



**PDHonline Course C370 (5 PDH)**

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# **Geotechnical Exploration and Testing for Roads**

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## CHAPTER 4.0 GEOTECHNICAL EXPLORATION AND TESTING

### 4.1 INTRODUCTION

The purpose of the geotechnical subsurface investigation program for pavement design and construction is to obtain a thorough understanding of the subgrade conditions along the alignment that will constitute the foundation for support of the pavement structure. The specific emphasis of the subsurface investigation is to identify the impact of the subgrade conditions on the construction and performance of the pavement, characterize material from cut sections that may be used as subgrade fill, and to obtain design input parameters. The investigation may be accomplished through a variety of techniques, which may vary with geology, design methodology and associated design requirements, type of project and local experience. To assist agencies in achieving the stated purpose of subsurface investigation, this chapter presents the latest methodologies in the planning and execution of the various exploratory investigation methods for pavement projects. It is understood that the procedures discussed in this chapter are subject to local variations. Users are also referred to AASHTO R 13 and ASTM D 420, *Conducting Geotechnical Subsurface Investigations* and FHWA NHI-01-031 *Subsurface Investigations*, for additional guidance.

In Chapter 1, a simplistic subsurface exploration program consisting of uniformly spaced soil borings (*i.e.*, systematic sampling) with SPT testing was mentioned as an antiquated method for determining the subsurface characteristics for pavement design. “Adequate for design and low cost” are often used in defense of this procedure. The cost-benefit of additional subsurface exploration is a subject that is often debated. This subject is now addressed in the new NCHRP 1-37A Design Guide. The guide allows for use of default values in the absence of sufficient data for characterizing the foundation, thus minimizing agency design costs, but at the increased risk of over- or under-designing the pavement structure.

In evaluating the cost-benefit of the level of subsurface investigation, all designers must recognize that the reliability and quality of the design will be directly related to the subsurface information obtained. The subsurface exploration program indeed controls the quality of the roadway system. A recent FHWA study indicated that a majority of all construction claims were related to inadequate subsurface information. With great certainty, inadequate information will lead to long-term problems with the roadway design. The cost of a subsurface exploration program is a few thousand dollars, while the cost of over-conservative designs or costly failures in terms of construction delays, construction extras, shortened design life, increased maintenance, and public inconvenience is typically in the hundreds of thousands of dollars.

Engineers should also consider that the actual amount of subgrade soil sampled and tested is typically on the order of one-millionth to one-billionth of the soil being investigated. Compare this with sampling and testing of other civil engineering materials. Sampling and testing of concrete is on the order of 1 sample (3 test specimens, or about ¼ cubic meter) every 40 cubic meters, which leads to 1 test in 100,000. Sampling and testing of asphalt is on the same order as concrete. Now consider that the variability in properties of these well-controlled, manufactured materials is much less than the properties of the subgrade, which often have coefficients of variation of well over 100% along the alignment. Again cost, not quality is usually the deciding factor. The quality of sampling can be overcome with conservative designs (as is often the case; *e.g.*, AASHTO 1972). For example, laboratory tests are often run on soil samples in a weaker condition than in the ground, rather than running more tests on the full range of conditions that exist in the field. While this approach may provide a conservative value for design purposes, there are hidden costs in both conservatism and questionable reliability. Modern pavement design uses averages with reliability factors to account for uncertainty (AASHTO, 1993 and NCHRP 1-37A). However, sufficient sampling and testing are required to check the variability of design parameters to make sure that they are within the bounds of reliability factors; otherwise, on highly variable sites designs, they will not be conservative and on very uniform sites, they will still be over conservative.

The expense of conducting soil borings is certainly a detriment to obtaining subsurface information. However, exploration itself is not just doing borings. There is usually a significant amount of information available from alternate methods that can be performed prior to drilling to assist in optimizing boring and sampling locations (*i.e.*, representative sampling). This is especially the case for reconstruction and rehabilitation projects. Significant gains in reliability can be made by investigating subgrade spatial variability in a pavement project and often at a cost reduction due to decreased reliance on samples. This chapter provides guidelines for a well-planned exploration program for pavement design, with alternate methods used to overcome sampling and testing deficiencies. Geotechnical exploration requirements for borrow materials (base, subbase, and subgrades) are also reviewed.

Figure 4-1 provides a flow chart of the process for performing a geotechnical exploration and testing program. As shown in the flow chart, the steps for planning and performing a complete geotechnical and testing program include

### **Subsurface Exploration Steps**

- 1) Establish the type of pavement construction.
- 2) Search available information.
- 3) Perform site reconnaissance.
- 4) Plan the exploration program for evaluation of the subsurface conditions and identification of the groundwater table, including methods to be used with consideration for using
  - remote sensing,
  - geophysical investigations,
  - in-situ testing,
  - disturbed sampling, and
  - undisturbed sampling.
- 5) Evaluate conceptual designs, examine subsurface drainage and determine sources for other geotechnical components (*e.g.*, base and subbase materials).
- 6) Examine the boring logs, classification tests, soil profiles and plan view, then select representative soil layers for laboratory testing.

### **Relevance to Pavement Design**

Whether new construction, reconstruction, or rehabilitation.

To identify anticipated subsurface conditions at the vertical and horizontal location of the pavement section.

To identify site conditions requiring special consideration.

To identify and obtain

- more information on site conditions,
- spacial distribution of subsurface conditions,
- rapid evaluation of subsurface condition,
- subgrade soils & classification test samples,
- samples for resilient modulus tests and calibration of in-situ results.

Identify requirements for subsurface drainage and subgrade stabilization requirements, as well as construction material properties.

Use the soil profile and plan view along the roadway alignment to determine resilient modulus or other design testing requirements for each influential soil strata encountered.

Each of these steps will be reviewed in the following sections of this chapter.

## **4.2 LEVELS OF GEOTECHNICAL EXPLORATION FOR DIFFERENT TYPES OF PAVEMENT PROJECTS**

There are three primary types of pavement construction projects. They are

- new construction,
- reconstruction, and
- rehabilitation.

Each of these pavement project types requires different considerations and a corresponding level of effort in the geotechnical exploration program.

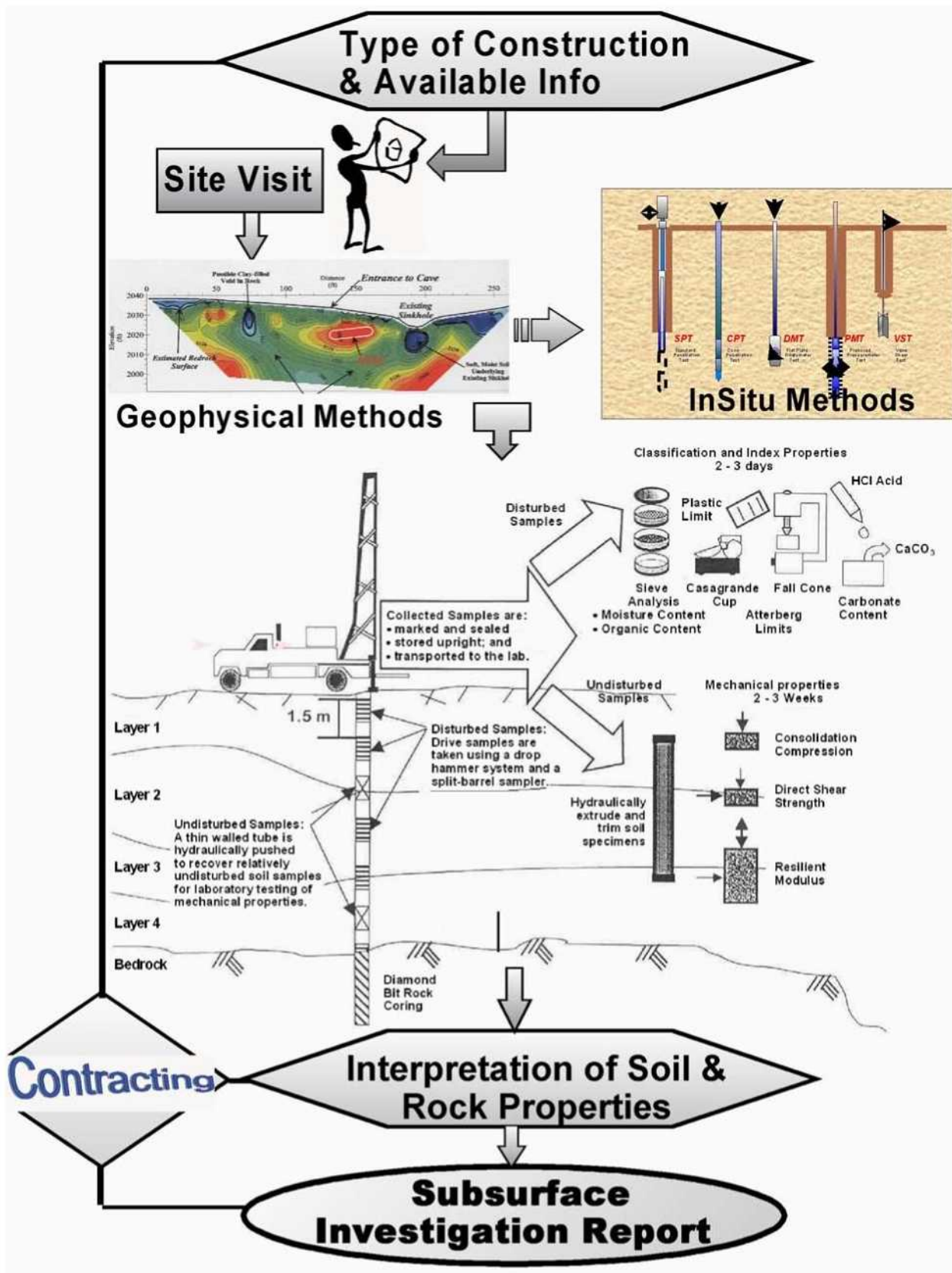


Figure 4-1. Geotechnical exploration and testing for pavement design.

### 4.2.1 New Pavement Construction

For new construction, the exploration program will require a complete evaluation of the subgrade, subbase, and base materials. Sources of materials will need to be identified and a complete subsurface exploration program will need to be performed to evaluate pavement support conditions. Prior to planning and initiating the investigation, the person responsible for planning the subsurface exploration program (*i.e.*, the geotechnical engineer or engineer with geotechnical training) needs to obtain from the designers the type, load, and performance criteria, location, geometry and elevations of the proposed pavement sections. The locations and dimensions of cuts and fills, embankments, retaining structures, and substructure elements (*e.g.*, utilities, culverts, storm water detention ponds, etc.) should be identified as accurately as practicable.

Also, for all new construction projects, samples from the subgrade soils immediately beneath the pavement section and from proposed cut soils to be used as subgrade fill will be required to obtain the design-input parameters for the specific design method used by the agency. Available site information (*e.g.*, geological maps and United States Department of Agriculture Natural Resources Conservation Service's soil survey reports) as discussed in Section 4.3, site reconnaissance (see Section 4.4), air photos (see Section 4.5.3) and geophysical tests (see Section 4.5.4) can all prove beneficial in identify representative and critical sampling locations.

For all designs using AASHTO 1993 or NCHRP 1-37A, particularly for critical projects, repeated load resilient modulus tests are needed to evaluate the support characteristics and the effects of moisture changes on the resilient modulus of each supporting layer. The procedures, sample preparation and interpretation of the resilient modulus test are discussed in Chapter 5. For designs based on subgrade strength, either lab tests (*e.g.*, CBR) as discussed in Chapter 5 or in-situ tests (*e.g.*, DCP) as discussed later in Section 4.5.5 of this chapter can be used to determine the support characteristics of the subgrade.

Another key part of subsurface exploration is the identification and classification (through laboratory tests) of the subgrade soils in order to evaluate the vertical and horizontal variability of the subgrade and select appropriate representative design tests. Field identification along with classification through laboratory testing also provides information to determine stabilization requirements to improve the subgrade should additional support be required, as discussed in Chapter 7.

Location of the groundwater table is also an important aspect of the subsurface exploration program for new construction to evaluate water control issues (*e.g.*, subgrade drainage

requirements) with respect to both design and construction. Methods for locating the groundwater level are discussed in Section 4.5.6. Other construction issues include the identification of rock in the construction zone, rock rippability, and identification of soft or otherwise unsuitable materials to be removed from the subgrade. The location and rippability of rock can be determined by geophysical methods (*e.g.*, seismic refraction), as discussed in section 4.5.4 and/or borings and rock core samples.

#### **4.2.2 Reconstruction**

For pavement reconstruction projects, such as roadway replacement, full depth reclamation, or road widening, information may already exist on the subgrade support conditions from historical subsurface investigations. Existing borings should be carefully evaluated with respect to design elevation of the new facility. A survey of the type, severity, and amount of visible distress on the surface of the existing pavement (*i.e.*, a condition survey as described in the NHI, 1998, “Techniques for Pavement Rehabilitation” Participant’s Manual) can also indicate local issues that need a more extensive evaluation. However, an additional limited subsurface investigation is usually advisable to validate the pavement design calculations and design for weak subgrade conditions, if present. It is also likely that resilient modulus, CBR or other design input values used by agencies would need to be obtained for the existing materials using current procedures. Test methods used by the agency often change over time (*e.g.*, lab CBR versus field CBR). Previous data may also not be valid for current conditions (*e.g.*, traffic). Water in old pavements can often result in poorer subgrade conditions than originally encountered. Drainage features, or lack thereof, in the existing pavement and their functionality should be examined. Again, subgrade soil identification and classification will be required to provide information on subgrade variability and assist in selection of soils to be tested.

It is possible to determine the value of reworking the subgrade (*i.e.*, scarifying, drying, and recompacting) if results indicate stiffness and/or subgrade strength values are below expected or typical values. This comparison can be made by examining the resilient modulus of undisturbed tube samples obtained to verify backcalculated moduli to that of a recompacted specimen remolded to some prescribed level of density and moisture content. For example, this comparison may ultimately lead to the need for underdrain installation in order to reduce and maintain lower moisture levels in the subgrade.

Subsurface investigation on reconstruction projects can usually be facilitated by using non-destructive tests (NDT) (a.k.a. geophysical methods) performed over the old pavement (or shoulder section for road widening) with one or more of the variety of methods presented in Section 4.5. For example, resilient modulus properties can best be obtained from non-

destructive geophysical methods, such as falling weight deflectometer (FWD) tests and back calculating elastic moduli to characterize the existing structure and foundation soils needed for design. This approach is suggested because it provides data on the response characteristics of the in-situ soils and conditions. Back calculation of layer elastic moduli from deflection basin data is discussed later in Section 4.5.4 of this chapter. These results can be supported by laboratory tests on samples obtained from a minimal subsurface exploration program (described in Section 4.5). Old pavement layer thickness (i.e., asphalt or concrete, base and/or subbase) should also be obtained during sampling to provide information for back-calculation of the modulus values.

For designs based on subgrade strength (*e.g.*, CBR), in-situ tests (*e.g.*, Dynamic Cone Penetrometer (DCP), field CBR, and other methods as described in Section 4.7) can be performed to obtain a rapid assessment of the variability in subgrade strength and to determine design strength values via correlations. Some samples should still be taken to perform laboratory tests and confirm in-situ test correlation values. Geophysical test results (*e.g.*, FWD, Ground Penetrating Radar (GPR), and others described in Section 4.5.4) can also be used to assist in locating borings.

The potential sources of new base and subbase materials will need be identified and laboratory tests performed to obtain resilient modulus, CBR or other design values, unless catalogued values exist for these engineered materials. For pavement reclamation or recycling projects, composite samples should be obtained from the field and test specimens constituted following the procedures outlined in Chapter 5 to obtain design input values. The subgrade soils will also need to be evaluated for their ability to support construction activities, such as rubblize-and-roll type construction.

### **4.2.3 Rehabilitation**

As discussed in Chapter 3, rehabilitation projects include a number of strategies, including overlays, rubbilization, and crack and seat. The details required for the subsurface investigation of pavement rehabilitation projects depends on a number of variables:

- The condition of the pavement to be rehabilitated (*e.g.*, pavement rutting, cracking, riding surface uniformity and roughness, surface distress, surface deflection under traffic, presence of water, etc., as described in the condition survey section of NHI, 1998, “Techniques for Pavement Rehabilitation” Participants Manual.)
- If the facility is distressed, the type, severity and extent of distress (pavement distress, pavement failures, crack-type pattern, deep-seated failures, settlement, drainage and water flow, and collapse condition) (see NHI, 1998, “Techniques for Pavement Rehabilitation” Participants Manual) should be quantified. Rutting and fatigue



cracking are often associated with subgrade issues and general require coring, drilling, and sampling to diagnose the cause of these conditions.

- Techniques to be considered for rehabilitation.
- Whether the facility will be returned to its original and as-built condition, or whether it will be upgraded, for example, by adding another lane to a pavement. If facilities will be upgraded, the proposed geometry, location, new loads and structure changes (*e.g.*, added culverts) must be considered in the investigation.
- The required performance period of the rehabilitated pavement section.

Selection of the rehabilitation alternative will partly depend on the condition assessment. NHI, 1998, “Techniques for Pavement Rehabilitation” Participants Manual covers condition surveys and selection of techniques for pavement rehabilitation. Information from the subsurface program performed for the original pavement design should also be reviewed. However, as with reconstruction projects, some additional corings and borings will need to be performed to evaluate the condition and properties of the of the pavement surface and subgrade support materials. Pavements are frequently cored at 150 – 300 m (500 – 1000 ft) intervals for rehabilitation projects. The core holes in the pavement also provide access to investigate the in-situ and disturbed properties of the base, subbase, and subgrade materials. Samples can be taken and/or in-situ tests (*e.g.*, DCP) can be used to indicate structural properties, as well as layer thickness.

Geophysical tests (*e.g.*, FWD, GPR, and others described in Section 4.5.4) can be used to assist in locating coring and boring locations, especially if the base is highly contaminated or there are indications of subgrade problems. Otherwise, the frequency of corings and borings should be increased. As with reconstruction projects, rehabilitation projects can use FWD methods and associated back-calculated elastic modulus to characterize the existing structure and foundation. Again, the FWD method is covered in Section 4.5.4 and back-calculation of layer elastic moduli from deflection basin data is discussed in Chapter 5. FWD results can also be correlated with strength design values (*e.g.*, CBR). A limited subsurface drilling and sampling program can then be used to confirm the back-calculated resilient modulus values and/or correlation with other strength design parameters. The layer thickness of each pavement component (*i.e.*, surface layer, base, and or subbase layer) is critical for back-calculation of modulus values.

