retaining walls and reinforced slopes.

Regarding usability of on-site materials, soil units should be identified as to their feasibility of use as fill material, the type of fill material for which the on-site soils are feasible, the need for aeration, and the effect of weather conditions on its usability, and identification of materials that should definitely be considered as waste. Borrow sites also should be identified and issues regarding suitability, excavation, and dewatering should be discussed.

Geotechnical recommendations for stabilization of unstable slopes (e.g., landslides, rockfall areas, debris flows, etc.) and/or ground improvement.

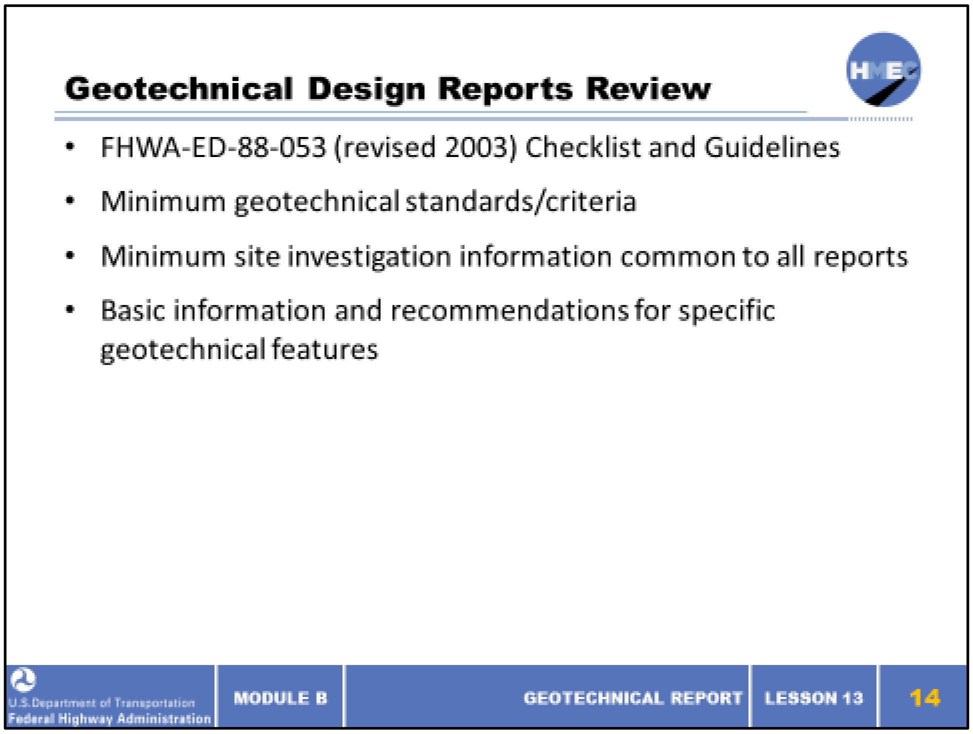
Geotechnical recommendations for rock slopes and rock excavation. Such recommendations should include, but are not limited to, stable rock slope, rock bolting/dowelling, and other stabilization requirements, including recommendations to prevent erosion/undermining of intact blocks of rock, internal and external slope drainage requirements, feasible methods of rock removal, etc.

Slide 13



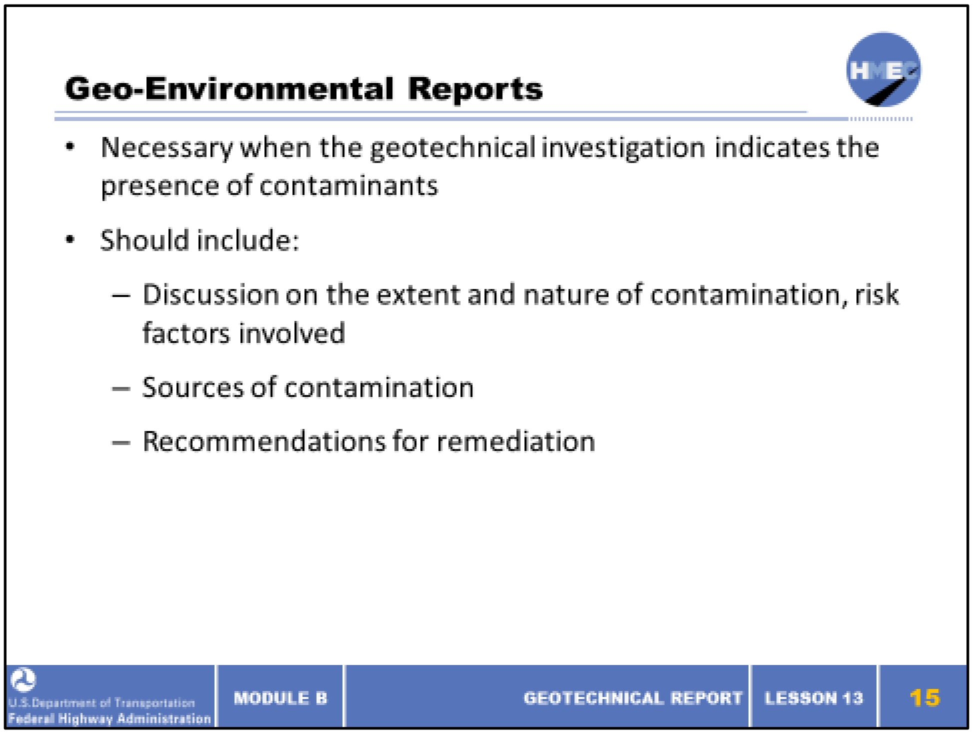
Of particular importance is an assessment of the impact of existing subsurface conditions on construction operations, phasing, and timing. Properly addressing any construction issues in the report that are related to subsurface conditions can preclude change-of-conditions claims.

Slide 14



To aid engineers with a quantitative review of geotechnical reports, FHWA has prepared review checklists and technical guidelines (FHWA, 2003b). One of the primary purposes of the FHWA guidelines is to provide transportation agencies and consultants with minimum standards/criteria for the geotechnical information that FHWA recommends be included in geotechnical reports as well as plans and specification packages. Technical guidelines for minimum site investigation information common to all geotechnical reports for any type of geotechnical feature and basic information and recommendations for specific geotechnical features are provided in the checklists and technical guidelines.

Slide 15



The preparation of such a report usually requires the geotechnical specialist to work with a team of experts since many aspects of the contamination or the remediation may be beyond his/her expertise. A representative team preparing a geo-environmental report may be composed of chemists, geologists, hydrogeologists, environmental scientists, toxicologists, air quality, and regulatory experts, as well as one or more geotechnical specialists.

The geo-environmental report will have a clear and concise discussion of the nature and extent of contamination, the risk factors involved (if applicable), a contaminant transport model, and the source of the contamination.