#### **4.2.4 Subsurface Exploration Program Objectives**

As stated in the NCHRP 1-37A Design Guide, the objective of subsurface investigation or field exploration is to obtain sufficient subsurface data to permit the selection of the types, locations, and principal dimensions of foundations for all roadways comprising the proposed

project, thus providing adequate information to estimate their costs. More importantly, these explorations should identify the site in sufficient detail for the development of feasible and cost-effective pavement design and construction.

As outlined in the FHWA Soils and Foundation Workshop manual (FHWA NHI-00-045), the subsurface exploration program should obtain sufficient subsurface information and samples necessary to define soil and rock subsurface conditions as follows:

- 1) Stratigraphy (for evaluating the areal extent of subgrade features)
  - a) Physical description and extent of each stratum
  - b) Thickness and elevation of various locations of top and bottom of each stratum
- 2) For cohesive soils (identify soils in each stratum, as described in Section 4.7, to assess the relative value for pavement support and anticipated construction issues, *e.g.*, stabilization requirements)
  - a) Natural moisture contents
  - b) Atterberg limits
  - c) Presence of organic materials
  - d) Evidence of desiccation or previous soil disturbance, shearing, or slickensides
  - e) Swelling characteristics
  - f) Shear strength
  - g) Compressibility
- 3) For granular soils (identify soils in each stratum, as described in Section 4.7, to assess the relative value for pavement support and use in the pavement structure)
  - a) In-situ density (average and range)
  - b) Grain-size distribution (gradations)
  - c) Presence of organic materials
- 4) Groundwater (for each aquifer within zone of influence on construction and pavement support, especially in cut sections as detailed in Section 4.5)
  - a) Piezometric surface over site area, existing, past, and probable range in future
  - b) Perched water table
- 5) Bedrock (and presence of boulders) (within the zone of influence on construction and pavement support as detailed in Section 4.5)
  - a) Depth over entire site
  - b) Type of rock
  - c) Extent and character of weathering
  - d) Joints, including distribution, spacing, whether open or closed, and joint infilling
  - e) Faults
  - f) Solution effects in limestone or other soluble rocks
  - g) Core recovery and soundness (RQD)
  - h) Ripability

### 4.3 SEARCH AVAILABLE INFORMATION

The next step in the investigation process is to collect and analyze all existing data. A complete and thorough investigation of the topographic and subsurface conditions must be made prior to planning the field exploration program so that it is clear where the pavement subgrade will begin and to identify the type of soils anticipated within the zone of influence of the pavement. The extent of the site investigation and the type of exploration required will depend on this information. (“If you do not know what you should be looking for in a site investigation, you are not likely to find much of value.” Quote from noted speaker at the 8<sup>th</sup> Rankine Lecture). Simply locating borings without this information is like sticking a needle in your arm blindfolded and hoping to hit the vein. A little sleuthing can greatly assist in gaining an understanding of the site and planning the appropriate exploration program.

An extensive amount of information can be obtained from a review of literature about the site. There are a number of very helpful sources of data that can and should be used in planning subsurface investigations. Review of this information can often minimize surprises in the field, assist in determining boring locations and depths, and provide very valuable geologic and historical information, which may have to be included in the exploration report.

The first information to obtain is prior agency subsurface investigations (historical data) at or near the project site, especially for rehabilitation and reconstruction projects. To determine its value, this data should be carefully evaluated with respect to location, elevation, and site variability. Also, in review of data, be aware that test methods change over time. For example, SPT values 20 to 30 years ago were much less efficient than today, as discussed in Section 4.5.5. Prior construction and records of structural performance problems at the site (*e.g.*, excessive seepage, unpredicted settlement, and other information) should also be reviewed. Some of this information may only be available in anecdotal forms. For rehabilitation and reconstruction projects, contact agency maintenance personnel and discuss their observations and work along the project alignment. The more serious construction and/or maintenance problems should be investigated, documented if possible, and evaluated by the engineer.

In this initial stage of site exploration, for new pavement projects, the major geologic processes that have affected the project site should also be identified. Geology will be a key factor to allow the organization and interpretation of findings. For example, if the pavement alignment is through an ancient lakebed, only a few representative borings will be required to evaluate the pavement subgrade. However, in highly variable geologic conditions, additional borings (*i.e.*, in excess of the normal minimum) should be anticipated. Geological information is especially beneficial in pavement design and construction to identify the

presence and types of shallow rock, rock outcrops, and rock excavation requirements. Geological information can readily be obtained from U.S. Geological Survey (USGS) maps, reports, publications and websites ([www.usgs.gov](http://www.usgs.gov)), and State Geological Survey maps and publications.

Soils deposited by a particular process assume characteristic topographic features, called landforms, which can be readily identified by a geotechnical specialist or geologist. A landform contains soils with generally similar engineering properties and typically extends irregularly over wide areas of a project alignment. The soil may be further described as a residual or transported soil. A residual soil has been formed at a location by the in-place decomposition of the parent material (sedimentary, igneous, or metamorphic rock). Residual soils often contain a structure and lose strength when disturbed. A transported soil was formed at one location and has been transported by exterior forces (*e.g.*, water, wind, or glaciers). Alluvium soils are transported by water, loess type soils are transported by wind, and tills are transported by ice. Transported soils (especially alluvium and loess) are often fine grained and are usually characterized as poorly draining, compressible when saturated, and frost susceptible (*i.e.*, not the most desirable soils for supporting pavement systems). Sources of information for determining landform boundaries and their functional uses are given in Table 4-1.

One of the more valuable sources of landform information for pavement design and construction are soil survey maps produced by the U.S. Department of Agriculture, Natural Resources Conservation Service, in cooperation with state agricultural experiment stations and other Federal and State agencies. The county soil maps provide an overview of the spatial variability of the soil series within a county. These are well-researched maps and provide detailed information for shallow surficial deposits, especially valuable for pavements at or near original surface grade. They may also show frost penetration depths, drainage characteristics, and USCS soil types. Knowledge of the regional geomorphology (*i.e.*, the origin of landforms and types of soils in the region and the pedologic soil series definitions) is required to take full advantage of these maps. Such information will be of help in planning soil exploration activities. Plotting the pavement alignment on a USDA map and/or a USGS map can be extremely helpful. Figure 4-2 shows an example for a section of the Main Highway project.

The majority of the above information can be obtained from commercial sources (*i.e.*, duplicating services) or U.S. and state government, local or regional offices. Specific sources (toll-free phone numbers, addresses, etc.) for flood and geologic maps, aerial photographs, USDA soil surveys, can be very quickly identified through the Internet (*e.g.*, at the websites listed in Table 4-1).

**Table 4-1. Sources of topographic & geologic data for identifying landform boundaries.**

Source	Functional Use
Topographic maps prepared by the United States Coast and Geodetic Survey (USCGS).	Determine depth of borings required to evaluate pavement subgrade; determine access for exploration equipment; identify physical features, and find landform boundaries.
County agricultural soil maps and reports prepared by the U.S. Department of Agriculture's Natural Resources Conservation Service (a list of published soil surveys is issued annually, some of which are available on the web at <a href="http://soils.usda.gov/survey/online_surveys/">http://soils.usda.gov/survey/online_surveys/</a> ).	Provide an overview of the spatial variability of the soil series within a county.
U.S. Geological Survey (USGS) maps, reports, publications and websites ( <a href="http://www.usgs.gov">www.usgs.gov</a> ), and State Geological Survey maps and publications.	Type, depth, and orientation of rock formations that may influence pavement design and construction.
State flood zone maps prepared by state or U.S. Geological Survey or the Federal Emergency Management Agency (FEMA: <a href="http://www.fema.gov">www.fema.gov</a> ) can be obtained from local or regional offices of these agencies.	Indicate deposition and extent of alluvial soils, natural flow of groundwater, and potential high groundwater levels (as well as danger to crews in rain events).
Groundwater resource or water supply bulletins (USGS or State agency).	Estimate general soils data shown, and indicate anticipated location of groundwater with respect to pavement grade elevation.
Air photos prepared by the United States Geologic Survey (USGS) and others ( <i>e.g.</i> , state agencies).	Detailed physical relief shown; flag major problems. By studying older maps, reworked landforms from development activities can be identified along the alignment, <i>e.g.</i> , buried streambed or old landfill.
Construction plans for nearby structures (public agency).	Foundation type and old borings shown.

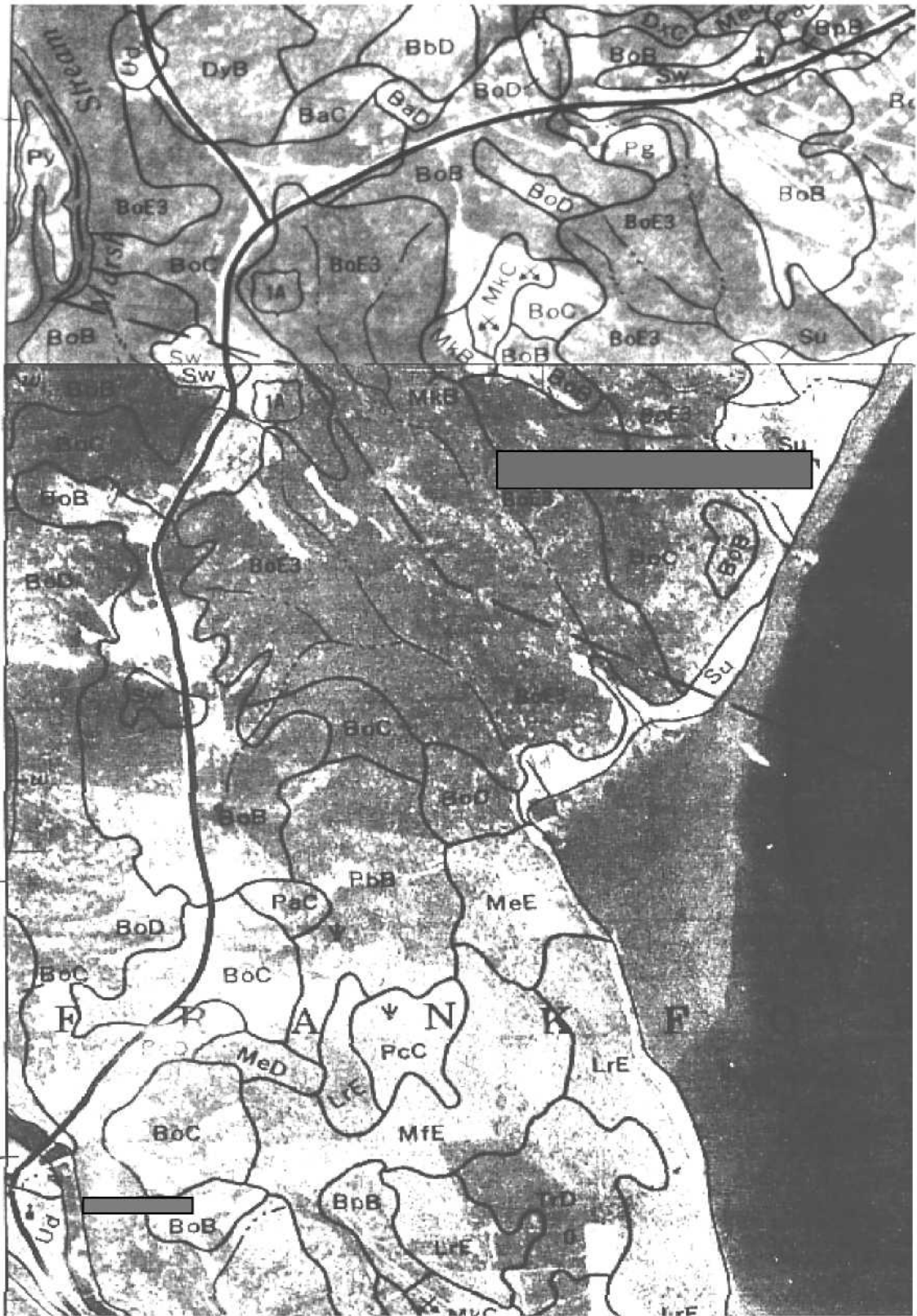


Figure 4-2. Soil Survey information along the Main Highway pavement alignment.

#### 4.4 PERFORM SITE RECONNAISSANCE

A very important step in planning the subsurface exploration program is to visit the site with the project plans (*i.e.*, a plan-in-hand site visit). It is imperative that the engineer responsible for exploration, and, if possible, the project design engineer, conduct a reconnaissance visit to the project site to develop an appreciation of the geotechnical, topographic, and geological features of the site and become knowledgeable of access and working conditions. A plan-in-hand site visit is a good opportunity to learn about

- design and construction plans.
- general site conditions including special issues and local features, such as lakes and streams, exploration and construction equipment accessibility.
- surficial geologic and geomorphologic reconnaissance for mapping stratigraphic exposures and outcrops and identifying problematic surficial features, such as organic deposits.
- type and condition of existing pavements at or in the vicinity of the project.
- traffic control requirements during field investigations (a key factor in the type of exploration, especially for reconstruction and rehabilitation projects).
- location of underground and overhead utilities for locating in-situ tests and borings. (For pavement rehab projects, the presence of underground utilities may also support the use of non-destructive geophysical methods to assist in identifying old utility locations.
- adjacent land use (schools, churches, research facilities, etc.).
- restrictions on working hours (*e.g.*, noise issues), which may affect the type of exploration, as well as the type of construction.
- right-of-way constraints, which may limit boring locations.
- environmental issues (*e.g.*, old service stations for road widening projects).
- escarpments, outcrops, erosion features, and surface settlement.
- flood levels (as they relate to the elevation of the pavement and potential drainage issues).
- benchmarks and other reference points to aid in the location of borehole.
- subsurface soil and rock conditions from exposed cuts in adjacent works.

For reconstruction or rehabilitation projects, the site reconnaissance should include a condition survey of the existing pavement as detailed in NHI (1998) “Techniques for Pavement Rehabilitation.” During this initial inspection of the project, the design engineer, preferably accompanied by the maintenance engineer, should determine the scope of the primary field survey, begin to assess the potential distress mechanisms, and identify the

candidate rehabilitation alternatives. As part of this activity, subjective information on distress, road roughness, surface friction, and moisture/drainage problems should be gathered. Unless traffic volume is a hazard, this type of data can be collected without any traffic control, through both “windshield” and road shoulder observations. In addition, an initial assessment of traffic control options (both during the primary field survey and during rehabilitation construction), obstructions, and safety aspects should be made during this visit.

#### **4.5 PLAN AND PERFORM THE SUBSURFACE EXPLORATION PROGRAM**

Following the collection and evaluation of available information from the above sources, the geotechnical engineer (or engineer or geologist with geotechnical training) is ready to plan the field exploration program. The field exploration methods, sampling requirements, and types and frequency of field tests to be performed will be determined based on the existing subsurface information obtained from the available literature and site reconnaissance, project design requirements, the availability of equipment, and local practice. A geologist can often provide valuable input regarding the type, age, and depositional environment of the geologic formations present at the site for use in planning and interpreting the site conditions.

The subsurface investigation for any pavement project should be sufficiently detailed to define the depth, thickness, and area of all major soil and rock strata that will affect construction and long-term performance of the pavement structure. The extent of the exploration program depends on the nature of both the project and the site-specific subsurface conditions. To acquire reliable engineering data, each job site must be explored and analyzed according to its subsurface conditions. The engineer in charge of the subsurface exploration must furnish complete data so that an impartial and thorough study of practical pavement designs can be made.

##### **4.5.1 Depth of Influence**

Planning the subsurface exploration program requires a basic understanding of the depth to which subsurface conditions will influence the design, construction, and performance of the pavement system. For pavement design, the depth of influence is usually assumed to relate only to the magnitude and distribution of the traffic loads imposed on the pavement structure under consideration. Current AASHTO (1993) describes this depth at 1.5 m (5 ft) below the proposed subgrade elevation with this depth increased for special circumstances (*e.g.*, deep deposits of very soft soils). In this section, support for the recommended depth is provided, and special circumstances where this depth should be extended are reviewed.



The zone of influence under the completed pavement varies with the pavement section, but typically 80 – 90 percent of the applied stress is dissipated within 1 m (3 ft) below the asphalt section as shown in Figure 4-3. However, consideration must also be given to the roadway section (*i.e.*, height and width of the roadway embankment for fill application), the nature of the subsurface conditions, and consideration for construction (*e.g.*, depth of soils that may require stabilization to allow for construction). A common rule of thumb in geotechnical engineering is that the depth of influence is on the order of two times the width of the load. This adage is also true for pavement sections during construction and for unpaved roads. Considering a dual wheel is about 1 m (3 ft) in width, subsurface investigations for shallow cut and fill with no special problems should generally extend to 1.5 – 2 m (5 – 7 ft) below the proposed subgrade level to account for construction conditions. Special problems requiring deeper exploration may include deep highly compressible deposits (*e.g.*, peat or marsh areas) or deep deposits of frost-susceptible soils in cold regions. Greater depths may also be required for embankment design.

From a pavement design perspective, the critical layers are in the upper meter of the subgrade. This understanding is especially critical for rehabilitation projects. Mechanistic design is based upon the critical horizontal tensile strain at the base of an asphalt layer or the critical vertical compressive strain at the surface of the subgrade (and within the other pavement layers) under repetitions of a specific wheel or axle load (Huang, 1993). Subgrade strain often controls the pavement design except for very thick asphalt layers or overlays. For rehabilitation projects and in consideration of sampling for roadway design, the depth of influence should be evaluated based on the type of pavement and the reconstruction layering. The subsurface investigations should focus on these depths (typically the upper 1 to 2 meters). However, as discussed in Chapter 3, groundwater and bedrock at depths of less than 3 m (10 ft) beneath the pavement can have an influence on pavement design. In addition, location of the groundwater level within 3 m (10 ft) of the pavement will influence decisions of frost susceptibility, as discussed later in Chapter 7, and the presence of bedrock within 6 – 9 m (20 – 30 ft) can influence deflections of pavement layers and FWD results. Therefore, in order to confirm that there are no adverse deeper deposits, to identify groundwater conditions, and to locate bedrock within the influence zone, a limited amount of exploration should always be performed to identify conditions in the subgrade to depths of 6 m (20 ft). However, as discussed in the next section, this does not necessarily mean borings to that depth.

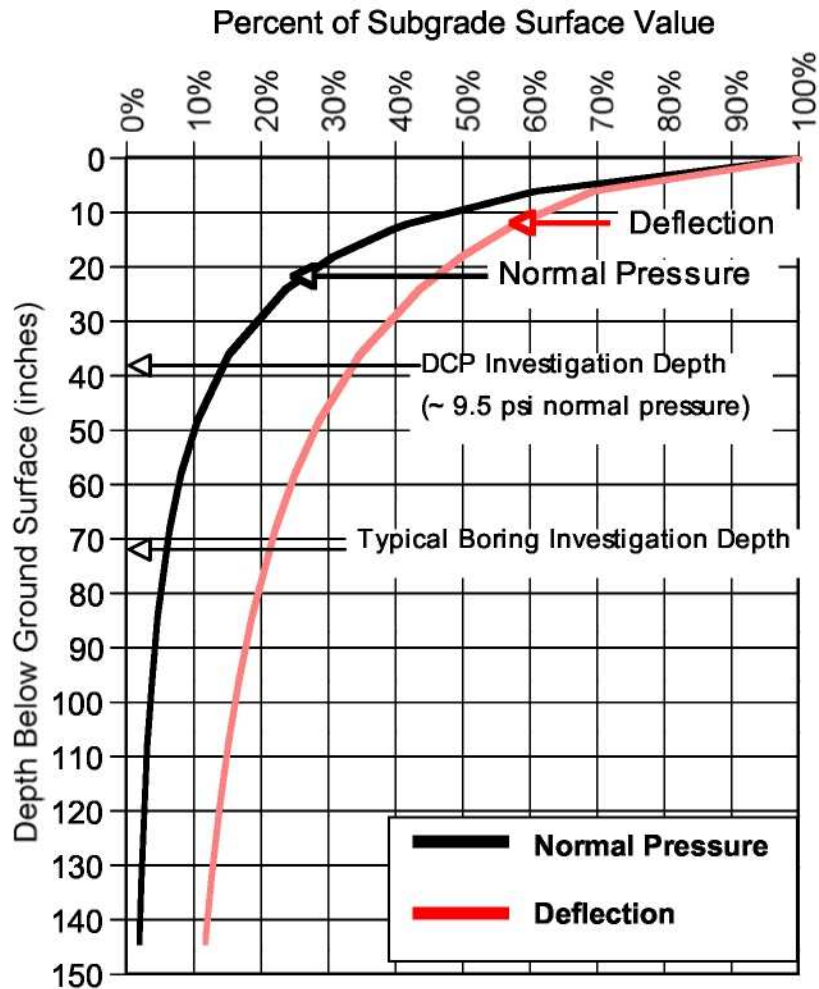


Figure 4-3. Typical zone of influence for an asphalt pavement section (Vandre et al., 1998).

#### 4.5.2 Subsurface Exploration Techniques

Generally, there are four types of field subsurface investigation methods, best conducted in this order:

1. Remote sensing
2. Geophysical investigations
3. In-situ investigation
4. Borings and sampling

All of these methods are applicable for pavement design. For example, in new pavement construction projects, the location of old streambeds, usually containing soft, organic deposits that will require removal or stabilization, can usually be identified by remote

sensing. Once identified, the vertical and horizontal extent of the streambed deposit can be explored by using geophysics to determine the horizontal extent, followed by in-situ tests to quantify the vertical extent and qualitatively evaluate soil properties, and borings with samples to quantify soil properties. The extent of use for a specific exploration method will be dependent upon the type of pavement project (*i.e.*, new construction, reconstruction, or rehabilitation), as discussed in the following subsections.

### **4.5.3 Remote Sensing**

Remote sensing data from satellite and aircraft imagery can effectively be used to identify terrain conditions, geologic formations, escarpments and surface reflection of faults, buried stream beds, site access conditions, and general soil and rock formations that may impact new pavement design and construction. Infrared imagery can also be used to identify locally wet areas.

While remote sensing methods are most valuable for new construction, this information may also be used to explain poor performance of existing pavements in rehabilitation and reconstruction projects.

Remote sensing data from satellites (*e.g.*, LANDSAT images from NASA), aerial photographs from the USGS or state geologists, U.S. Corps of Engineers, commercial aerial mapping service organizations) can be easily obtained. State DOTs also use aerial photographs for right-of-way surveys and road and bridge alignments, and they can make them available for use by the engineer responsible for exploration. Especially valuable are old air photos compared to new ones in developed areas, which often identify buried features, such as old streambeds. Some ground control (*e.g.*, borings) is generally required to verify the information derived from remote sensing data.

### **4.5.4 Geophysical Investigations**

Geophysical survey methods can be used to selectively identify boring locations, supplement borehole data, and interpolate between borings. There are several kinds of geophysical tests that can be used for stratigraphic profiling and delineation of subsurface geometries. These include the measurement of mechanical waves (deflection response, seismic refraction surveys, crosshole, downhole, and spectral analysis of surface wave tests), as well as electromagnetic techniques (resistivity, EM, magnetometer, and radar). Mechanical waves are additionally useful for the determination of elastic properties of subsurface media, primarily the small-strain shear modulus. Electromagnetic methods can help locate anomalous regions, such as underground cavities, buried objects, and utility lines. The

geophysical tests do not alter the soil conditions and, therefore, classify as *non-destructive*. Several are performed at the surface level (termed *non-invasive*). The advantages of performing geophysical methods include

- nondestructive and/or non-invasive,
- fast and economical testing,
- provide theoretical basis for interpretation, and are
- applicable to soils and rocks.

The primary disadvantage is that no samples or direct physical penetration tests are taken. Models are also assumed for interpretation, which sometimes appears to be an art. The results are also affected by cemented layers or inclusions, and are influenced by water, clay, and depth.

The most common geophysical methods used for pavement evaluation is deflection response testing, with a majority of the agencies using the falling weight deflectometer (FWD) impulse type method (as previously mentioned in Sections 4.2.2 and 4.2.3 for rehabilitation and reconstruction projects). In rehabilitation and reconstruction projects, this method provides a direct evaluation of the stiffness of the existing pavement layers under simulated traffic loading. FWD, especially the newer lightweight deflectometers (LWD), can also be used during the construction of new pavements to confirm subgrade stiffness characteristics, either for verifying design assumptions or providing a quality control (QC) tool. LWD, along with other methods used for evaluating the stiffness of natural or compacted subgrades for construction control, are discussed in Chapter 8.

Other deflection methods include steady-state dynamic methods, which produce a sinusoidal vibration in the pavement, and quasi-static devices, which measure pavement deflections from a slow, rolling load (*e.g.*, the Benkelman beam). The most promising development in deflection methods is the high-speed deflectometers, which measure deflections while continuously moving. While these methods increase the complexity of measurement, they offer significant advantages in terms of safety, through reduced traffic control requirements, productivity (typically 3 – 20 km/hr {2 – 12 mph}), and increased volume of information. A detailed review of each of these deflection methods is provided in NCHRP Synthesis 278 (Newcomb and Birgisson, 1999), and guidelines for deflection measurements are provided in ASTM, one on general dynamic deflection equipment (ASTM D 4695) and one on falling-weight-type impulse load devices (ASTM D 4602).

Electrical-type geophysical tests may also be used in pavement design and construction, including surface resistivity (SR), ground penetrating radar (GPR), electromagnetic conductivity (EM), and magnetic survey (MS). These electrical methods are based on the resistivity or, conversely, the conductivity of pore water in soil and rock materials. Mineral

grains comprised of soils and rocks are essentially nonconductive, except in some exotic materials, such as metallic ores, so the resistivity of soils and rocks is governed primarily by the amount of pore water, its resistivity, and the arrangement of the pores. These techniques allow for mapping of the entire surface area of the site, making them useful in imaging the generalized subsurface conditions and detecting utilities, hidden objects, boulders, and other anomalies. The mapping is conducted on a relative scale of measurements that reflect changes across the property. For rehabilitation projects and reconstruction projects, GPR is often used for mapping the thickness of existing pavement layers. Electrical-type methods may also aid in finding underground cavities, caves, sinkholes, and erosion features in limestone and dolomite terrain. In developed areas, they may be used to detect underground utility lines, buried tanks and drums, and objects of environmental concern. Additional details on SR, EM, GPR, and MS can be found in Greenhouse, et al. (1998), FHWA manual on *Application of Geophysical Methods to Highway Related Problems* (Wightman et al., 2003), and in the geophysical information portion of the Geoforum website at <http://www.geoforum.com/info/geophysical/>.

Mechanical wave geophysical methods are also used in pavement design and construction, including seismic refraction, seismic reflection, and, most recently, spectral analysis of surface waves (SASW). Both methods can be used to locate the depth to bedrock. Seismic refraction is also a key method for estimating rippability of rock. The use of the SASW mechanical wave method for determining subgrade modulus values for pavement design has recently been demonstrated in field trails. An automated device has been developed and is being tested by the Texas DOT. However, the testing and interpretation time is still somewhat long for use in pavement applications (Newcomb and Birgisson, 1999, and Wightman et al., 2003).

A general summary for each of the more common geophysical methods used in pavement design is outlined in Tables 4-2 through 4-7. Application examples are provided following the tables.

**Table 4-2. Falling weight deflectometer (FWD).**

Reference Procedures	ASTM D4694 ( <i>Deflections with a Falling Weight Type Impulse Load Device</i> ); <i>LTPP Manual for Falling Weight Deflectometer Measurement: Operational Field Guidelines</i> (August 2000).
Purpose	Used to determine the variation of pavement layer and subgrade stiffness along a length of pavement. For geotechnical features, can be used to backcalculate resilient modulus of subgrade and previously constructed base layers and to identify areas (e.g., weak subgrade conditions) requiring boring and sampling.
Procedure	As described in ASTM D4694, the FWD method consist of applying an impulse load to the paved or unpaved road surface using a falling weight, typically between 4 – 107 kN (1,000 – 24,000 lbs), dropped on a plate resting on the pavement surface, as shown in Figure 4-4 below. The peak force at impact is measured by a load cell and can be recorded as the impact force or the mean stress (by dividing the load by the plate area). The vertical deflection of the pavement surface is measured at the center of the applied load and at various distances (up to eight locations are typical) away from the load, as shown in Figure 4-4. The method usually uses a vehicle or a trailer that is brought to a stop with the loading plate positioned over the desired test location. Several tests may be performed at the same location and at the same or different heights. By measuring the deflection response at the same location under different loads (drop heights), the linear or non-linear characteristics of the pavement system and individual layers can be evaluated. The vehicle is then moved to the next location. The plate and deflection sensors are lowered to the pavement surface. For routine surveys, the tests are typically performed on a spacing of 20 – 50 m (70 – 160 ft) along the road. The deflection at the center gives an idea of the overall pavement performance and the difference in deflection between deflections at various distances indicates the conditions of the pavement layers (bound, unbound, and subgrade). Profiles of the deflections can then be plotted over the length of the pavement (Figure 4.4c), in order to show the variation of pavement layer and subgrade stiffness.

The deflection bowls obtained from the FWD data can be analyzed to back calculate the effective stiffness (or load spreading ability) of the various pavement and subgrade layers, by matching measured deflections to computed values. Back calculation is most commonly performed using a multi-layer linear elastic model for the pavement layers. For example, the effective pavement modulus, which is a measure of the effective or combined stiffness of all layers above the subgrade, can be determined from the center deflection as follows (AASHTO, 1993):

$$d_0 = 1.5pa \left\{ \frac{1}{M_R \sqrt{1 + \left( \frac{D}{a} \sqrt[3]{\frac{E_P}{M_R}} \right)^2}} + \frac{1 - \frac{1}{\sqrt{1 + \left( \frac{D}{a} \right)^2}}}{E_P} \right\}$$

where,

$d_0$  = deflection measured at the center of the load plate (and adjusted to a standard temperature of 20°C {68°F} for hot mix asphalt), inches

$p$  = load plate pressure, psi

$a$  = load plate radius, inches  
 $D$  = total thickness of pavement layers above the subgrade, inches  
 $E_p$  = effective modulus of all pavement layers above the subgrade, psi  
 1 in = 25.4 mm, 1 psi = 6.7 kPa

Outer deflection reading of the deflection basin primarily reflects the in-situ modulus of the lower soil (or subgrade). The subgrade resilient modulus can be calculated as follows:

$$M_R = \frac{0.24P}{d_r r}$$

$$r \geq 0.7a_e$$

$$a_e = \sqrt{\left[ a^2 + \left( D \sqrt[3]{\frac{E_p}{M_R}} \right)^2 \right]}$$

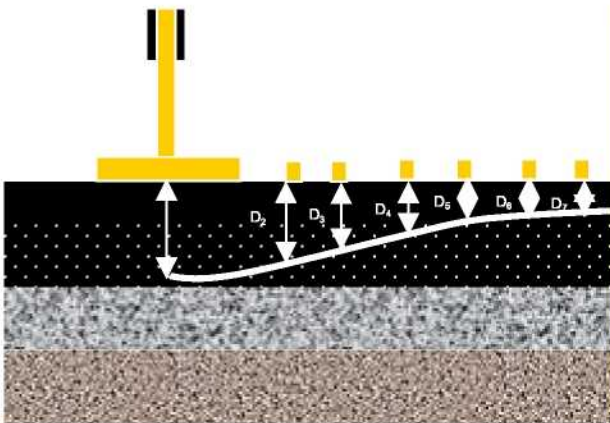
where,

$M_R$  = back-calculated subgrade resilient modulus, psi  
 $P$  = applied load, pound  
 $d_r$  = deflection at a distance  $r$  from the center of the load, inches  
 $r$  = distance from center of load, inches  
 $a_e$  = radius of the stress bulb at subgrade-pavement interface, inches  
 $a$  = load plate radius, inches  
 $D$  = total pavement thickness above subgrade, inches  
 1 in = 25.4 mm, 1 psi = 6.7 kPa,

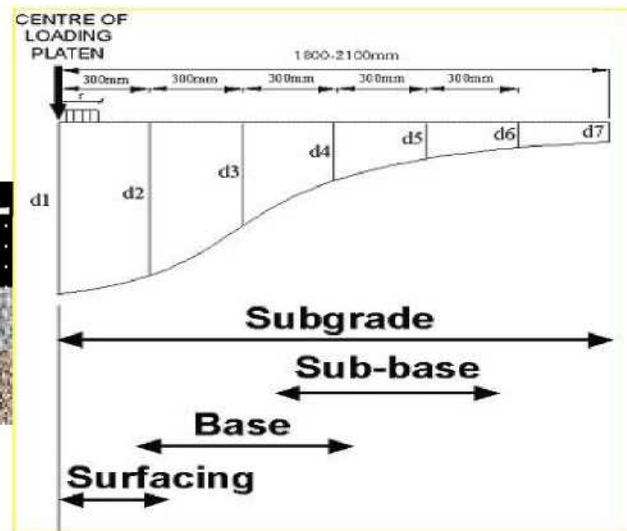
Subgrade resilient modulus for design purposes is usually less than the value directly from FWD data. The AASTHO design guide (1993) recommends a design subgrade resilient modulus equal to 33% of that back calculated from FWD data for flexible pavement and 25% of the back-calculated value for PCC pavement.



a)



b)



c)

Figure 4.4. FWD showing: a) typical equipment; b) schematic of procedure; and, c) sketch of deflection bowl for interpretation of results.



Commentary

The FWD produces a dynamic impulse load that simulates a moving wheel load, rather than a static, semi-static, or vibratory load. FWD tests can be used for all construction types (*i.e.*, new construction, rehabilitation, or reconstruction). For new construction, testing can be performed directly on cleared subgrade, or done during construction, after placement of the subbase, base, or pavement surface layer. The method can also be used to evaluate the effectiveness of subgrade improvements (for weak subgrades). Based on the results of FWD, the roadway section can be delineated into design sections with similar properties and the intrusive explorations methods (*i.e.*, in-situ testing and borings) located accordingly to obtain the thickness of layers, confirm subgrade stiffness values, and obtain other characteristics of the subgrade (*e.g.*, soil type and moisture conditions).

The deflection profile along the project may be examined to determine if changes exist in the pavement's structural response. The profile can be used to assist in locating areas where more intensive sampling and testing will be required, greatly improving the efficiency of laboratory evaluation. The profile may also be used to divide the project into design sections. For example, in rehabilitation projects, FWD results can be used to optimize the overlay design and/or subdrainage design in each of the design sections.

It should be noted that the influence depth for elastic deflections measured with FWD may extend more than 9 m (30 ft) and, as a result, may miss near-surface critical features. Also, the results may be influenced by deep and often unknown conditions. FWD results are also affected by temperature and freezing conditions. Thus, it is important to consider the time of day and the season when scheduling the FWD program. As previously indicated, deflection measurements are corrected to a standard temperature, typically 20°C (68°F), and critical season equivalent deflections based on locally-developed procedures.

ADVANTAGES

- Speed, repeatability & equipment robustness
- Easily transported
- Simulates moving traffic loads
- Direct evaluation of design  $M_R$  values
- Non-destructive

DISADVANTAGES

- Static method requires stopping between readings
- Traffic control required
- Deep features (*e.g.*, water table and bedrock) and temperature affect results
- $M_R$  over predicted
- Requires well-defined layer thickness

**Table 4-3. Surface resistivity (SR).**

Reference	ASTM G57 ( <i>Field Measurement of Soil Resistivity [Wenner Array]</i> )
Procedures	
Purpose	Resistivity is used to locate bedrock, stratigraphy, wet regions, compressible soils, map faults, karstic features, contamination plumes, buried objects, and other uses.
Procedure	Resistivity is a fundamental electrical property of geomaterials and can be used to evaluate soil types and variations of pore fluid and changes in subsurface media (Santamarina et al., 2001). The resistivity ( $\rho_R$ ) is measured in ohm-meters and is the reciprocal of electrical conductivity ( $k_E = 1/\rho_R$ ). Conductivity is reported in siemens per meter (S/m), where S = amps/volts. Electrical current is put into the ground using two electrodes, and the resulting voltage is measured using two other electrodes. Using pairs or arrays of electrodes embedded into the surface of the ground, a surface resistivity survey can be conducted to measure the difference in electrical potential of an applied current across a site. The spacing of the electrodes governs the depth of penetration by the resistivity method and the interpretation is affected by the type of array used (Wenner, dipole-dipole, Schlumberger). The entire site is gridded and subjected to parallel arrays of SR-surveys, if a complete imaging map is desired. Mapping allows for relative variations of soil types to be discerned, as well as unusual features. Recently developed <i>automated resistivity systems</i> collects much more data than simple SR and combines resistivity sounding and traverse data to form a resistivity section with detailed interpretation, as shown in Figure 4-5.
Commentary	In general, resistivity values increase with soil grain size. Figure 4-6 presents some illustrative values of bulk resistivity for different soil and rock types. Figure 4-7 shows the field resistivity illustrative showing stratigraphic changes. Downhole resistivity surveys can also be performed using electronic probes that are lowered vertically down boreholes, or are direct-push placed. The latter can be accomplished using a resistivity module that trails a cone penetrometer, termed a resistivity piezocone (RCPTu). Downhole resistivity surveys are particularly advantageous in distinguishing the interface between upper freshwater and lower saltwater zones in coastal regions.