The team may also be required to present solutions to remediate the site.

____

____

____

____

____

Slide 16

The data collected from the site investigation and laboratory testing is typically presented in the form of boring logs, cone penetration logs, and subsurface profiles.

Boring logs, rock coring, soundings, and exploration logging should be prepared in accordance with the standard procedures and formats. Test boring logs and exploration test pit records can be prepared by using software capable of storing, manipulating, and presenting geotechnical data in simple one-dimensional profiles, or alternatively two-dimensional graphs of the subsurface profiles, or three-dimensional representations.

Most agencies are now maintaining a historical boring log database. Borings can be easily pulled up for future work on the same site or on a nearby site.

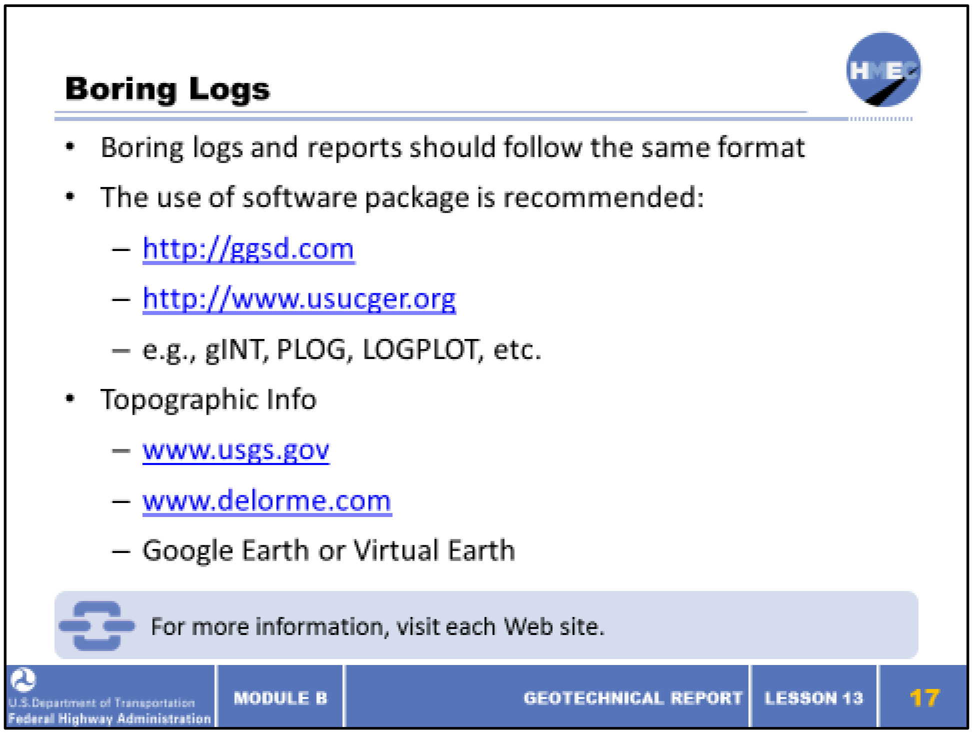
____

____

____

____

Slide 17



To ensure consistency in format, it is recommended that a software package be used. The Web sites listed provided source information for various commercial software packages and topographic information.

The use of software package is recommended:
http://ggsd.com
http://www.usucger.org

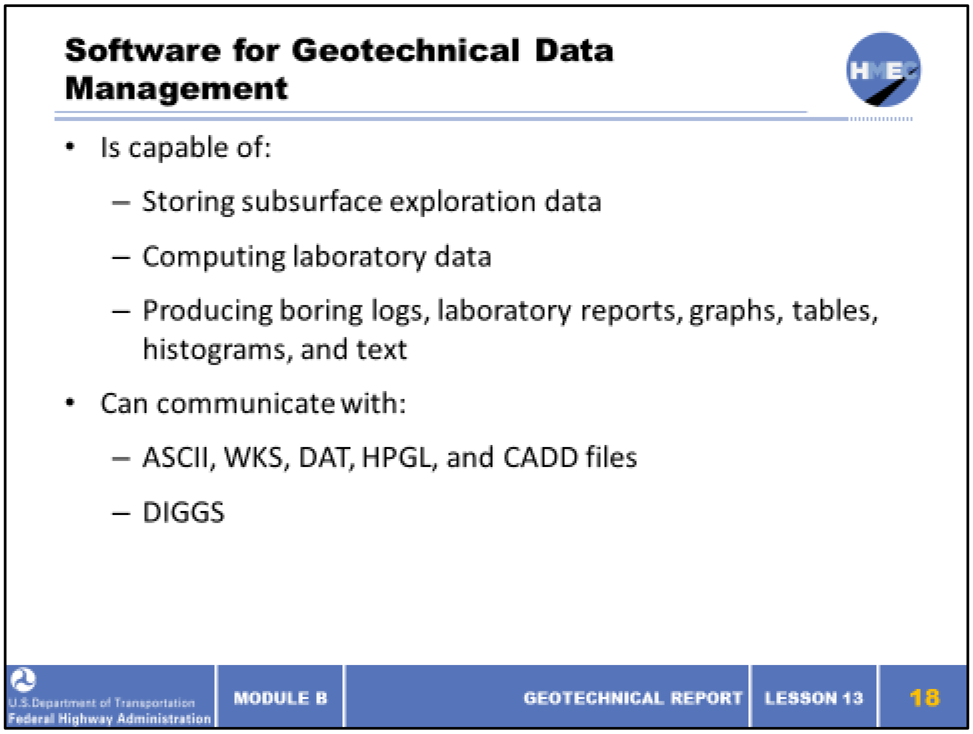
Topographic Info
http://www.usgs.gov
http://www.delorme.com
gINT, PLOG, and LOGPLOT are examples of software packages available to generate boring logs and profiles. Most agencies have standardized their logs and have preferred software.