For new pavement design, surface resistivity can be used to evaluate the areal extent of soil deposits and assist in identifying sample locations. Resistivity is also related to moisture content and can be used to map variations in moisture, and, thus, regions of compressible soils can be delineated. This moisture relation can also be valuable for rehabilitation and reconstruction projects, indicating areas requiring special considerations, such as improved drainage. SR may also be useful in determining the depth of rock, which as previously indicated may have an influence on FWD results and is a design input for ME design. SR can also be used in construction to assist in locating prospective sand, gravel, or other sources of borrow material.

<u>ADVANTAGES</u>	<u>DISADVANTAGES</u>
- Moderately fast: ~150 m/hr (500 ft/hr)	- Requires coring concrete or asphalt to insert electrodes
- Fairly simple	- Traffic control required
- Can evaluate significant depths	- Lateral resistivity variations affect results
- Works for higher- or lower-resistance sublayers	- Nearby grounded metal objects affect data
- Automation improves interpretation	- Wetting electrodes required in dry ground

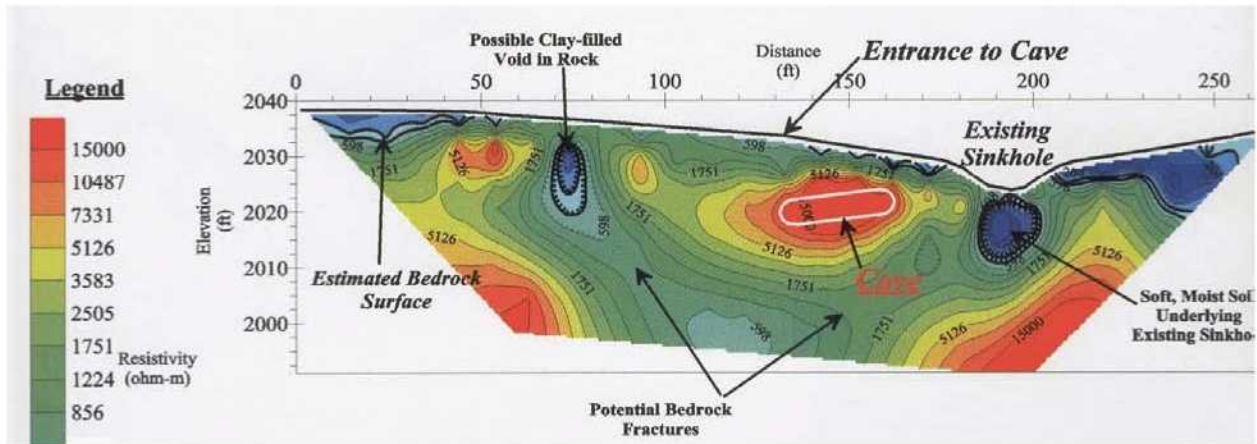


Figure 4-5. Two-dimensional cross-section resistivity profile for detection of sinkholes and caves in limestone (from Schnabel Engineering Associates).

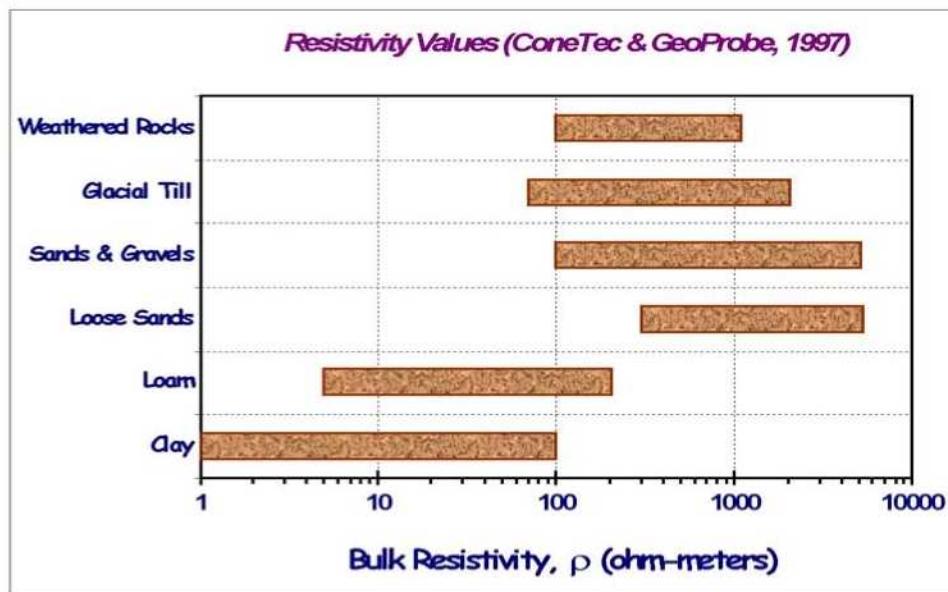


Figure 4-6. Representative values of resistivity for different soils (Mayne et al., 2002).

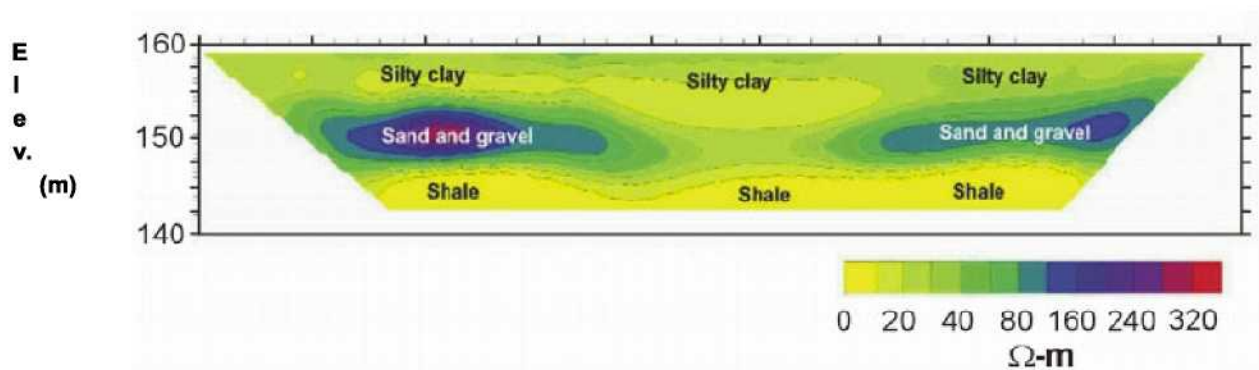


Figure 4-7. Resistivity data showing stratigraphic changes (Advanced Geosciences, Inc.).

**Table 4-4. Ground-penetrating radar (GPR).**

Reference Procedures	Wightman et al., (2003) <i>Application of Geophysical Methods to Highway Related Problems</i> , FHWA Contract Number DTFH68-02-P-00083.				
Purpose	GPR can be a valuable tool used to define subsoil strata, moisture variation, depth to rock, voids beneath pavement, buried pipes, cables, as well as to characterize archaeological sites before soil borings, probes, or excavation operations. It can also be utilized to determine the thickness of pavement layers, thus complementing FWD evaluation, and mapping reinforcing steel in concrete surface pavements.				
Procedure	Short impulses of a high-frequency electromagnetic waves are transmitted into the ground using a pair of transmitting & receiving antennae. The reflected signals, which occur at dielectric discontinuities in the pavement system and subgrade, are recorded. Thus changes in the dielectric properties (permittivity) of the soil reflect relative changes in the subsurface environment. The GPR surveys are made by driving over the surface with air-coupled antennas mounted on the vehicle or pulling a tracking cart with ground-coupled antenna mounted on a sled across the ground surface. Air-coupled antennas are used to evaluate shallow depths (e.g., thickness of pavement layers) at highway speeds. Ground-coupled antennas are used to evaluate greater depths (up to 18 m (60 ft)). The EM frequency and electrical conductivity of the ground control the depth of penetration of the GPR survey. Many commercial systems come with several sets of paired antennas to allow variable depths of exploration, as well as accommodate different types of ground.				
Commentary	<p>The GPR surveys provide a quick imaging of the subsurface conditions, leaving everything virtually unchanged and undisturbed. In pavement engineering practice, GPR using air-coupled antenna is most commonly used to identify layer thickness of pavement materials and perform condition evaluation of pavement surface materials. Methods for improving the accuracy of thickness measurements are reported by Al-Qadi et al., 2003. For subsurface evaluation, ground-coupled GPR is required. The GPR subsurface surveys are particularly successful in deposits of dry sands with depths of penetration up to 20 m or more (65 ft). In wet, saturated clays, GPR is limited to shallow depths of only 1 – 3 m (3 – 10 ft) (still adequate for pavement subgrade evaluation. Searches below the water table are difficult and, in some cases, not possible. Several illustrative examples of GPR surveys are shown in Figure 4-8. Additional information on current usage of GPR by state agencies is contained in NCHRP Synthesis 255 on <i>Ground Penetrating Radar for Evaluating Subsurface Conditions for Transportation Facilities</i> (Morey 1998).</p> <p>A recent development (GeoRadar) uses a variably-sweeping frequency to capture data at a variety of depths and soil types. Other developments include combining the use of air-coupled antenna with ski-mounted, ground-coupled antenna to allow for surface and subsurface evaluation at highway speeds.</p> <table border="0" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="text-align: center; border-bottom: 1px solid black;"><i>ADVANTAGES</i></th> <th style="text-align: center; border-bottom: 1px solid black;"><i>DISADVANTAGES</i></th> </tr> </thead> <tbody> <tr> <td style="vertical-align: top;"> <ul style="list-style-type: none"> <li>- Fast: 2 – 80 km/h (1 – 50 mph) &amp; easy to use</li> <li>- Different antenna provide different penetration depths and resolution</li> <li>- Produces real-time, continuous subsurface data</li> <li>- Non-destructive</li> </ul> </td> <td style="vertical-align: top;"> <ul style="list-style-type: none"> <li>- Perception of difficult interpretation</li> <li>- May require traffic control</li> <li>- Restricted depth in saturated clay soils</li> </ul> </td> </tr> </tbody> </table>	<i>ADVANTAGES</i>	<i>DISADVANTAGES</i>	<ul style="list-style-type: none"> <li>- Fast: 2 – 80 km/h (1 – 50 mph) &amp; easy to use</li> <li>- Different antenna provide different penetration depths and resolution</li> <li>- Produces real-time, continuous subsurface data</li> <li>- Non-destructive</li> </ul>	<ul style="list-style-type: none"> <li>- Perception of difficult interpretation</li> <li>- May require traffic control</li> <li>- Restricted depth in saturated clay soils</li> </ul>
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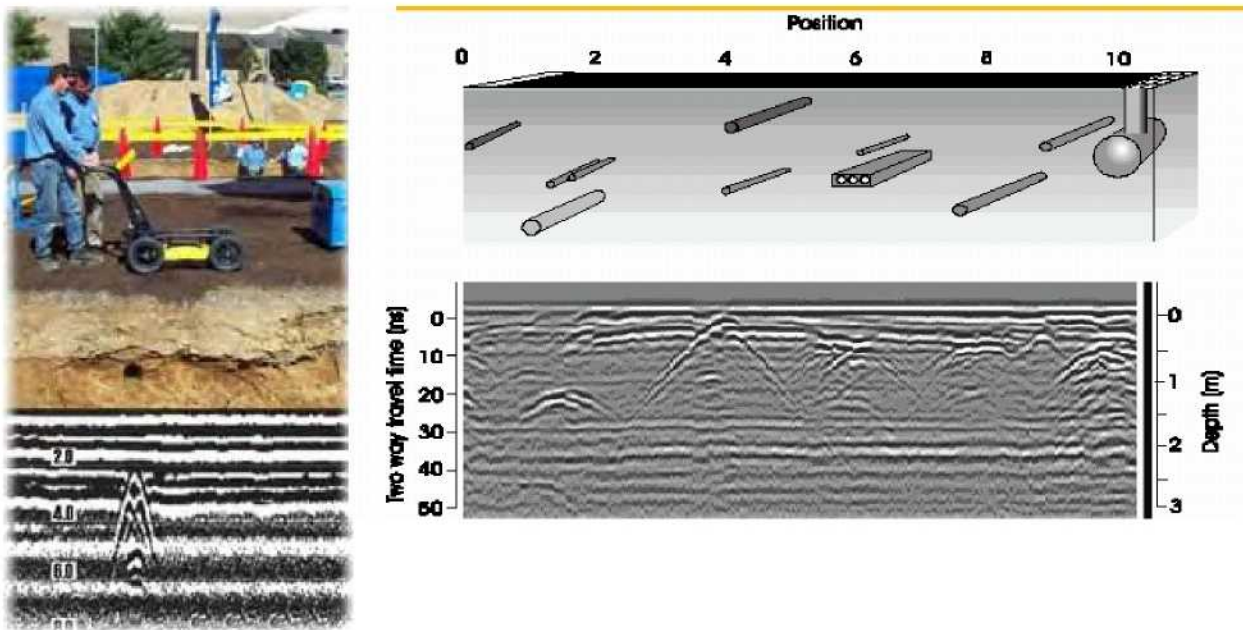


Figure 4-8. Representative ground-coupled GPR results showing buried utilities and soil profile (from EKKO Sensors & Software: [www.sensoft.on.ca](http://www.sensoft.on.ca)).

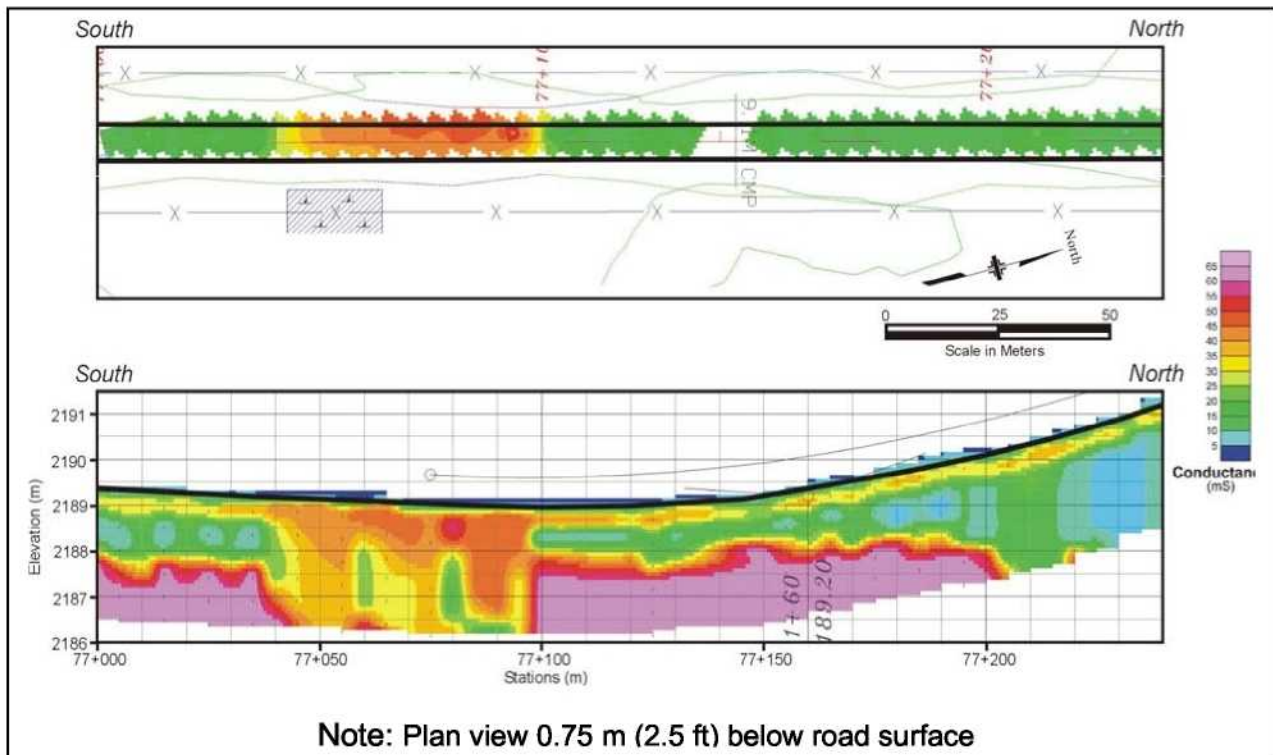


Figure 4-9. Conductivity results along a road in New Mexico (Blackhawk GeoServices, Inc).

**Table 4-5. Electromagnetic conductivity (EM).**

Reference Procedures	Wightman et al., (2003) <i>Application of Geophysical Methods to Highway Related Problems</i> , FHWA Contract Number DTFH68-02-P-00083.
Purpose	The EM methods provide a very good tool for identifying areas of clay beneath existing pavements in rehabilitation and road widening projects, or in the subgrade for new construction. These methods are also excellent at tracking buried metal objects and are well known in the utility locator industry. They can also be used to detect buried tanks, map geologic units, and groundwater contaminants, generally best within the upper 1 or 2 m (3 or 6 ft), yet can extend to depths of 5 m (16 ft) or more.
Procedure	<p>Several types of electromagnetic (EM) methods can be used to image the ground and buried features, including: induction, frequency domain, low frequency, and time domain systems. Ground conductivity methods can rapidly locate conductive areas in the upper few meters of the ground surface. These measurements are recorded using several instruments that use electromagnetic methods. Electromagnetic instruments that measure ground conductivity use two coplanar coils, one for the transmitter and the other for the receiver. The transmitter coil produces an electromagnetic field, oscillating at several kHz, that produces secondary currents in conductive material in the ground. The amplitude of these secondary currents depends on the conductivity of the material. These secondary currents then produce secondary electromagnetic fields that are recorded by the receiver coil. Surveys are best handled by mapping the entire site to show relative variations and changes. Areas of high electrical conductivity are likely places to find clay.</p> <p>The choice of which instrument to use generally depends on the depth of investigation desired. Instruments commonly used include the EM38, EM31, EM34 (Geonics Ltd, Canada), and GEM2 (Geophex, USA). The EM38 is designed to measure soil conductivities and has a maximum depth of investigation of about 1.5 m (5 ft). The EM31 has a depth of investigation to about 6 m (20 ft), and the EM34 has a maximum depth of investigation to about 60 m (200 ft). The depth of investigation of the GEM2 is advertised to be 30 – 50 m (100 – 165 ft) in resistive terrain (&gt;1000 ohm-m) and 20 – 30 m (65 – 100 ft) in conductive terrain (&lt;100 ohm-m).</p>
Commentary	Clay is almost always electrically conductive, and areas of high conductivity have a reasonable chance that they will contain clay (e.g., see Figure 4-9). However, estimating the amount of clay from conductivity measurements alone is generally not possible. Conductivity is influenced by many factors including the degree of saturation, porosity, and salinity of the pore fluids. Conductivity measurements taken with instruments that investigate to depths greater than the upper layer are also influenced by other layers.

ADVANTAGES

- Moderately fast: 2 – 8 km/hr (1 – 5 mph)
- Data is easy and efficient to record
- Instruments used in different modes to maximize information at tailored depths

DISADVANTAGES

- Influenced by above- and below-ground metal (e.g., fence post, utilities, rebar).
- Several passes may be required
- Traffic control required

**Table 4-6. Mechanical wave using seismic refraction.**

Reference	ASTM D5777 .
Procedures	Wightman et al., (2003) <i>Application of Geophysical Methods to Highway Related Problems</i> , FHWA Contract Number DTFH68-02-P-00083.
Purpose	Seismic refraction surveys are used to locate depth and characteristics (e.g., rippability) of bedrock, as well as evaluate dynamic elastic properties of the soil and rock .
Procedure	Seismic refraction involves placing a line of regularly spaced sensors (geophones) on the surface and measuring the relative arrival time of seismic energy transmitted from a specified source location. Seismic waves produced by the energy source penetrate the overburden and refract along the bedrock surface, continually radiating seismic waves back to the ground surface, as shown in Figure 4-10. Refraction data are recorded in the field using a portable seismograph, multiple (generally 12 per line) geophones (generally <15 Hz), a repeatable seismic source (e.g., sledgehammer striking a metal plate or light explosive charges), and a power source. Sledgehammer sources are generally used for depths less than 10 – 15 m (30 – 50 ft) and explosives for greater depths up to 30 m (100 ft). Mechanical waves generated by the seismic source include the compression (P-wave) and shear (S-wave) wave types that are measured. P waves are the first arrival waves, are the easiest to measure, and are not absorbed by saturated soil units (i.e., shear waves cannot transmit through water). Seismic energy travels with a compression velocity that is characteristic of the density, porosity, structure, and water content of each geologic layer. The seismic refraction survey is planned with respect to anticipated soil/rock velocities to be encountered, the approximate depth to rock, and the end-use of the data (e.g., rippability of the rock). Multiple seismic source points permit improved delineation of soil/rock interfaces.
Commentary	Seismic surveys are not intended to supplant subsurface sampling investigations, but aid in quickly and economically extending subsurface characterization over large areas, filling in the gaps between discrete borings.

Although a number of parameters (e.g., uniaxial strength, degree of weathering, abrasiveness, frequency of planes of weakness) relate to rippability of rock, seismic refraction has historically been the geophysical method utilized to predetermine the degree of rippability. Correlations of rock rippability as published by the Caterpillar Company are shown in Figure 4-11.

***ADVANTAGES***

- Lightweight equipment, 2-person crew
- Very effective at locating bedrock
- Well-established correlation with rippability
- Can be used where drilling is physically or economically limited
- Background seismic noise may interfere with data refinement and interpretation
- Lateral deposition may influence results

***DISADVANTAGES***

- Slow (but faster than borings)
- Traffic control required
- Velocities must increase in successively deeper strata.
- Water has a higher velocity than soil and some weak or highly jointed rock

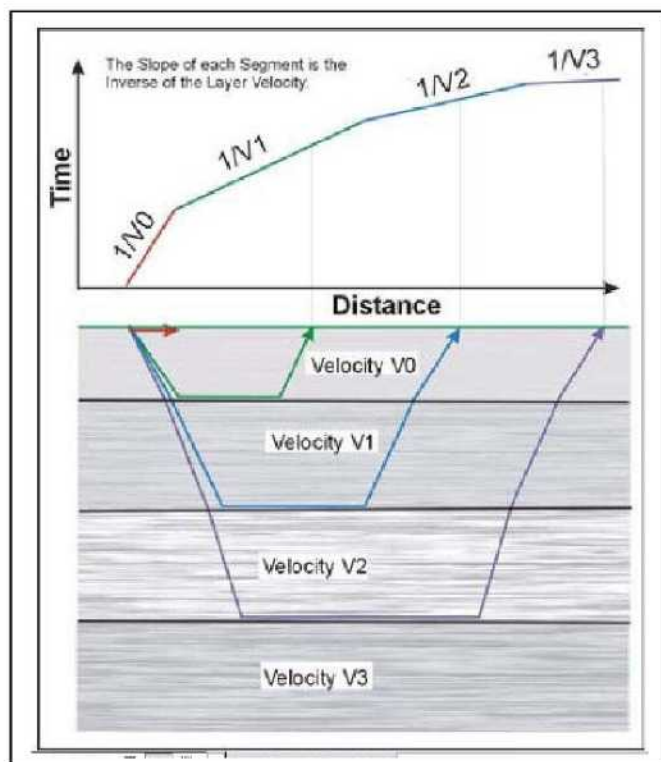


Figure 4-10. Seismic refraction survey (Blackhawk GeoServices, Inc.).

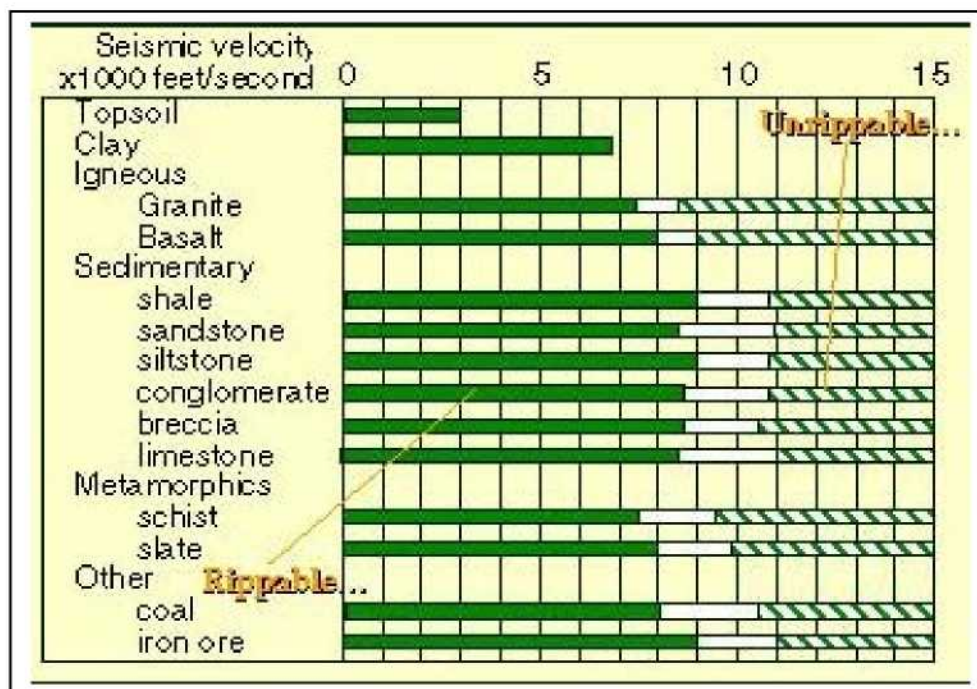


Figure 4-11. Rippability (using a D9 Caterpillar tractor) versus seismic velocity (Caterpillar Handbook of Ripping, 8<sup>th</sup> Edition).



Figure 4-9 shows an example of using a geophysical method (*i.e.*, EM) for locating clay seams on a project in New Mexico. This project demonstrates the ability to correlate the data with soil type and clay content. For this project survey, data were recorded with one of the EM instruments mounted on a trailer constructed primarily from non-conductive materials. The trailer was towed by an All Terrain Vehicle (ATV). A GPS receiver was also mounted on the trailer to provide position information. Data were recorded automatically at half-second intervals with the EM31 and EM38. Recordings with the EM31 were made at two different instrument heights above the ground, giving two different penetration depths. This procedure required several passes along the road. Having obtained conductivity measurements at a number of different depths at each recording location, the data were modeled and provided the interpreted vertical distribution of conductivity with depth. This interpretation is shown in the lower plot in Figure 4-9. The upper plot shows the ground conductivity measured with the EM38 at a depth of 0.75 m (2.5 ft) below the road surface. This data clearly shows the location of clay materials and provides a clear road map for planning additional exploration and sampling (Wightman et al., 2003).

Another example of a project that effectively incorporated geophysical testing into the investigation program has been reported by the Missouri DOT. Geophysical surveys were conducted for the Missouri Department of Transportation (MoDOT) by the Department of Geology and Geophysics at the University of Missouri-Rolla to determine the most probable cause or causes of ongoing subsidence along a distressed section of Interstate 44 in Springfield, Missouri. Ground penetrating radar (GPR) and reflection seismic survey quickly assessed roadway and subsurface conditions with non-destructive, continuous profiles. The GPR proved to be of useful utility in defining upward-propagating voids in embankment fill material. The reflection seismic survey established the presence of reactivated paleosinkholes in the area. These were responsible for swallowing the fill material as water drained through the embankment. On the basis of interpretation of these data, MoDOT personnel were able to drill into the voids that had developed beneath the pavement (as a result of washing out of the fine-grained material of the embankment fill), and to devise an effective grouting plan for stabilization of the roadway (Newton, et al., 1998).

Geophysical data was used to preclude additional subsurface exploration on a rehabilitation project by the Texas DOT. The project consisted of a 16.9 km (10.5 mile) section of road, which was exhibiting substantial alligator cracking and potholes in the southbound lane, as observed in 1999. The project was constructed in 1979 with 152 mm (6 in.) of lime stabilized subgrade where clay subgrade was present, 254 mm (10 in.) of granular base, and a 54-mm (2-in.) ACP surface. ACP level up courses and two open graded friction courses were then placed in 1988 and 1992, respectively. Maintenance forces had subsequently placed several seal coat patches, AC patches, and ACP overlays. FWD and GPR data were taken in the

outside wheelpath at 160 m (0.1 mile) and 3 m (10 ft), respectively. Cores were taken at select locations based on the GPR data analysis. The data indicated that the open graded friction courses were holding water where the maintenance forces had placed the ACP overlays. Cores indicated that the open graded friction course was disintegrating; however, the original ACP layer underneath the friction course was in good shape. FWD data analysis indicated that the base material was structurally in good shape and no base repair and, consequently, no additional subsurface exploration were needed. The overlay and friction course were removed and replaced with a 127 mm (5 in.) ACP overlay in 2001, and was reported to be performing well (Wimsatt, A.J. and Scullion, T., 2003).

#### **4.5.5 In-Situ Testing**

In-situ testing can also be used to supplement soil borings and compliment geophysical results. In-situ geotechnical tests include penetration-type and probing-type methods, in most cases without sampling, to directly obtain the response of the geomaterials under various loading situations and drainage conditions. In-situ methods can be particularly effective when they are used in conjunction with conventional sampling to reduce the cost and the time for field work. These tests provide a host of subsurface information, in addition to developing more refined correlations between conventional sampling, testing, and in-situ soil parameters.

With respect to pavement design, in-situ tests can be used to rapidly evaluate the variability of subgrade support conditions, locate regions that require sampling, identify the location of rock and groundwater with some methods, and, with correlation, provide estimates of design values. Design values should always be confirmed through sampling and testing. Table 4-7 provides a summary of in-situ subsurface exploration tests that have been used for design of pavements and evaluation of pavement construction considerations.

For new pavement design, the most utilized in-situ method is the standard penetration test (SPT); however, the dynamic cone penetrometer (DCP) and/or electronic cone penetrometer test (CPT) (see Figure 4-12) should be given special consideration for pavement design and evaluation (as many agencies are currently doing). DCP and CPT offer a more efficient and rapid method for subgrade characterization and have a significantly greater reliability than SPT, as explained in Table 4-8 on the SPT, Table 4-9 on the DCP, and Table 4-10 on the CPT.

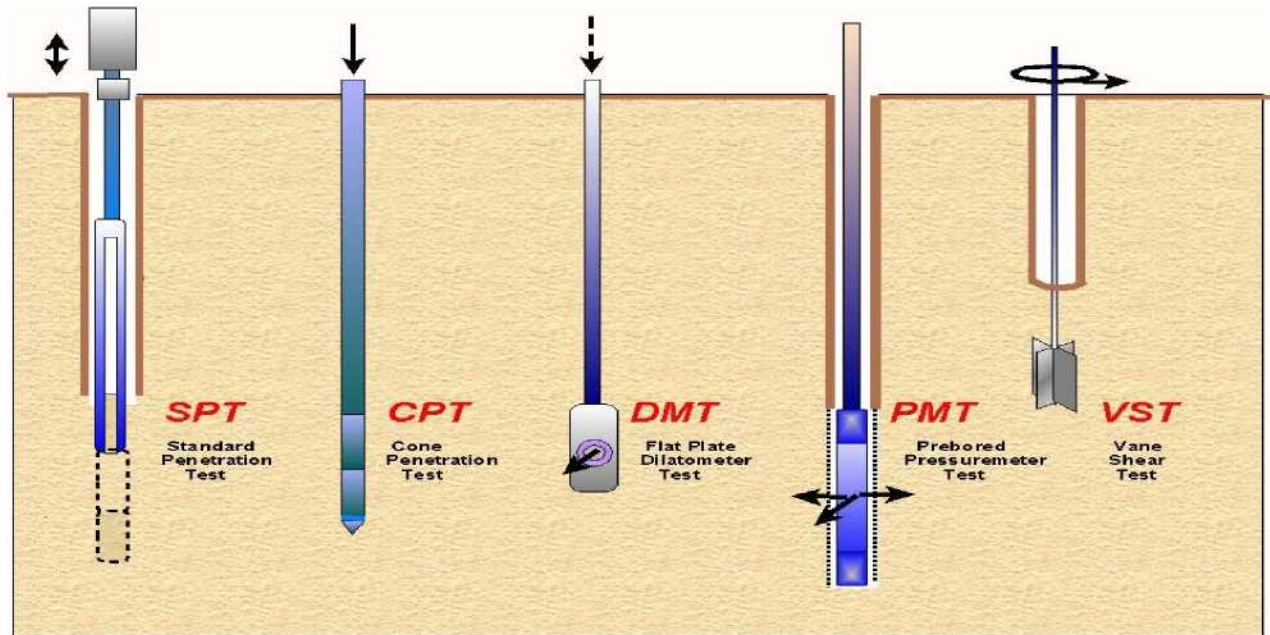


Figure 4-12. Common in-situ tests for pavement evaluation (Mayne et al., 2002).

The CPT and DCP provide information on subsurface soils, without sampling disturbance effects, with data collected continuously on a real-time basis. Stratigraphy and strength characteristics are obtained as the CPT or DCP progresses. Since all measurements are taken during the field operations and there are no laboratory samples to be tested, considerable time and cost savings may be appreciated. DCP is more qualitative than CPT, and is only useful for identifying variation in the upper meter of soil; however, it is performed with low cost, lightweight equipment, with a one- to two-person crew. DCP offers an excellent tool to perform initial exploration through core holes in the surface pavement in rehabilitation projects. Results of DCP tests through the pavement can be compared to test in the shoulders for road widening projects. DCP can also be an effective tool in the construction of pavement to evaluate the suitability of the subgrade after cut, fill, or stabilization operations, as discussed in Chapter 8, and the requirements for stabilization (as discussed in Chapter 7). The CPT provides more quantitative results, can be correlated directly to design properties and types of subgrade, and is useful to greater depths than the DCP in fine grained and sand type soils. Use of CPT and these correlations are detailed in the FHWA Subsurface Investigation Manual (FHWA NHI-01-031).

Other in-situ tests, such as pressuremeter (PM), dilatometer test (DMT) and vane shear test (VST), are also useful in obtaining in-situ design properties, as outline in Table 4-7 but require special skilled personnel and are time intensive. Thus, they are not often used for pavement design. There are also a number of static load tests (*e.g.*, plate load and field CBR) that can be used to assess stiffness and/or strength of the subgrade surface. These tests are

most valuable for reconstruction and rehabilitation projects. Additional information on in-situ testing can be found on the website <http://www.ce.gatech.edu/~geosys/misc/links.htm>.

The relevance of each test also depends on the project type and its requirements. The general applicability of the test method depends in part on the geomaterial types encountered during the site investigation, as shown in Table 4-7.

#### **4.5.6 Borings and Sampling**

The final exploration method includes drilling bore holes or, in some cases, making excavations to obtain samples. This is the most complex and expensive part of the exploration program, and requires a great degree of care. Disturbed and undisturbed samples of the subsurface materials must be obtained for laboratory analyses (and/or tested in the field) to determine their engineering properties and verify geophysical and in-situ exploration results.

Disturbed samples are generally obtained to determine the soil type, gradation, classification, consistency, moisture-density relations (Proctor), CBR, presence of contaminants, stratification, etc. The methods for obtaining disturbed samples vary from hand or mechanical excavation of test pits using truck-mounted augers and other rotary drilling techniques. These samples are considered “disturbed,” since the sampling process modifies their natural structure.

Undisturbed samples are obtained where necessary to determine the in-place stiffness and strength, compressibility (settlement), natural moisture content, unit weight, permeability, discontinuities, fractures, and fissures of subsurface formations. Even though such samples are designated as “undisturbed,” in reality they are disturbed to varying degrees. The degree of disturbance depends on the type of subsurface materials, type and condition of the sampling equipment used, the skill of the drillers, and the storage and transportation methods used. **Serious and costly inaccuracies may be introduced into the design if proper protocol and care is not exercised during recovery, transporting, or storing of the samples.**

Table 4-11 provides a summary of the use and limitation of boring methods using disturbed and undisturbed sampling equipment. Additional information on each of these methods is contained in FHWA NHI-01-031.