____

____

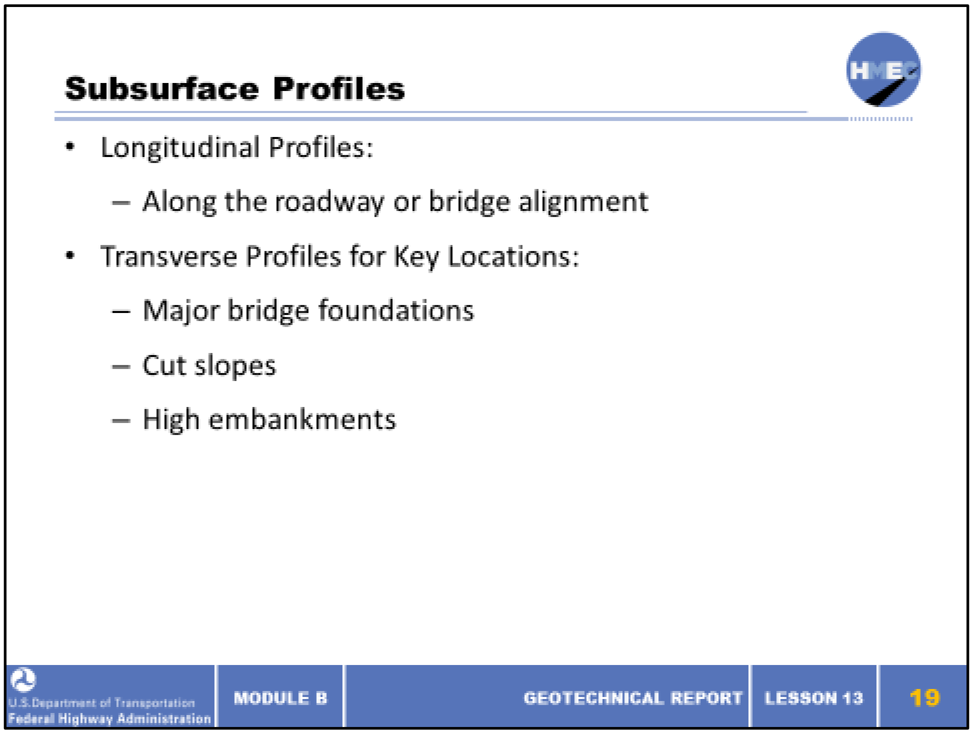
____

Slide 18



Once data is stored in an electronic format within the software package, it can be used during design of the current project. It will also be available for future projects at this project site or nearby projects.

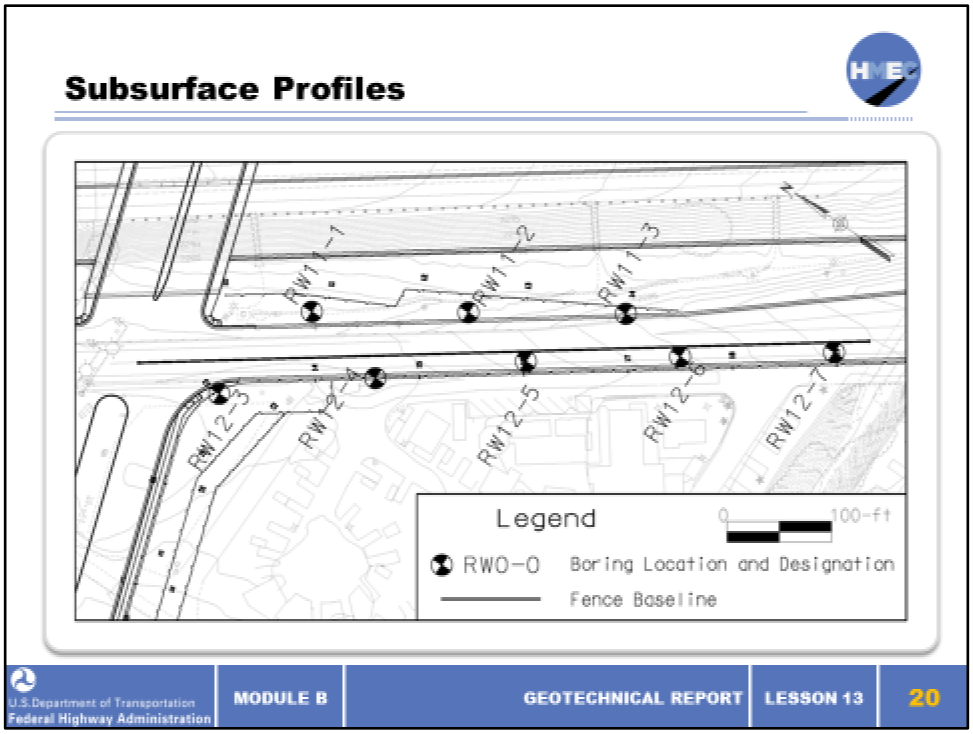
Slide 19



Geotechnical reports are normally accompanied by the presentation of subsurface profiles developed from the field and laboratory test data. Longitudinal profiles are typically developed along the roadway or bridge alignment, and a limited number of transverse profiles may be included for key locations such as at major bridge foundations, cut slopes, high embankments, or adjacent to wetlands, streams, rivers, and lakes. Such profiles provide an effective means of summarizing pertinent subsurface information. The subsurface profiles, coupled with judgment and an understanding of the geologic setting, aid the geotechnical specialist in his/her interpretation of subsurface conditions between the investigation sites.

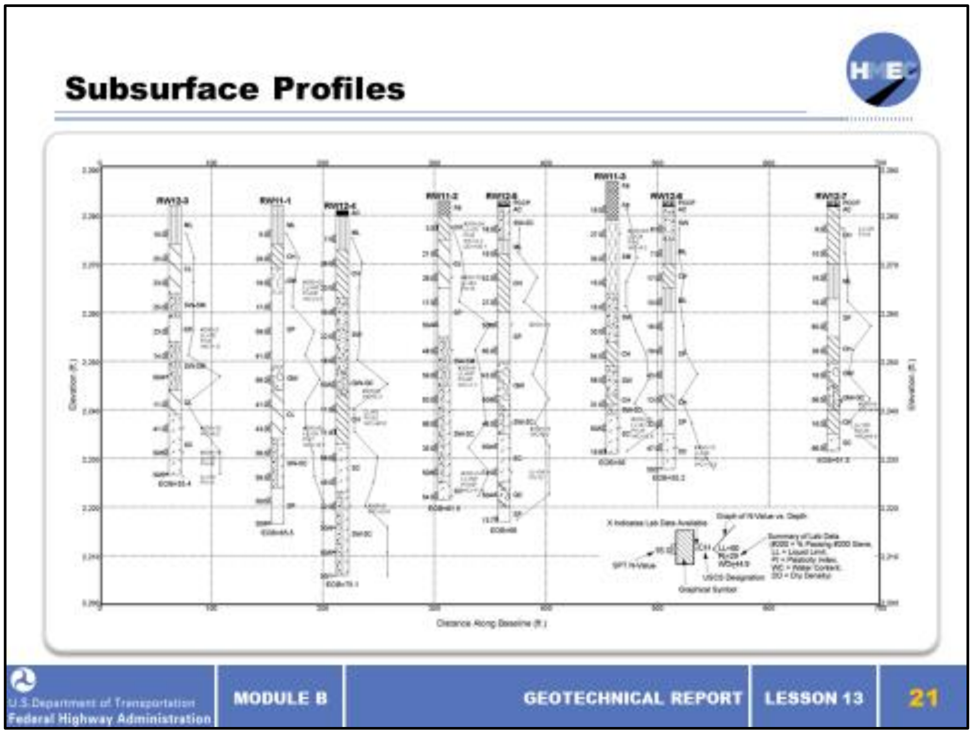
These profiles also provide a summarized presentation of data for use by construction personnel and contractors.