**Table 4-7. In-situ tests for subsurface exploration in pavement design and construction.**

Type of Test	Best Suited For	Not Applicable	Properties That Can be Determined for Pavement Design and Construction	Remarks
Standard Penetration Test (SPT)* AASHTO T 206 & ASTM D1586	Sand & Silt	Gravel, questionable results in saturated Silt	Crude estimate of modulus in sand. Disturbed samples for identification and classification. Evaluation of density for classification.	Test best suited for sands. Estimated clay shear strengths are crude & should not be used for design. See Table 4-8 and FHWA NHI-01-031.
Dynamic Cone Test (DCP)* ASTM D6951	Sand, Gravel, & Clay	Clay with varying gravel content	Qualitative correlation to CBR. Identify spatial variation in subgrade soil and stratification.	See Table 4-9 and FHWA TS-78-209.
Static Piezocone Test (CPT)* ASTM D3441	Sand, Silt, Clay		Undrained shear strength and correlation to CBR in clays, density & strength of sand & gravel. Evaluation of subgrade soil type, vertical strata limits, and groundwater level.	Use piezocone for pore pressure data. Tests in clay are reliable only when used in conjunction with other calibration tests (e.g., vane tests). See Table 4-10 and FHWA NHI-01-031.
Field CBR	Sand, Gravel, Silt, Clay	Granular Soils (Lab and field correlations erratic.)	Load-deflection test providing direct evaluation of CBR and can be correlated with subgrade modulus k-value.	Slow, and field moisture may not represent worst-case condition.
Plate Load Test AASHTO T222 & ASTM D1196	Sand, Gravel, Silt, Clay		Subgrade modulus k-value.	Slow and labor intensive.
Vane Shear Test (VST) AASHTO T-223	Clay	Silt, Sand, Gravel	Undrained shear strength, $C_u$ with correlation to CBR.	Test should be used with care, particularly in fissured, varved, & highly plastic clays. See FHWA NHI-01-031.
Permeability Test ASTM D51216 & ASTM D6391	Sand, Gravel	Clay	Evaluation of coefficient of permeability in base and subbase for rehabilitation projects.	Variable head tests in boreholes have limited accuracy. See FHWA NHI-01-031.
Pressuremeter Test (PMT) ASTM D4719	Soft rock, Sand, Silt, Clay		Subgrade modulus k-value & undrained shear strength with correlation to CBR.	Requires highly skilled field personnel. See FHWA IP-89-008 and FHWA NHI-01-031.
Dilatometer Test (DMT)	Sand, Clay		Soil stiffness can be related to subgrade modulus k and compressibility.	Limited database and requires highly skilled field personnel. See FHWA NHI-01-031.

\* These tests can be used in pavement design to qualitatively evaluate subgrade stratification and determine optimum undisturbed sample locations required to obtain design property values.

**Table 4-8. Standard penetration test (SPT).**

Reference	AASHTO T 206 and ASTM D 1586.
Procedures	Standard Penetration Test and Split-Barrel Sampling of Soils.
Purpose	A quick means to evaluate the variability of the subgrade with correlation to density of granular soils and to obtain disturbed samples.
Procedure	<p>The SPT involves the driving of a hollow, thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler a vertical distance of 300 mm (1 ft). A drop weight system is used for the pounding where a 63.5-kg (140-lb) hammer repeatedly falls from 0.76 m (30 in.) to achieve three successive increments of 150-mm (6-in.) each. The first increment is recorded as a “seating,” while the number of blows to advance the second and third increments are summed to give the N-value (“blow count”) or SPT-resistance (reported in blows/0.3 m or blows per foot). If the sampler cannot be driven 450 mm, the number of blows per each 150-mm increment and per each partial increment is recorded on the boring log. For partial increments, the depth of penetration is recorded in addition to the number of blows. In current U.S. practice, three types of drop hammers (donut, safety, and automatic) and four types of drill rods (N, NW, A, and AW) are used in the conduct of the SPT. The test, in fact, is highly dependent upon the equipment used and the operator performing the test. Most important factor is the energy efficiency of the system. The range of energy efficiency for the current US standard of practice varies from 35 – 85% with cathead equipment, and 80 – 100% with automated trip Hammer equipment. A calibration of energy efficiency for a specific drill rig &amp; operator is recommended by ASTM D-4633 using instrumented strain gages and accelerometer measurements in order to better standardize the energy levels. If the efficiency is measured (<math>E_f</math>), then the energy-corrected N-value (adjusted to 60% efficiency) is designated <math>N_{60}</math> and given by</p>

$$N_{60} = (E_f/60) N_{meas} \quad (5-1)$$

The measured N-values should be corrected to  $N_{60}$  for all soils, if possible.

**Commentary** The test can be performed in a wide variety of soil types, as well as weak rocks, yet is not particularly useful in the characterization of gravel deposits nor soft clays. The fact that the test provides both a sample and a number is useful, yet problematic, as one cannot do two things well at the same time. SPT correlations exist with angle of internal friction, undrained shear strength, and modulus. However, the SPT value and these correlations have large scatter, and should not be used alone for design.

For pavement design and construction, SPT provides a measure of subgrade variability. In granular soils, the method provides an evaluation of relative density, which can be correlated to CBR. In addition, disturbed samples are obtained for identification of subgrade materials and for classification tests. SPT results can be used to identify

locations where undisturbed samples should be taken. SPT data can also be compared with FWD results to confirm reasonableness (*i.e.*, low resilient modulus values should

compare to low SPT values). The following relationships have been suggested by Kulhovey and Mayne (1990) as a first order estimate of Young's modulus ( $E/P_a$ ):

$$\begin{aligned} E/P_a &\sim 5 N_{60} && \text{(sands with fines)} \\ E/P_a &\sim 10 N_{60} && \text{(clean, normally consolidated sands)} \\ E/P_a &\sim 15 N_{60} && \text{(clean, over consolidated sands)} \end{aligned}$$

Where,  $P_a$  = atmospheric pressure

Correlations have been attempted for estimating undrained shear strength and correspondingly CBR values in cohesive soils from N values. These relationships are extremely widespread in terms of interpretations, soil types, and testing conditions, such that a universal relationship cannot be advanced. In cohesive subgrades, SPT is better used to evaluate the variability of the subgrade (*e.g.*, based on identification and classification of soil types encountered) and identify locations where proper samples (*e.g.*, undisturbed tube samples for resilient modulus tests or bulk samples for CBR tests) should be taken. Alternatively, drill crews could be instructed to switch to tube samples when cohesive soils are encountered.

#### ADVANTAGES

- Obtain both a sample & a number
- Simple & rugged
- Suitable in many soil types
- Can perform in weak rocks
- Available throughout the U.S.

#### DISADVANTAGES

- Obtain both a sample & a number\*
- Disturbed sample (index tests only)
- Crude number for analysis
- Not applicable in soft clays & silts
- High variability and uncertainty (COV of N =15 to 100%)\*\*

Note: \*Collection simultaneously results in poor quality for both the sample and the number.

\*\*COV, coefficient of variation, as reported by Kulhawy and Mayne, 1990.

**Table 4-9. Dynamic cone penetrometer (DCP).**

Reference	ASTM D 6951.
Procedures	Standard Test Method for Use of the Dynamic Core Penetrometer in Shallow Pavement Applications. The method is also described in FHWA TS-78-209.
Purpose	Another type of test that can be performed in the field to measure the strength of soils in-place, and is being used more commonly for pavement design purposes to estimate the in-place strength of both fine- and coarse-grained soils.
Procedure	The principle behind the DCP is that a direct correlation exists between the strength of a soil and its resistance to penetration by solid objects, such as cones (Newcombe and Birgisson, 1999). The DCP consists of a cone attached to a rod that is driven into soil by means of a drop hammer that slides along the penetrometer shaft. The mass of the hammer can be adjusted to 4.6 and 8 kg (10 and 18 lbs) with the lighter weight applicable for weaker soils. According to NCHRP Synthesis 278 (Newcomb and Birgisson, 1999), more recent versions of the DCP have a cone angle of 60 degrees, with a diameter of 20 mm (0.8 in.).
Commentary	A number of empirical correlations exist to relate the DCP penetration index (DPI) to subgrade strength parameters required for pavement design. The most widely used is (Webster et al., 1994):

$CBR = 292/(DPI)^{1.12}$	for gravel, sand, and silt
$CBR = 1/0.002871 DCP$	for highly plastic clays
$CBR = 1/(0.017 DCP)^2$	for low plasticity clays

The above methods were based on a database of field CBR versus DCP penetration rate values collected for many sites and different soil types, and correlated to test results by others (e.g.,  $\log CBR = 2.61 - 1.26 \log DCP$  as developed by Kleyn, 1975, and currently used by the Illinois DOT). For DCPs with automatic release hammers (e.g., the Israeli automated DCP), CBR values are about 15% greater than the above correlations for manual hammers (after Newcomb and Birgisson, 1999).

ADVANTAGES

- Can be operated by one or two people
- Site access for testing not a problem
- Equipment is simple, rugged, and inexpensive
- Continuous record of soil properties with depth & immediate results
- Can be used in pavement core holes
- Suitable in many soil types & can perform in weak rocks
- Fair reliability (COV ~ 15 – 22 %)\*
- Available throughout the U.S.

DISADVANTAGES

- Does not obtain a sample
- Index tests only
- High variability and uncertainty in gravelly soils
- Limited depth to 1 m (3.3 ft) (however, adequate for most rehab projects and good for rapid surficial characterization).
- Extraction of cone can be difficult.

Note: \*COV, coefficient of variation as reported by Kulhawy and Mayne, 1990.



**Table 4-10. Cone penetrometer test (CPT).**

Reference	ASTM D-3441 (mechanical systems) and ASTM D 5778 (electric and electronic systems).
Procedures	Test Method for Electronic Cone Penetration Testing of Soils.
Purpose	Fast, economical, and provides continuous profiling of geostatigraphy and soil properties evaluation.
Procedure	The test consists of pushing a cylindrical steel probe into the ground at a constant rate of 20 mm/s (0.8 in/s) and measuring the resistance to penetration. The standard penetrometer has a conical tip with 60° angle apex, 35.7-mm (1.4 in.) diameter body (10-cm <sup>2</sup> (1.6-in <sup>2</sup> ) projected area), and 150-cm <sup>2</sup> (23-in <sup>2</sup> ) friction sleeve. The measured point or tip resistance is designated q <sub>c</sub> and the measured side or sleeve resistance is f <sub>s</sub> . The ASTM standard also permits a larger 43.7-mm (1.72-in.) diameter shell (15-cm <sup>2</sup> (2.3-in <sup>2</sup> ) tip and 200-cm <sup>2</sup> (31-in <sup>2</sup> ) sleeve). Piezocones are cone penetrometers with added transducers to measure penetration porewater pressures during the advancement of the probe. Most electric/electronic cones require a cable that is threaded through the rods to connect with the power supply and data acquisition system at the surface. An analog-digital converter and Pentium notebook are sufficient for collecting data at approximate 1-sec intervals. Depths are monitored using either a potentiometer (wire-spoiled LVDT), depth wheel that the cable passes through, or ultrasonic sensor. Systems can be powered by voltage using either generator (AC) or battery (DC), or alternatively run on current. New developments include (1) the use of audio signals to transmit digital data up the rods without a cable and (2) memocone systems where a computer chip in the penetrometer stores the data throughout the sounding.

**Commentary** The CPT can be used in very soft clays to dense sands, yet is not particularly appropriate for gravel or rocky terrain. The pros and cons are listed below. As the test provides more accurate and reliable numbers for analysis, yet no soil sampling, it provides an excellent complement to the more conventional soil test boring with SPT measurements. Figure 4-13 provides a comparison of CPT and SPT logs.

For pavement design, the CPT provides a continuous log of the vertical variability of the subgrade. CPT can be used to identify soil types and soil consistency, which in turn can be used to determine appropriate type(s) and location(s) for sampling. Empirical relations have been developed for undrained shear strength and elastic modulus, as reviewed in FHWA-NHI-01-031 (Mayne et al., 2002). The Louisiana Transportation Research Center in cooperation with FHWA has recently developed a correlation between cone parameters and resilient modulus (Muhammad et al., 2002).

ADVANTAGES

DISADVANTAGES

- |   |   |
|---|---|
| <ul style="list-style-type: none"> <li>- Fast and continuous profiling</li> <li>- Economical and productive</li> <li>- Results not operator-dependent</li> <li>- Strong theoretical basis in interpretation</li> <li>- Particularly suitable for soft soils</li> <li>- Good reliability (COV ~ 7 – 12 %)**</li> </ul> | <ul style="list-style-type: none"> <li>- High capital investment</li> <li>- Requires skilled operator to run</li> <li>- Electronic drift, noise, and calibration</li> <li>- No soil samples are obtained</li> <li>- Unsuitable for gravel or boulder deposits*</li> </ul> |
|---|---|

Note: \*Except where special rigs are provided and/or additional drilling support is available.  
 \*\*COV, coefficient of variation as reported by Kulhawy and Mayne, 1990.

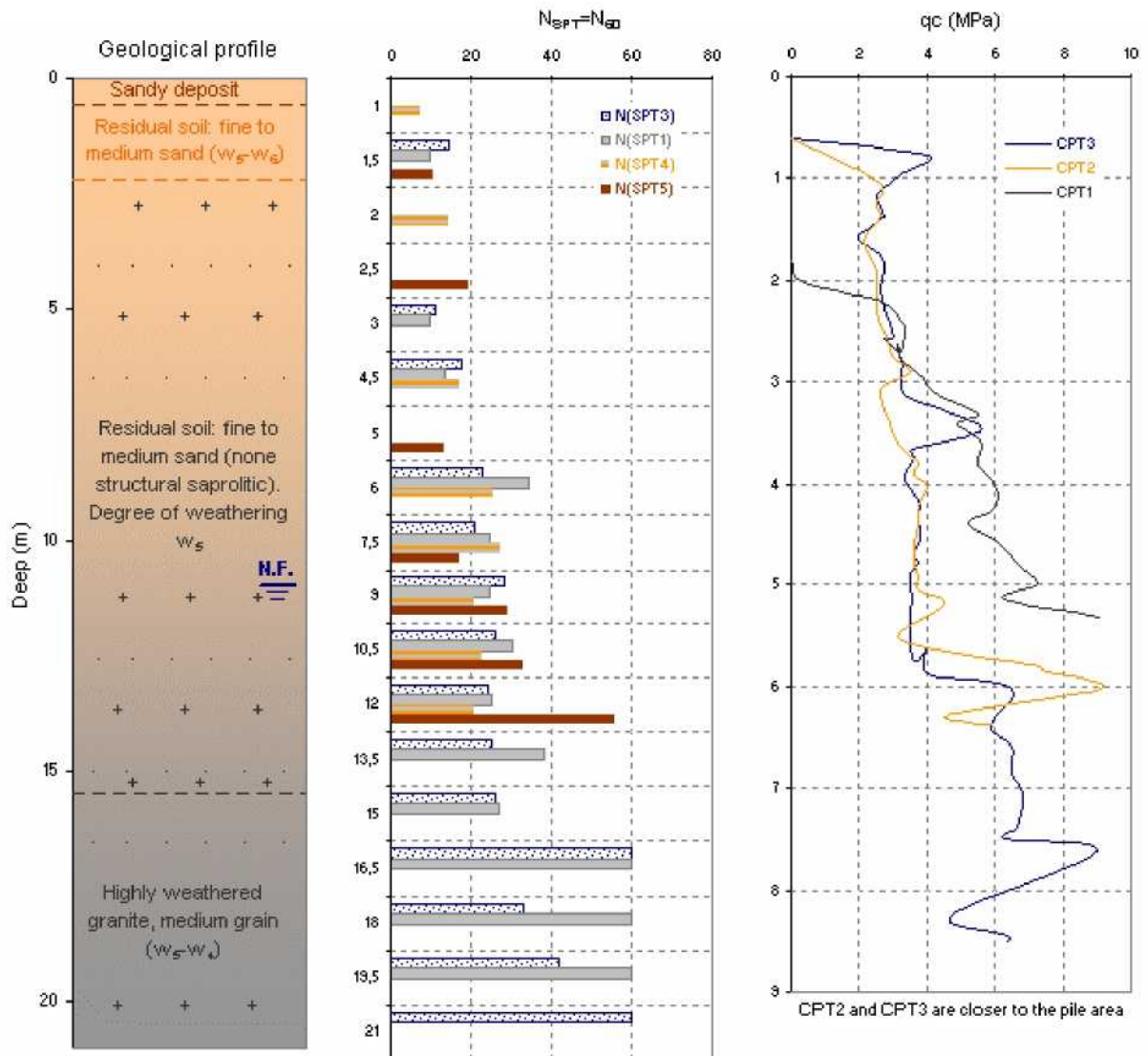


Figure 4-13. CPT log in comparison to SPT data from several locations.

To begin the boring and sampling exploration process, a boring layout and sampling plan should be established to ensure that the vertical and horizontal profile of the different soil conditions can be prepared. A typical design practice for pavements is to assign one subgrade support value to long roadway lengths, *i.e.*, 1 – 16 km (0.6 – 10 mi). This approach may be reasonable for uniform soil deposits, especially considering the construction advantage of maintaining a uniform pavement cross section. However, for highly variable sites, this approach is questionable, as it invariably leads to either an overly conservative design or premature pavement distress in some sections. Significant local variations can best be handled as special design features. There may be more variation of soil properties vertically (drill holes) than horizontally at shallow depths; however, again, only one value is assigned.

Thus, one of the primary sampling issues is how best to sample such that appropriate values can be assigned to long sections of roadway. Two sampling options are available: systematic or representative.

Systematic sampling is a common agency practice. It is done at uniform horizontal and/or vertical intervals. Intermediate locations are sampled when varying conditions are encountered. A large number of samples can be obtained, but the testing may either be on a random basis to obtain an average value for similar materials or a representative basis for variable conditions.

Representative sampling and testing consists of taking samples that are believed to be representative of the typical or conservative soil support values. This type of sampling is based primarily on engineering judgment based on other information about the site (*i.e.*, evaluation of available information, site reconnaissance, remote sensing, and geophysical testing) and involves fewer samples.