Slide 20



Preferably, the plan should be a topographic map with well-delineated elevation contours and a properly established benchmark. The direction of magnetic or true north should be shown. The baseline (see legend) defines the line along which a vertical profile of subsurface conditions will be developed based on information from adjacent boring logs. If multiple types of exploratory methods are used, the legend on the site test location plan should clearly show the different types of soundings.

Slide 21



The subsurface profile can be presented with reasonable accuracy and confidence at the locations of the borings. However, owners and designers generally expect the geotechnical specialist to present a continuous subsurface profile that shows an interpretation of the location, extent, and nature of subsurface formations or deposits between borings. At a site where rock or soil profiles vary significantly between boring locations, the value of such presentations become questionable. Should there be a need to provide more reliable continuous subsurface profiles, the geotechnical specialist should increase the frequency of borings and/or utilize geophysical methods to determine the continuity of subsurface conditions, or lack thereof.

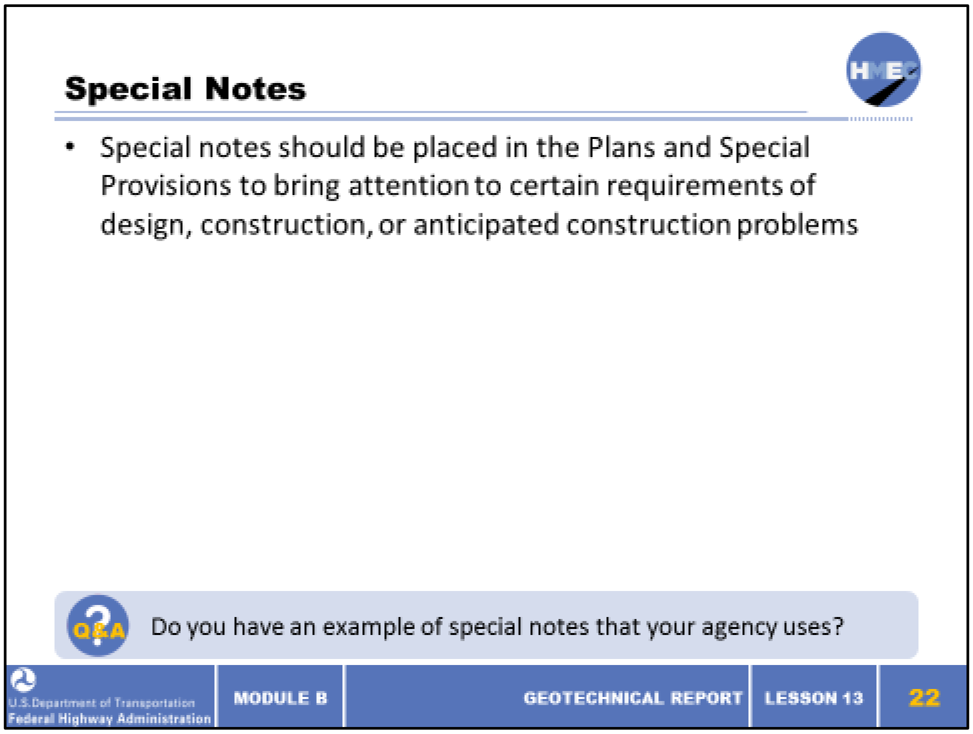
The profile may or may not include interpretive strata lines between boring locations. Since the strata lines are interpretive, their use by contractors should be limited by specific disclaimers, which we will discuss later.

This profile is along the baseline of retaining walls. (borings have RW designation)