**Table 4-11. Subsurface exploration-exploratory boring methods.**

<b>Method</b>	<b>Use</b>	<b>Limitations</b>
Auger Boring ASTM D – 1452	Obtain samples and identify changes in soil texture above water table. Locate groundwater.	Grinds soft particles – stopped by rocks, etc.
Test Boring ASTM D – 1586	Obtain disturbed split spoon samples for soil classification. Identify texture and structures; estimate density or consistency in soil or soft rock using SPT (N).	Poor results in gravel, hard seams.
Thin Wall Tube ASTM D – 1587	Obtain 51 – 86 mm (2 – 3-3/8 in.) diameter undisturbed samples of soft-firm clays and silts for later lab testing ( <i>e.g.</i> , resilient modulus tests).	Cutting edge wrinkled in gravel. Samples lost in very soft clays and silts below water table.
Stationary Piston Sampler	Obtain undisturbed 51 – 86 mm (2 – 3-3/8 in.) diameter samples in very soft clays. Piston set initially at top of tube. After press is completed, any downward movement of the sample creates a partial vacuum, which holds the sample in the tube. In pavement design, these samples can be used for evaluating pavement settlement and/or treatability studies.	Cutting edge wrinkled in gravel.
Pits, Trenches	Visual examination of shallow soil deposits and man-made fill above water table. Disturbed samples for density and CBR tests, or undisturbed block samples for resilient modulus tests, may be extracted.	Caving of walls, groundwater. Requires careful backfill and compaction.

AASHTO 1993 requires the use of average subgrade support values along the alignment, and uses reliability to account for variation in subgrade strength along the alignment. To obtain a true statistical average, random sampling would be appropriate, provided the soil conditions are rather homogeneous. Systematic is not random, but it may be close with respect to averaging. Unfortunately, with systematic, additional borings are often not performed in areas where varying conditions are encountered. So while an average may be achieved, localized conditions along the alignment that could significantly impact performance are often missed. Statistically, the objective is to delineate locations with similar properties (origins and moisture conditions) and assign design values using random methods for the defined population. This is best accomplished by a combination of methods, as outlined in the following subsections.

### ***Frequency (number/spacing) of Borings***

The design engineer should prescribe the spacing and depth of the borings based on an evaluation of available information. As indicated in the previous section, only limited representative borings and sampling are required if geophysical and in-situ testing have been performed. Again, some borings should be performed at several cone locations for calibration and at critical locations identified by the preceding methods. A more extensive program is required in the absence of this alternative exploration information.

The spacing and depth of these borings depend on the variability of the existing soil conditions, both vertically and horizontally, and the type of pavement project. Spacing of borings vary considerably among agencies, on the order of 12 per km (20 per mi) to as few as 2 per km (3 per mi), with spacing generally decreased with high-volume roads and fine-grained soils, as reported by Newcomb and Birgisson, 1999. Considering the variability of soils and the tests used to evaluate geotechnical materials, even the high number appears relatively low. The following provides a review of recommended practice from a geotechnical perspective based on guidelines from textbooks, several state agencies, and the FHWA.

The spacing of borings along the roadway alignment generally should not exceed 60 m (200 ft) for a fully invasive program. Where subsurface conditions are known to be uniform, a minimum spacing of 120 m (400 ft) is generally recommended. In a program supported by geophysical and in-situ tests, such as recommended in Sections 4.5.4 and 4.5.5, a spacing of 150 – 450 m (500 – 1500 ft) as indicated in NCHRP 1-37A may be all that is necessary, depending on the uniformity of site conditions. For new pavement projects, most agencies locate borings along the centerline, unless conditions are anticipated to be variable. Borings should be located to disclose the nature of subsurface materials at the deepest points of cuts, areas of transition from cut to fill, and subgrade areas beneath the highest points of

embankments. The spacing and location of the borings should be selected considering the geologic complexity and soil/rock strata continuity within the project area, with the objective of defining the vertical and horizontal boundaries of distinct soil and rock units within the project limits. It should be noted that **the cost for a few extra borings is insignificant in comparison to the cost of unanticipated field conditions or premature pavement failure.**

The spacing of borings for rehabilitation and reconstruction projects will depend on the condition of the existing pavement, the performance of non-destructive geophysical tests, and the availability of previous subsurface information. As indicated in the NHI (1998) “Techniques for Pavement Rehabilitation” Participants Manual, drilling and sampling is performed on three levels: 1) a high level in the absence of non-destructive geophysical tests, 2) a low level to complement geophysical tests, and 3) at a diagnostic level to evaluate mechanisms of distress where it occurs. In the absence of non-destructive geophysical tests, spacing on the order of one boring every 150 m (500 ft) would appear to be a minimum for pavements with no unusual distressed conditions. Additional borings should be located in problem areas (*e.g.*, areas of rutting or fatigue cracking, which are often associated with subgrade issues) identified in the condition survey as discussed in Sections 4.2.2 and 4.2.3. The number of borings should be increased to the level of new pavement projects when rehabilitation projects include substantial pavement removal and replacement. Again, performance of geophysical tests (*e.g.*, FWD) and/or in-situ tests (*e.g.*, DCP) tests could be used to supplement borings, in which case, sampling at a minimum of every 450 m (1500 ft) may be adequate to complement the geophysical or non-destructive test results, provided there are no areas of significant distress that require special attention. Spacing of borings should be decreased as the variability of the geophysical or in-situ results increase to verify those results via laboratory testing.

For pavement rehabilitation projects, borings should be located in the wheel path to evaluate performance of existing unbound materials, as well as the subgrade. Borings should also be specifically located (and the number increased as required) to investigate the presence of wet or soft subgrade conditions indicated by site reconnaissance and/or maintenance records. If the project involves replacing or rubbilizing the existing pavement, all borings would be drilled through the existing pavement. If the project involves adding a lane, plus replacing or rubbilizing the existing pavement, half the borings should be in the new lane and half in the existing pavement.

As previously indicated in the introduction of Section 4.5.6, borings should be taken to a minimum depth of 1.5 – 2 m (5 – 7 ft) below the proposed pavement subgrade elevation, with at least a few borings taken to 6 m (20 ft) below the grade line. These deeper borings should also be used to determine the water table depth and occurrence of bedrock. Deeper

borings are not generally required for rehabilitation projects, unless the previous section experienced premature failure due to subgrade conditions or there is a change in vertical alignment. All borings should extend through unsuitable foundation strata (for example, unconsolidated fill, highly organic materials, or soft, fine-grained soils) to reach relatively hard or compact materials of suitable bearing capacity to support the pavement system. Borings should extend a minimum of 1.5 m (5 ft) into relatively stiff or dense soils beneath soft deposits. Borings in potentially compressible fine-grained strata of great thickness should extend to a depth where the stress from superimposed traffic loads or a thick embankment is so small (less than 10% of the applied surface stress) that consideration will not significantly influence surface settlement.

Greater depth of borings may be required where deep cuts are to be made, side hill cuts are required, large embankments are to be constructed, or subsurface information indicates the presence of weak (or water-saturated) layers. In those cases, the borings should be deep enough to provide information on any materials that may cause problems with respect to stability, settlement, and drainage. For side hill cuts, additional borings should be performed on the uphill side in uniform soil conditions and on the uphill and downhill side for nonuniform conditions. Additional borings may be required for slope stability considerations and analysis.

Where stiff or compact soils are encountered at the surface and the general character and location of rock are known, borings should extend into sound rock. Where the location and character of rock are unknown or where boulders or irregularly weathered materials are likely to be found, the boring penetration into rock should be increased (NCHRP 1-37A, 2003), as discussed later in this section.

Take sufficient and appropriate auger, split tube, or undisturbed samples of all representative subsoil layers, as discussed in the next section. The soil samples must be properly sealed and stored to prevent moisture loss prior to laboratory testing. Prepare boring logs and soil profiles from this data.

Subsurface investigation programs, regardless of how well they may be planned, must be flexible to adjust to variations in subsurface conditions encountered during drilling. The project engineer should, at all times, be available to confer with the field inspector. On critical projects, the engineer responsible for the exploration program should be present during the field investigation. He/she should also establish communication with the design engineer to discuss unusual field observations and changes to be made in the investigation plans.

### ***Soil Sampling (after NCHRP 1-37A)***

Sampling will vary with the type of pavement project. For new construction projects, a majority of the samples taken will most likely be the disturbed type, such as those obtained by split barrel samplers. This will permit visual identification and classification of the soils encountered, as well as identification by means of grain size, water content, and Atterberg limit tests. In rehabilitation projects, sampling to determine the potential of full depth reclamation or the potential for rubbilization of asphalt pavements is somewhat different, requiring the sampling of the in-place base, subbase, and surface pavement to determine its suitability for reuse and/or rubblizing. The condition survey, as discussed in section 4.2.3, will help in identifying areas requiring sampling and the types of samples required. In general, sampling of the subgrade is not as intensive as is needed for new pavements. Detailed sampling of the base, subbase, and surface pavement will be required to determine if there is a large amount of variability in materials along the project and the condition of those materials for reuse (*e.g.*, base and subbase that has been contaminated with large quantities of fines would not be desirable).

Sampling at each boring location may be either continuous or intermittent. In the former case, samples are obtained throughout the entire length of the hole; in the latter (primarily used in areas of deep cuts), samples are taken about every 1.5 m (5 ft) and at every change in material. Initially, it is preferable to have a few holes with continuous sampling so that all major soil strata present can be identified. Every attempt should be made to obtain 100 percent recovery where conditions warrant. The horizontal and vertical extent of these strata can then be established by intermittent sampling in later borings, if needed.

To obtain a basic knowledge of the engineering properties of the soils that will have an effect on the design, undisturbed samples (such as those obtained with thin-wall samplers or double tube core barrel rock samplers) should be taken, if possible. The actual number taken should be sufficient to obtain information on the shear strength, consolidation characteristics, and resilient modulus of each major soil stratum. Undisturbed samples should comply with the following criteria:

1. The samples should contain no visible distortion of strata, or opening or softening of materials.
2. Specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent.
3. The samples should be taken with a sampler with an area ratio (cross sectional area of sampling tube divided by full area or outside diameter of sampler) less than 15 percent.

At least one representative undisturbed sample should be obtained in cohesive soil strata, in each boring for each 1.5-m (5-ft) depth interval, or just below the planned surface elevation of the subgrade. Recommended procedures for obtaining undisturbed samples are described in AASHTO Standard T207, *Thin-Walled Tube Sampling of Soils*. If undisturbed samples cannot be recovered, disturbed samples should be taken.

All samples (disturbed and undisturbed) and cores should be wrapped or sealed to prevent any moisture loss, placed in protective core boxes, and transported to the laboratory for testing and visual observations. Special care is required for undisturbed tube samples. When additional undisturbed sample borings are taken, the undisturbed samples are sent to a soils laboratory for testing. Drilling personnel should exercise great care in extracting, handling, and transporting these samples to avoid disturbing the natural soil structure. Tubes should only be pressed, not driven with a hammer. The length of press should be 100 – 150 mm (4 – 6 in.) less than the tube length (DO NOT OVERPRESS). A plug composed of a mixture of bees' wax and paraffin should be poured to seal the tube against moisture loss. The void at the upper tube end should be filled with sawdust, and then both ends capped and taped before transport. The most common sources of disturbance are rough, careless handling of the tube (such as dropping the tube samples in the back of a truck and driving 50 km (30 mi) over a bumpy road), or temperature extremes (leaving the tube sample outside in below zero weather or storing in front of a furnace). Proper storage and transport should be done with the tube upright and encased in an insulated box partially filled with sawdust or expanded polystyrene to act as a cushion. Each tube should be physically separated from adjacent tubes, like bottles in a case. A detailed discussion of sample preservation and transportation is presented in ASTM D 4220, *Practice for Preserving and Transporting Soil Samples*, along with a recommended transportation container design.

### ***Rock Sampling***

The need for sampling rock will depend on the location of bedrock with respect to the design subgrade elevation, geology of the region, the availability of geophysical data and local experience. The transition from soil to weathered rock to sound rock can be erratic and highly variable, often causing major geotechnical construction problems (*i.e.*, claims). Rock above the subgrade elevation will need to be removed by ripping or blasting. Considering blasting typically cost 4 to 20 times more than ripping, in addition to the noise and vibration problems associated with blasting, a determination of ripability is an important part of the subsurface exploration program. As previously discussed in Section 4.5.4, ripability can be determined by refraction survey methods, and should be confirmed by coring a sampling of the rock. SPT values have also been used to assess ripability, with values of 80 to 100 typically assumed to be the demarcation between ripping and blasting (Rolling and Rolling). However, there do not appear to be any hard-and-fast rules. The regional geology and the



local ability of the contractor are both significant factors. Considering the determination of ripping versus basting is not an exact science, test pits are recommended to confirm the exploration results.

If the bedrock is near the subgrade level, then the pavement design will dictate requirements for additional samples. Technically pavements can be located directly above competent, intact rock with only a cushion/drainage layer, generally consisting of 150 mm (6 in.) of gravel required between flexible or rigid pavement and the rock. The rock surface should be sloped to promote drainage. It is imperative that the rock surface be level to provide a uniform bearing surface and prevent water from being trapped in local depressions. Undulating rock may therefore require additional excavation, especially if pockets contain poor quality materials, such as frost susceptible soils. For example, Figure 4-14 shows representative excavation requirements where frost susceptible soils exist over undulating rock.

Highly weathered rock and deleterious rock (*i.e.*, rock that degrades easily when exposed to the environment) such as shale, will be required to be removed to a greater depth, on the order of 0.6 – 1 m (2 – 3 ft) based on local experience. In either case, the reason for sampling is to determine the competency of the rock and the amount of excavation required.

It is generally recommended that a minimum 1.5-m (5-ft) length of rock core be obtained to verify that the boring has indeed reached bedrock and not terminated on the surface of a boulder (Mayne et al., 2002). Coring methods and evaluation of rock quality is covered in FHWA NHI-01-031. This rock core depth should be followed if rock is encountered within 1 m (3 ft) of pavement subgrade level, and could be reduced if rock is located at greater depths.

Cores should be used to identify the rock, determine the quality of the rock, and evaluate its durability. Evaluation of durability should be based on a review of past performance, slaking tests and physical degradation tests (Rollins and Rollins, 1996). Many problems with deleterious rocks have been regionally identified across the U.S. Durability tests are reviewed in Chapter 5.

### ***Groundwater***

Observations of the groundwater level and pressure are an important part of geotechnical explorations for pavement design and construction, and the identification of groundwater conditions should receive the same level of care given to soil descriptions and samples. The water level is part of the input in the mechanistic-empirical design approach. Also, as mentioned in Section 4.5.4, the location of the water level will influence interpretation of FWD

and other geophysical results. The water level is also critical to determine the drainage requirements for construction and long-term performance of the pavement. In addition, the water level will influence the selection of appropriate stabilization methods, as discussed in Chapter 7.

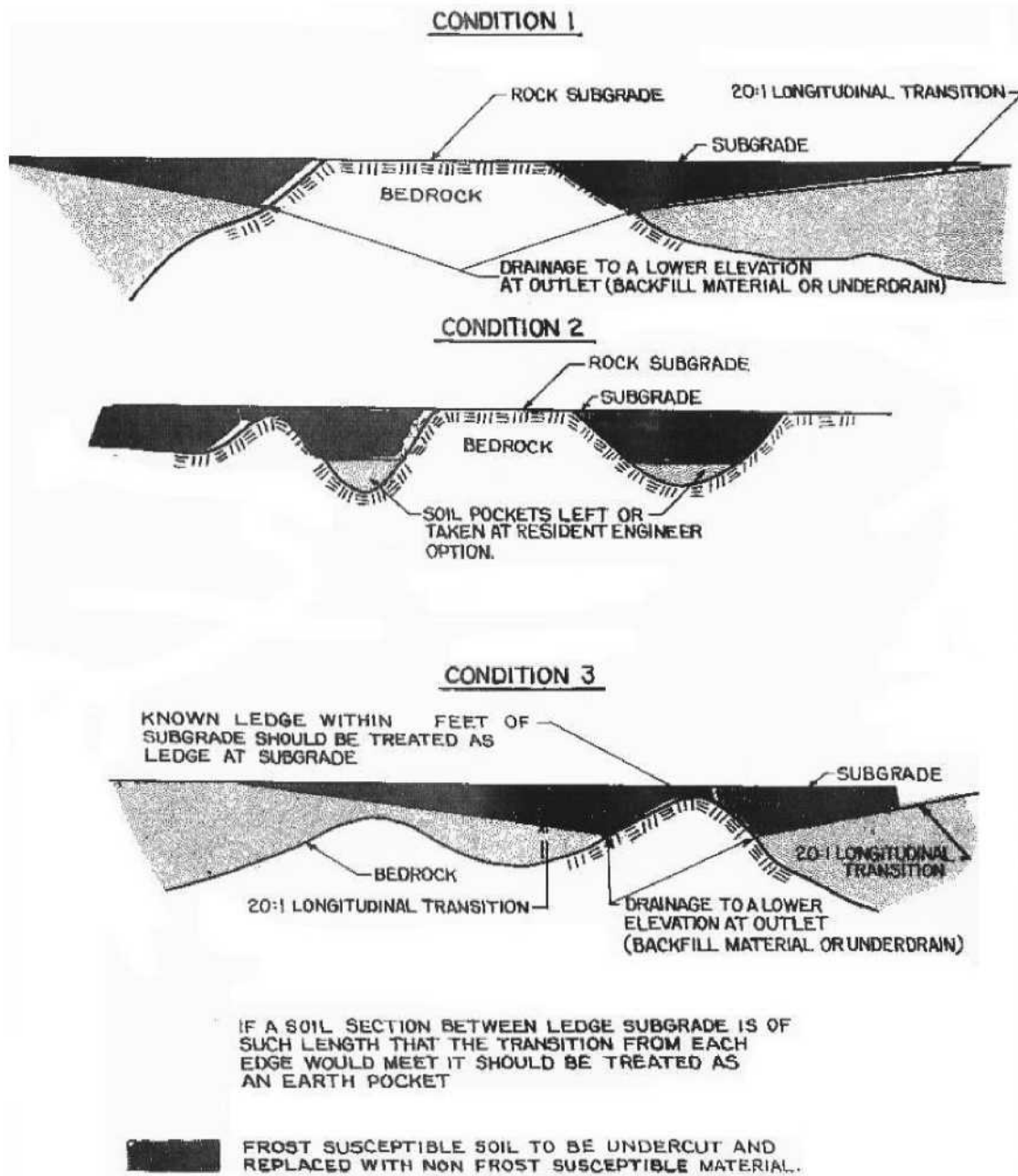


Figure 4-14. Excavation requirements for frost susceptible soils over undulating rock.

Measurements of water entry during drilling, and measurements of the groundwater level at least once following drilling, should be considered a minimum effort to obtain water level data, unless alternate methods, such as installation of observation wells or piezometers, are defined by the geotechnical engineer. Detailed information regarding groundwater observations can be obtained from ASTM D 4750, Standard Test Method For Determining Subsurface Liquid Levels in a Borehole or Monitoring Well and ASTM D 5092, *Design and Installation of Groundwater Wells in Aquifers*.