____

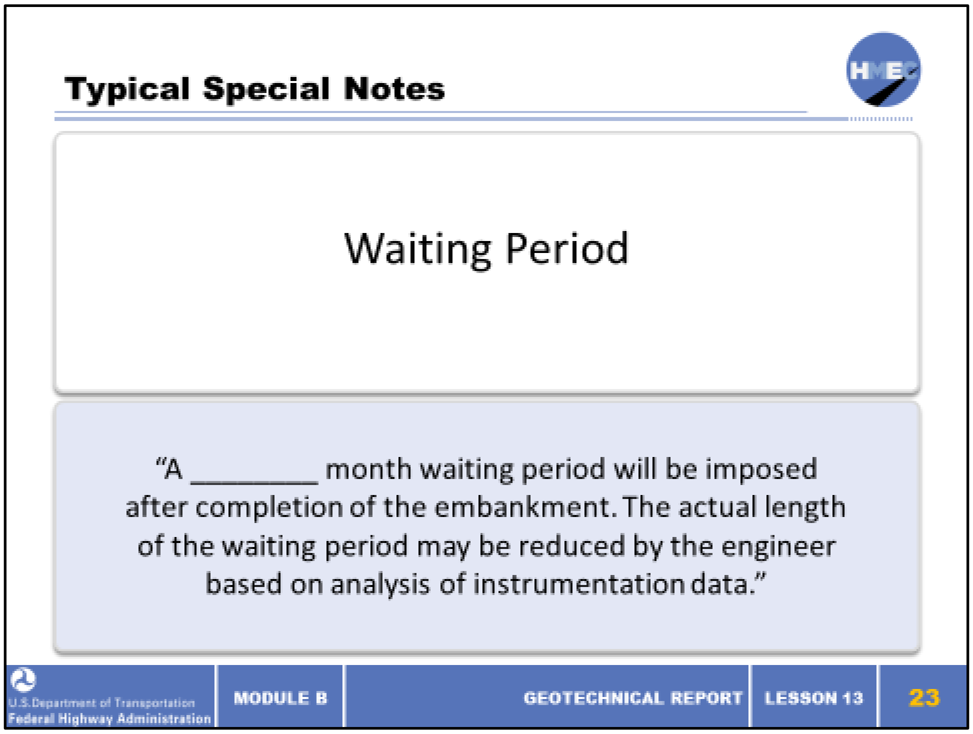
____

Slide 22



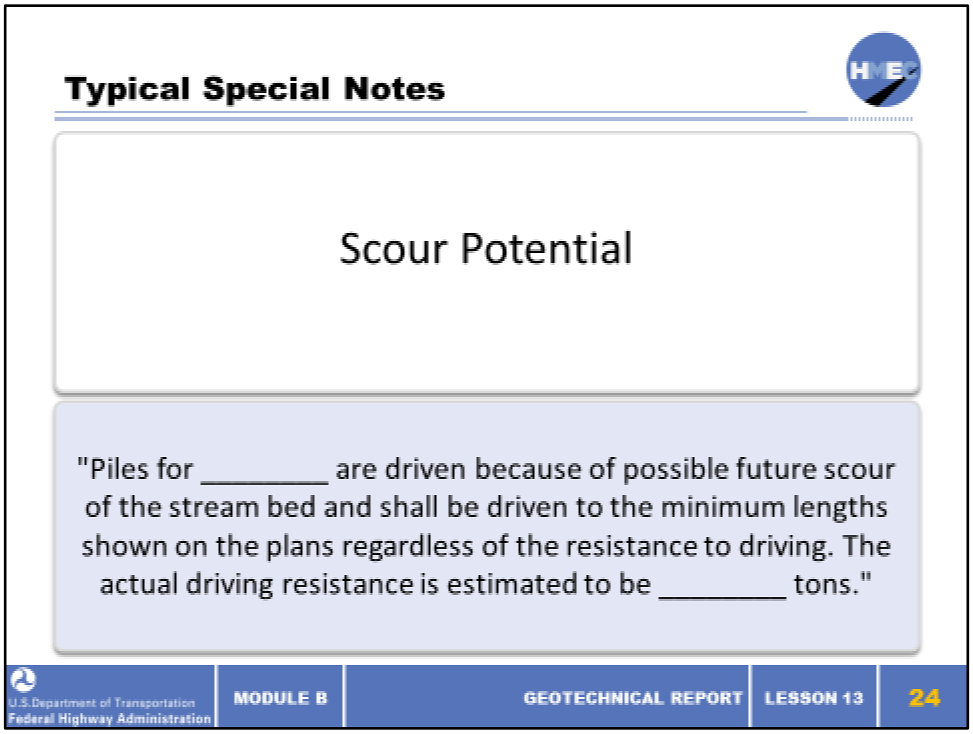
Examples of special notes may include issues such as time delay required for embankment settlement, dewatering issues, pile driving issues, problem soils encountered during the site investigation that may limit construction methods used, etc.

Slide 23



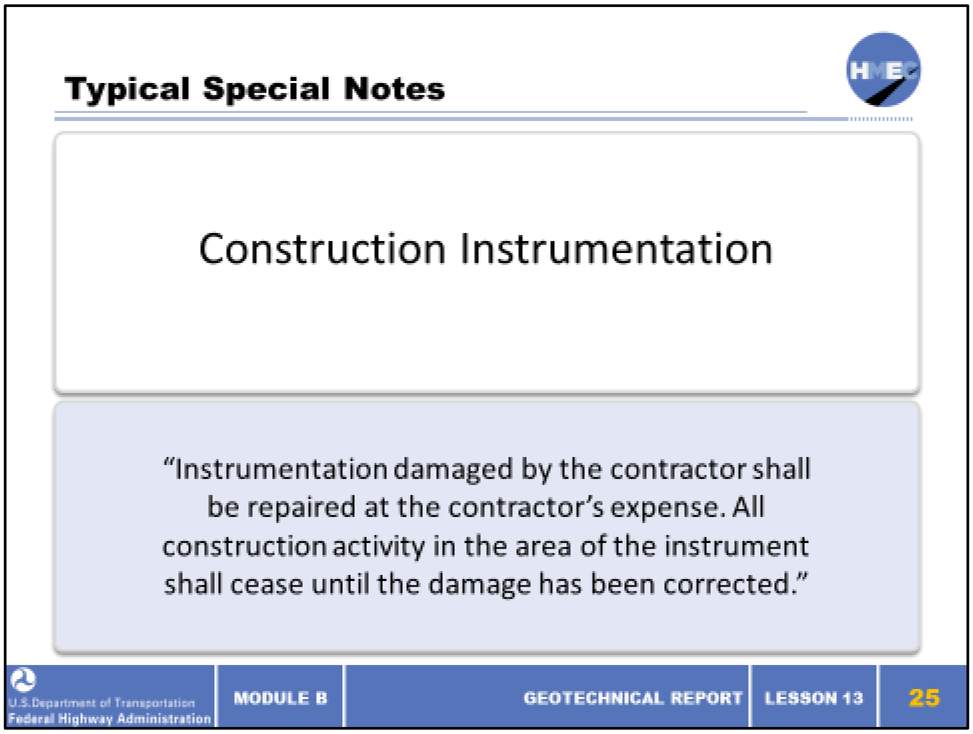
A waiting period may be required to allow the embankment to settle before driving the abutment piles. This technique is often used to reduce the magnitude of downdrag on piles. Instrumentation may be used to monitor settlement and more accurately determine the delay period, which may be more or less than indicated in the note.

Slide 24



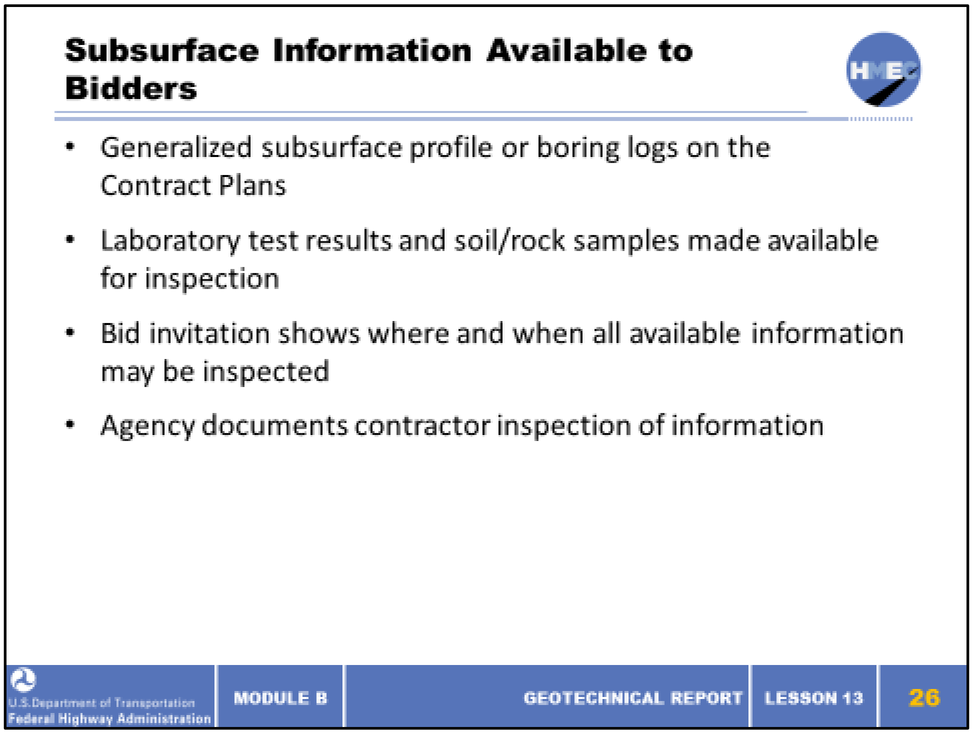
The geotechnical report should address the potential for scour and the recommended solution. Driving piling to a tip elevation below the predicted scour elevation is a viable solution. Many times that requires driving the pile to a resistance above what is required to resist the design loads. This example of a special note addresses that issue.

Slide 25



Many times construction instrumentation must be placed in areas that interfere with the contractor's operation and may become damaged. The contractor must be alerted that they need to account for instrumentation in their operation. This is typically accomplished by plan notes and special provisions. This example note requires the contractor to repair any instrumentation damaged by their operation and to stop work in that area of the project until the repair is complete.

Slide 26

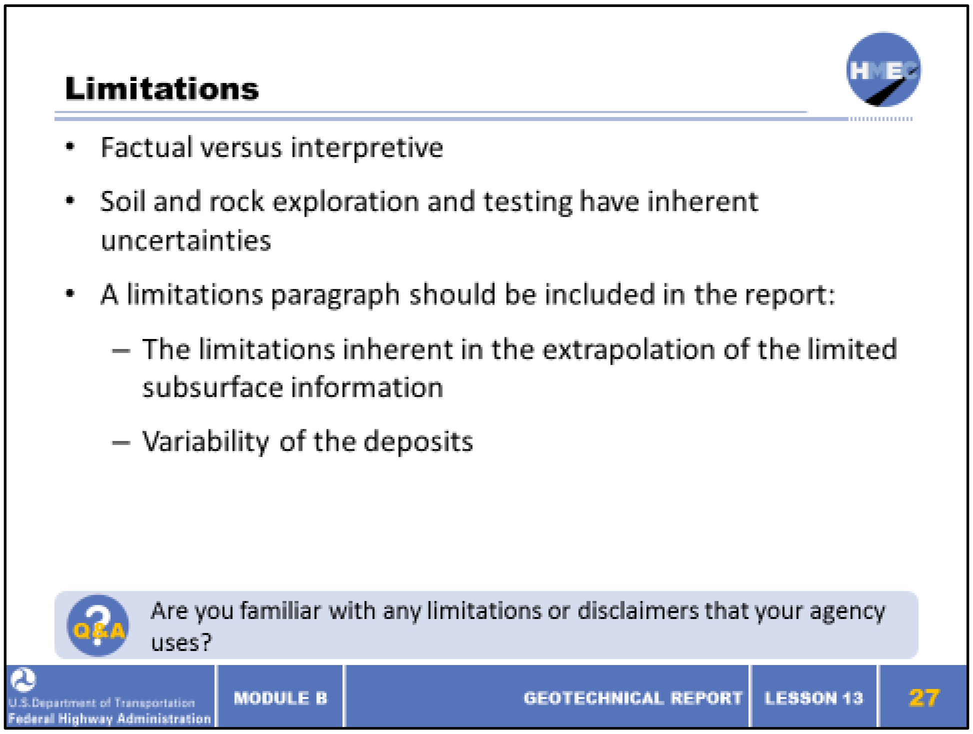


The information developed during the subsurface exploration is very useful in the selection of effective construction procedures and for estimating construction costs. Such information is, therefore, of value to knowledgeable contractors bidding on the project.

The finished boring logs and/or generalized soil profile should be made available to bidders and included with the contract plans. Other subsurface information, such as soil and rock samples and results of field and laboratory testing, should also be made available for inspection by bidders. The invitation for bids should indicate the type of information available and when and where it may be inspected. The highway agency should have a system for documenting what information each contractor inspects. Such documentation can be of major importance in the event of later claim actions.

The more information the bidders have available, the lower the risk (uncertainty) they will need to include in their bid and the more competitive their bid will be.

Slide 27



One of the best surveys of the problem, of what information to provide, was prepared by Standing Subcommittee No. 4 of the U.S. National Committee on Tunneling Technology. The subcommittee was composed of engineers and attorneys that have experience dealing with owners, engineering firms, and contracting organizations.

The following is excerpted from their recommendations:

In sum, all subsurface data obtained for a project, professional interpretations thereof, and the design considerations based on these data and interpretations should be included in the bidding documents or otherwise made readily available to prospective contractors. Fact and opinion should be clearly separated.

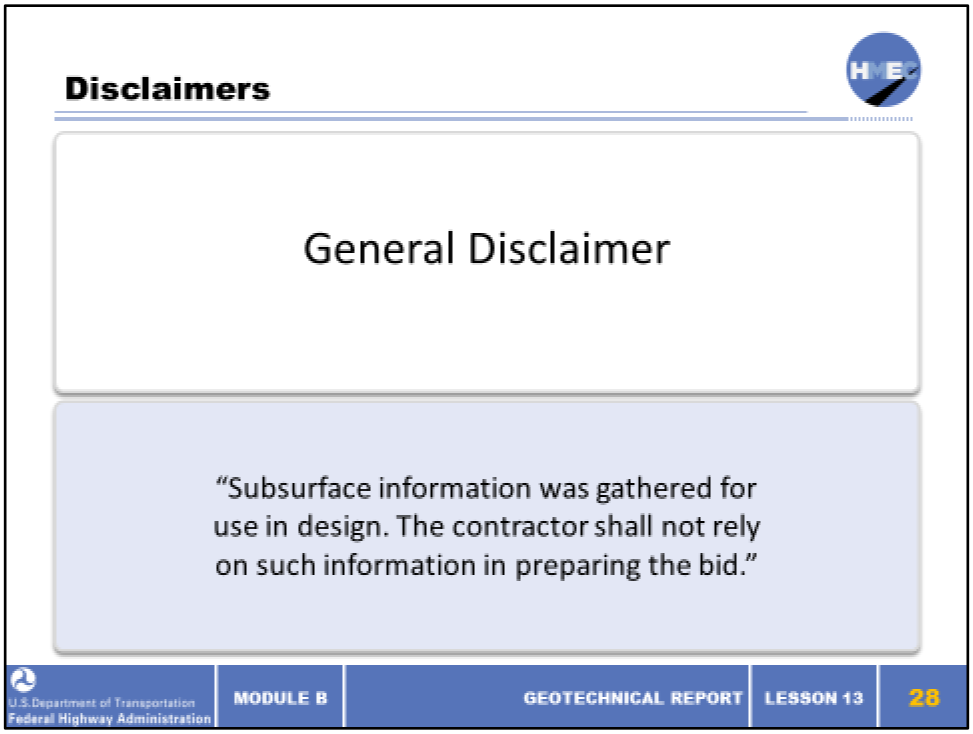
The bidder should be entitled to rely on the basic subsurface data, with no obligation to conduct his own subsurface survey.

It is considered, however, that specific disclaimers of responsibility for accuracy are appropriate, with respect to the following categories:

• Information obtained by others, perhaps at other times and for other purposes, which is being furnished to prospective bidders in order to comply with the legal obligation to make full disclosure of all available data.
• Interpretations and opinions drawn from basic subsurface data because equally competent professionals may reasonably draw different interpretations from the same basic data.

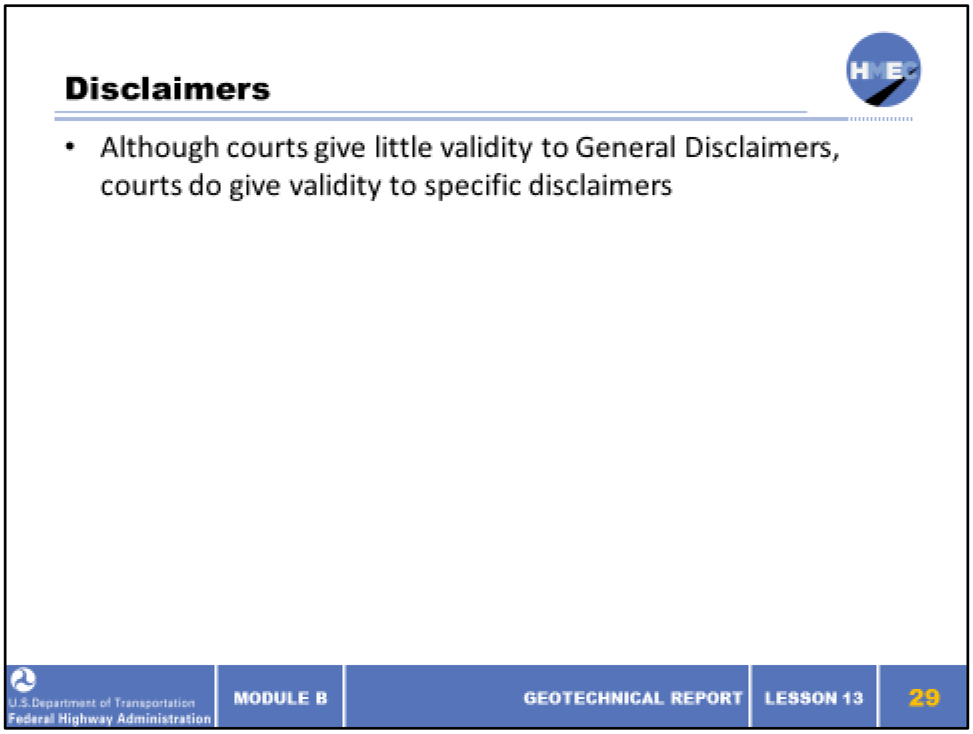
Soil and rock exploration and testing have inherent uncertainties. Thus users of the data who are unfamiliar with the variability of natural and manmade deposits should be informed in the report of the limitations inherent in the extrapolation of the limited subsurface information obtained from the site investigation.

Slide 28



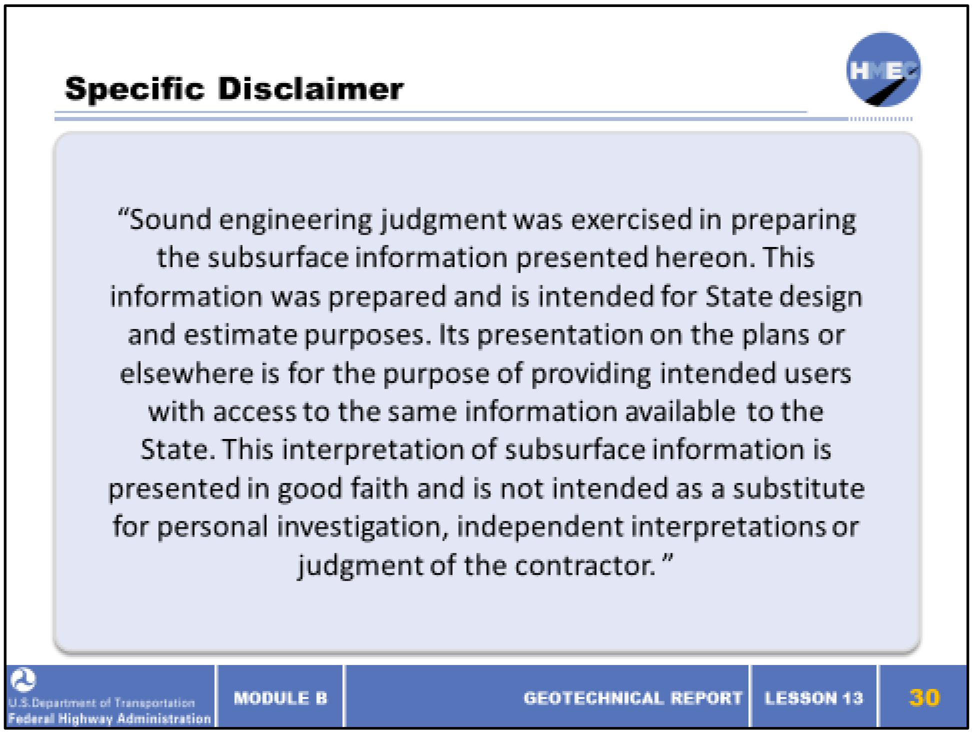
This is an example of a general disclaimer. Courts have typically given little weight to general disclaimers. If the contractor can't rely on the information the owner/engineer used for design, what information should they base their bid on?

Slide 29



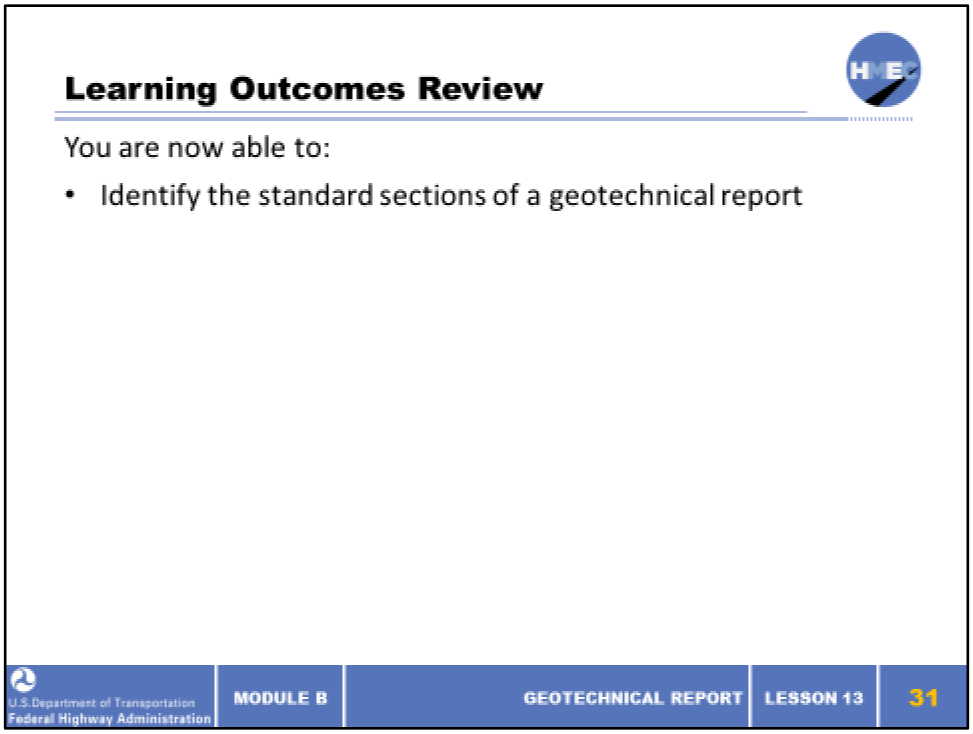
The courts have generally upheld the use of "specific" disclaimer clauses. The use of specific disclaimer clauses is strongly recommended over the use of general disclaimer clauses.

Slide 30



The owner has told the bidders/contractor that the owner used the information presented for design and that the bidders/contractors need to apply their own judgment when using the information.

Slide 31


Learning Outcomes Review
HMEC
You are now able to:
• Identify the standard sections of a geotechnical report
U.S. Department of Transportation
Federal Highway Administration
MODULE B
GEOTECHNICAL REPORT
LESSON 13
31

This page is intentionally blank.

Slide 1


ILT
HMEC
Highway Materials Engineering Course
Lesson 14: Hot Topics
Soils and Foundations
U.S.Department of Transportation
Federal Highway Administration
MODULE
B

This page is intentionally blank.

Appendix A: Acronyms

The following are acronyms referenced throughout the course that are important agencies or organizations:

Acronym	Proper Name
AASHTO	American Association of State Highway and Transportation Officials
ACAA	American Coal Ash Association
ACI	American Concrete Institute
ACPA	American Concrete Paving Association
AI	Asphalt Institute
ASTM	American Society for Testing and Materials
AWS	American Welding Society
CFR	Code of Federal Regulations
DOT	U.S. Department of Transportation
EPA	Environmental Protection Agency
FHWA	Federal Highway Administration
NACE	National Association of Corrosion Engineers
NAPA	National Asphalt Pavement Association
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NEPCOAT	North East Protective Coating
NHI	National Highway Institute
NRC	National Recycling Coalition
NRMCA	National Ready Mixed Concrete Association
NSA	National Slag Association
NSBA	National Steel Bridge Alliance

Acronym	Proper Name
NTPEP	National Transportation Product Evaluation Program
OSHA	Occupational Safety and Health Administration
RCSC	Research Council on Structural Connections
SSPC	Society for Protective Coatings
TRB	Transportation Research Board
USGS	U.S. Geological Survey

Appendix B: Resources

Additional information regarding Module B can be found in the following sources.

Primary Resources

- Modified Unified Description
- NHI 132012 Soils and Foundations Reference Manual (RM), Volumes I and II
- NHI 131023 Soils and Foundations (Module 3)
- NHI 132033 Soil Slope and Embankment Design and Construction
- Rock and Mineral Identification for Engineers

Additional Resources

- Abandoned Underground Mines: http://www.fhwa.dot.gov/engineering/geotech/hazards/mine/
- Developing a Subsurface Model Flowchart: http://harpersenterprise.adobeconnect.com/hmecb04subsurfacemodel/
- Development of a Quantitative Model for the Mechanism of Raveling Failure in Highway Rock Slopes using LIDAR: http://transportation.mst.edu/media/research/transportation/documents/R274.pdf
- Geocomp Corporation: www.geocomp.com
- Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties (GEC Category) http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm
- Geotechnical Engineering Circular No. 6: Shallow Foundations (GEC Category) http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm
- Geotechnical Engineering Circular No. 10 Drilled Shafts: Construction Procedures and LRFD Design Methods (GEC Category) http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm
- Hydraulics Engineering Publications (HEC) http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm
- National Geographic Database Map: http://ngmdb.usgs.gov/ngmdb/ngmdb_home.html
- Rock Slide Video: http://www.youtube.com/watch?v=uOJfcTZME0U

- Subsurface Investigations Geotechnical Site Characterization Reference Manual (Subsurface Exploration Category): http://www.fhwa.dot.gov/engineering/geotech/library_sub.cfm?keyword=017

AASHTO and ASTM Standards

- AASHTO M 145
- AASHTO R 13
- AASHTO R 58
- AASHTO T 88
- AASHTO T 89
- AASHTO T 90
- AASHTO T 207
- AASHTO T 208
- AASHTO T 215
- AASHTO T 216
- AASHTO T 236
- AASHTO T 256
- AASHTO T 267
- AASHTO T 296
- AASHTO T 297
- AASHTO T 307
- ASTM D 1587
- ASTM D 2166
- ASTM D 2434
- ASTM D 2435

- ASTM D 2487
- ASTM D 2488
- ASTM D 2974
- ASTM D 3080
- ASTM D 4186
- ASTM D 4220
- ASTM D 4452
- ASTM D 4546
- ASTM D 4767
- ASTM D 4945
- ASTM D 5079
- ASTM D 5084
- ASTM D 5321
- ASTM D 5333
- ASTM D 6528
- ASTM D 7181

NHI/FHWA Courses and Documents

- FHWA ED-88-053 Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications (January 2003)
- FHWA NHI-05-123 Common Stability Problems Soil Slope and Embankment Design
- FHWA TS-80-219 Design and Construction of Shale Embankments
- NHI 132012 Apple Freeway example project (example of project, not for use in NHI 131023 Soils and Foundations module)
- NHI 132014 Drilled Shafts
- NHI 132021 Driven Pile Foundations (Design and Construction)
- NHI 132022 Driven Pile Foundations (Construction Monitoring)
- NHI 132034 Ground Improvement Techniques
- NHI 132035 Rock Slopes

- NHI 132036 Earth Retaining Structures
- NHI 132037 Spread Footings: Load and Resistance Factor Design (LRFD) and Construction
- NHI 132040 Geotechnical Aspects of Pavements
- NHI 132043 Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
- NHI 132070 Drilled Shaft Foundation Inspection
- NHI 132079 Subsurface Investigation Qualification
- NHI 132080 Inspection of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
- NHI 132081 Slope Maintenance and Slide Restoration
- NHI 1320082A LRFD for Highway Bridge Substructures and Earth Retaining Structures
- NHI 132083 Implementation of LRFD Geotechnical Design for Bridge Foundations
- NHI 132094 LRFD Seismic Analysis