The water level in the boring is not the only indication of the groundwater level. If the borehole has caved, the depth to the collapsed region should be recorded and reported on the boring record, as this may have been caused by groundwater conditions. The elevations of the caved depths of certain borings may be consistent with groundwater table elevations at the site, and this may become apparent once the subsurface profile is constructed. Drilling mud obscures observations of the groundwater level owing to filter cake action and the higher specific gravity of the drilling mud compared to that of the water. If drilling fluids are used to advance the borings, the drill crew should be instructed to bail and flush the hole prior to making groundwater observations.

Unless the soils are granular with little or no fines (*i.e.*, clay and/or silt size particles), the water level in the boring may take days or weeks to rise to the actual groundwater level. Considering the potential for cave-in and infiltration of surface water during this period and with consideration for the potential for seasonal changes in the groundwater level, a bore hole is usually not the best means to get a true picture of the long-term water conditions at a site. For accurate measures of groundwater, observation wells or piezometers should be installed in the borehole. An “observation well” measures the level in a water table aquifer, while a “piezometer” measures the pressure in a confined aquifer, or at a specific horizon of the geologic profile (Powers, 1992).

The simplest type of observation well is formed by a small-diameter polyvinyl chloride (PVC) pipe set in an open hole. The bottom of the pipe is slotted and capped, and the annular space around the slotted pipe is backfilled with clean sand. The area above the sand is sealed with bentonite, and the remaining annulus is filled with grout, concrete, or soil cuttings. A surface seal, which is sloped away from the pipe, is commonly formed with concrete in order to prevent the entrance of surface water. The top of the pipe should also be capped to prevent the entrance of foreign material; a small vent hole should be placed in the top cap.

Piezometers are available in a number of designs. Commonly used piezometers are of the pneumatic and the vibrating wire type. Interested readers are directed to the reference

manuals of the FHWA NHI course on Geotechnical Instrumentation (FHWA-NHI-98-034), FHWA NHI course on Subsurface Investigation (FHWA-NHI-01-031), or Dunnycliff (1988) for a detailed discussion of the various types of piezometers.

Permeability of the subgrade is rarely an issue for pavement design, but may be of interest in terms of dewatering requirements for excavations or installation of interceptor drains to lower groundwater. For rehabilitation projects, permeability of existing base and subbase may be of interest in order to evaluate drainage characteristics (*e.g.*, time to drain) of in-place materials. Field permeability tests may be conducted on natural soils (and rocks) by a number of methods, including simple falling head, packer (pressurized tests), pumping (drawdown), slug tests (dynamic impulse), and dissipation tests. Simple falling head tests are typically used for evaluating the permeability of in-place base and subbase materials. A brief listing of the field permeability methods is given in Table 4-12.

### ***Test Pits***

Exploration pits and trenches, excavated by hand, a backhoe, or bulldozer, permit detailed examination of the soil and rock conditions at shallow depths and relatively low cost. Exploration pits can be an important part of geotechnical explorations where significant variations in soil conditions occur (vertically and horizontally), large soil and/or non-soil materials exist (boulders, cobbles, debris) that cannot be sampled with conventional methods, or buried features must be identified and/or measured.

The depth of the exploration pit is determined by the exploration requirements, but is typically about 2 – 3 m (6.5 – 10 ft). In areas with high groundwater level, the depth of the pit may be limited by the water table. Exploration pit excavations are generally unsafe and/or uneconomical at depths greater than about 5 m (16 ft), depending on the soil conditions. **The U.S. Department of Labor's Construction Safety and Health Regulations, as well as regulations of any other governing agency, must be reviewed and followed prior to excavation of the exploration pit, particularly in regard to shoring requirements.**

During excavation, the bottom of the pit should be kept relatively level so that each lift represents a uniform horizon of the deposit. At the surface, the excavated material should be placed in an orderly manner adjoining the pit with separate stacks to identify the depth of the material. The sides of the pit should be cleaned by chipping continuously in vertical bands, or by other appropriate methods, so as to expose a clean face of rock or soil. Survey control at exploration pits should be done using optical survey methods to accurately determine the ground surface elevation and plan locations of the exploration pit. Measurements should be taken and recorded documenting the orientation, plan dimensions and depth of the pit, and the depths and the thicknesses of each stratum exposed in the pit. In logging the exploration

pit, a vertical profile should be made parallel with one pit wall. After the pit is logged, the shoring will be removed and the pit may be photographed or video logged at the discretion of the geotechnical engineer. Photographs and/or video logs should be located with reference to project stationing and baseline elevation. A visual scale should be included in each photo or video.

Exploration pits can, generally, be backfilled with the spoils generated during the excavation. The backfilled material should be compacted to avoid excessive settlements. Tampers or rolling equipment may be used to facilitate compaction of the backfill.

### ***Sampling for Fill/Borrow Materials***

Samples are also required to determine the suitability of cut materials to be used as fill and to evaluate suitable borrow sources for additional fill, as required, and for base and subbase materials. Many different soils may be suitable for use in the construction of the roadway embankment or fill. The fill for the subgrade material must be of high quality and, preferably, granular material. Silt and clay type soils are less desirable for subgrade, as they will dictate a thicker pavement section. Bulk samples should be obtained in order to determine the moisture-density relations (Proctor) of each soil type encountered. Moisture-density tests should be used to determine the compaction characteristics for embankment and/or surface soils and untreated pavement materials. AASHTO T99 should be used for medium to high plasticity fine-grained soils, whereas AASHTO T180 should be used for coarse-grained and low plasticity fine-grained soils. The degree of compaction required for the in-place density should be expressed as a percentage of the maximum density from the specified test procedure. Design tests (*e.g.*, resilient modulus, CBR, etc.) are also required on the compacted subgrade material.

### ***Standards and Guidelines***

Field exploration by borings should be guided by local practice, by applicable FHWA and state agency procedures, and by the AASHTO and ASTM standards listed in Table 4-12. Current copies of these standards and manuals should be maintained in the engineer's office for ready reference. The geotechnical engineer and field inspector should be thoroughly familiar with the contents of these documents, and should consult them whenever unusual subsurface situations arise during the field investigation. The standard procedures should always be followed; improvisation of investigative techniques may result in erroneous or misleading results that may have serious consequences on the interpretation of the field data.

**Table 4-12. Frequently-used standards for field investigations.**

<i>Standard</i>		<i>Title</i>
<i>AASHTO</i>	<i>ASTM</i>	
M 146	C 294	Descriptive Nomenclature for Constituents of Natural Mineral Aggregates
T 86	D 420	Guide for Investigating and Sampling Soil and Rock
-	D 1195	Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Airport and Highway Pavements
-	D 1196	Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements
T 203	D 1452	Practice for Soil Investigation and Sampling by Auger Borings
T 206	D 1586	Standard Penetration Test and Split-Barrel Sampling of Soils
T 207	D 1587	Practice for Thin-Walled Tube Sampling of Soils
T 225	D 2113	Practice for Diamond Core Drilling for Site Investigation
M 145	D 2487	Test Method for Classification of Soils for Engineering Purposes
-	D 2488	Practice for Description and Identification of Soils (Visual-Manual Procedure)
T 223	D 2573	Test Method for Field Vane Shear Test in Cohesive Soil
-	D 3385	Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer
-	D 3550	Practice for Ring-Lined Barrel Sampling of Soils
-	D 4220	Practice for Preserving and Transporting Soil Samples
-	D 4428	Test Method for Crosshole Seismic Test
-	D 4544	Practice for Estimating Peat Deposit Thickness
-	D 4694	Test Method for Deflections with a falling-Weight-Type Impulse Load Device
-	D 4700	General Methods of Augering, Drilling, & Site Investigation
-	D 4719	Test Method for Pressuremeter Testing in Soils
-	D 4750	Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)
-	D 5079	Practices for Preserving and Transporting Rock Core Samples
-	D 5092	Design and Installation of Ground Water Monitoring Wells in Aquifers
-	D 5126	Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone
-	D 5777	Guide for Seismic Refraction Method for Subsurface Investigation
-	D 5778	Test Method for Electronic Cone Penetration Testing of Soils
-	D 6391	Field Measurement of Hydraulic Conductivity Limits of Porous Materials Using Two Stages of Infiltration from a Borehole
-	D 6635	Procedures for Flat Plate Dilatometer Testing in Soils
-	D 6951	Test Method for Use of Dynamic Cone Penetrometer in Shallow Pavement Applications
-	G 57	Field Measurement of Soil Resistivity (Wenner Array)

#### 4.5.7 Guidelines for Idealized Subsurface Exploration Program

The ideal exploration program would begin with remote sensing to survey the area for site access issues and to identify geologic formations and other features that would guide the selection and suitability of geophysical test methods. Next, geophysical testing would be performed using FWD as the principle tool, where possible, for back-calculation of resilient modulus values and/or profiling the site, thus, potentially reducing the number of borings required and the cost of laboratory testing. Resistivity would be used in conjunction with FWD to evaluate the extent of significant soil strata, and ground probing radar could be used to provide continuous thickness profiles for the pavement layers, as well as the location of groundwater. CPT or DCP would then be used to classify soil strata, obtain characteristic strength values, and confirm thickness profiles. This would be followed by limited borings and sampling, with some borings performed at several cone locations for calibration/verification, and at critical locations identified by the preceding methods. Again, disturbed samples are generally obtained to determine the soil type, gradation, classification, consistency, moisture-density relations (Proctor), CBR, presence of contaminants, stratification, etc. Undisturbed samples are obtained where necessary to determine the in-place stiffness and strength, compressibility (settlement), natural moisture content, unit weight, permeability, discontinuities, fractures, and fissures of subsurface formations.

The primary reason for following this idealized program is to develop a detailed understanding of subgrade and/or the existing unbound pavement layers that will impact design, construction, and the long-term performance of the pavement structure. There is also a cost implication for this program. Figure 4-15 provides an indication of relative cost for each phase. However, the reduced number of borings and sampling, and the improved reliability of the pavement system, should more than offset the cost of this program.

The Finnish roadway authority has fully integrated this approach into their pavement design. For example, to obtain an initial evaluation of the existing pavement section in rehabilitation and reconstruction projects, they use 1) GPR to provide an evaluation of the thickness of existing pavement components (using air-coupled antenna) and subgrade quality information (using ground-coupled antenna); 2) FWD to obtain the existing roadway support conditions; 3) roughness and rutting measurements; 4) pavement distress mapping; 5) GPS positioning; and reference drilling based on GPR results. The collected road survey data is processed, interpreted, analyzed, and classified, using Road Doctor<sup>TM</sup> software specifically developed for this purpose, as shown in Figure 4-16a. Most recently, they have added resistivity surveys to evaluate moisture content. By combining technologies, they are able to develop a complete map of the subgrade system, including moisture (Figure 4-16b) and corresponding settlement profiles (Dumas et al., 2003). The analysis includes a classification of the critical elements

affecting the lifetime of the road, including 1) overall pavement condition, 2) condition assessment of the unbound pavement structure, 3) road fatigue related to subgrade frost-action, 4) drainage condition, and 5) local damages, such as settlement of the road (Roimela et al, 2000). This information provides a better understanding of the causes of pavement distress and more precise rehabilitation measures for problem layers in the existing pavement system. Similar combinations of technology are used for the evaluation of subgrade conditions for new pavement design. This approach supports Finnish philosophy in pavement design, which presumes that any treatment to the subgrade should last from 60 – 100 years, the base and subbase should last from 30 – 50 years, and the surface should have a life of from 15 – 20 years. This sound philosophy is based on the relative cost of rehabilitation associated with each of these layers, and the importance of engineering in characterizing the soil and selecting material of the lower pavement layers.

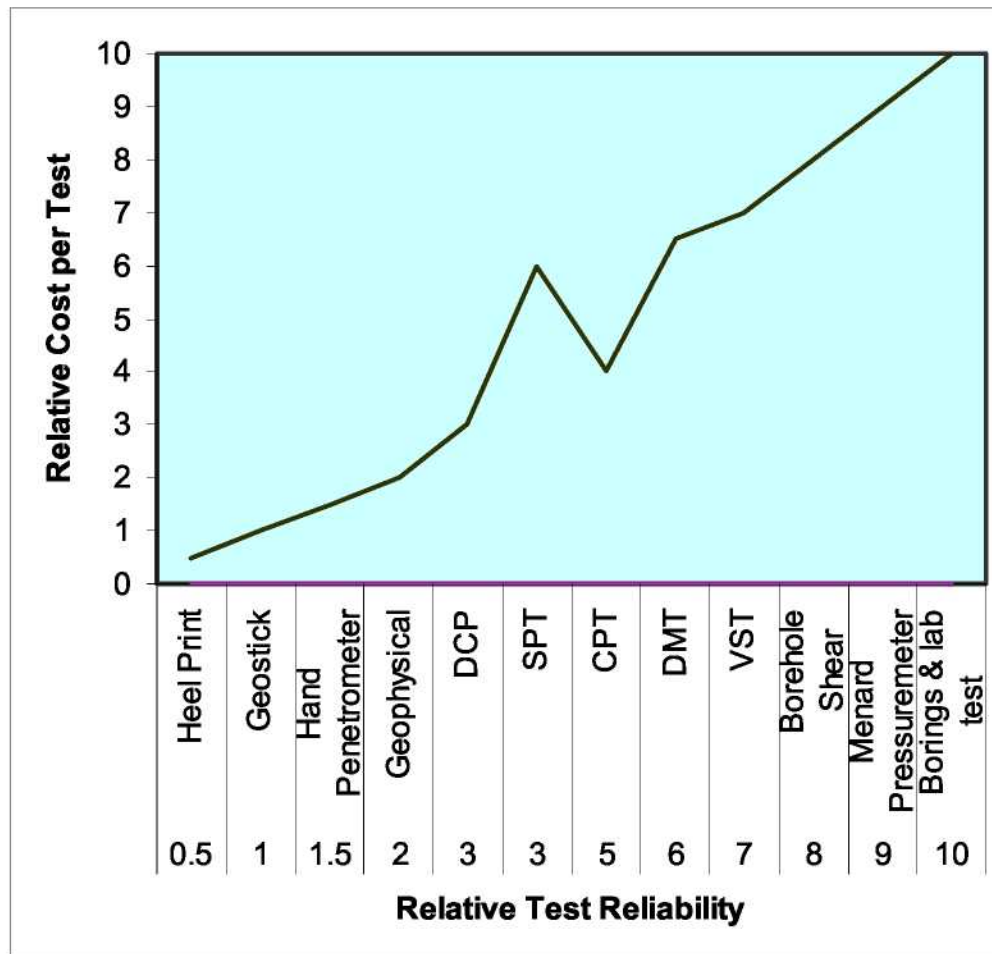
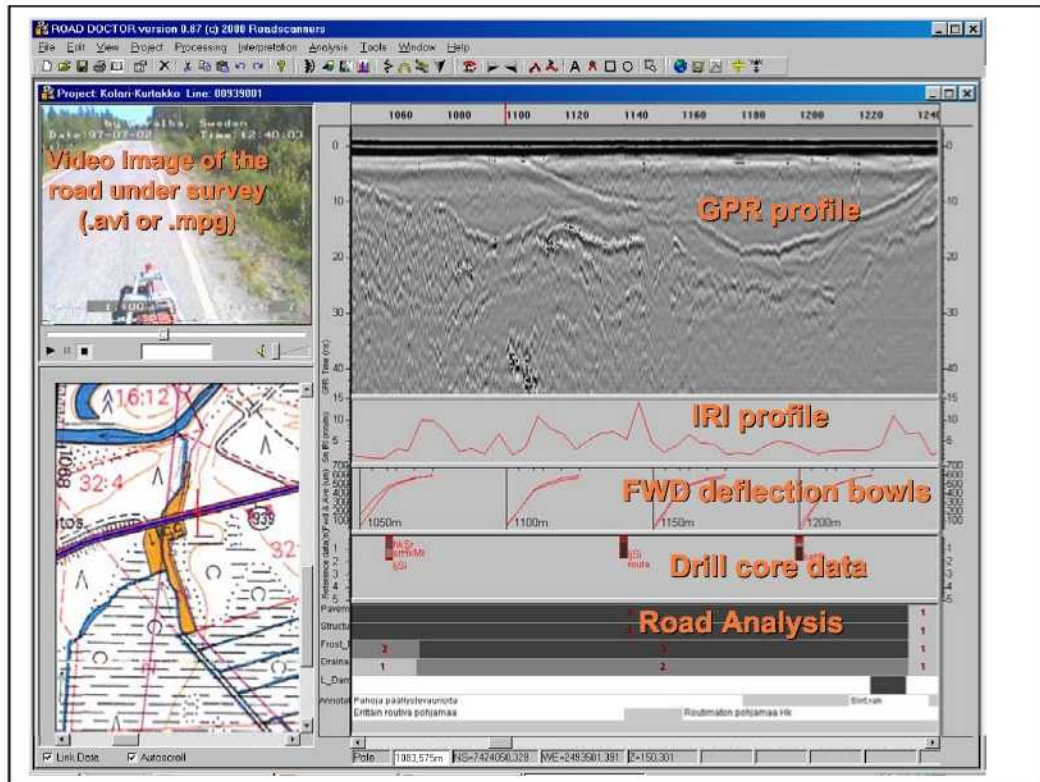
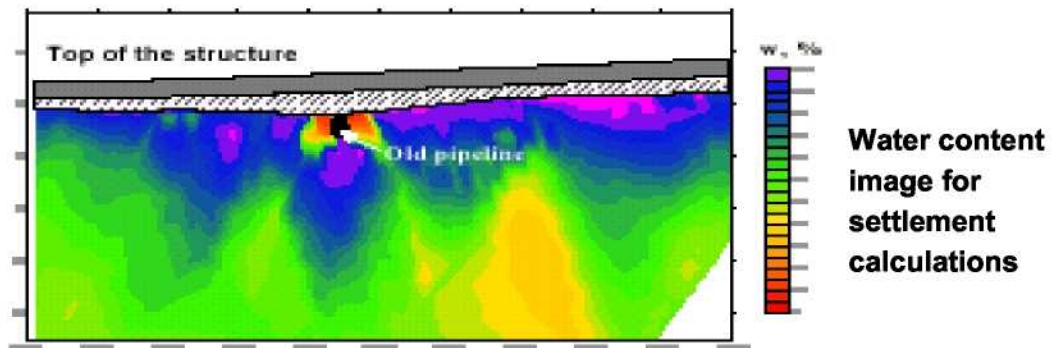


Figure 4-15. Qualitative relationship between relative subsurface exploration cost and reliability (after Handy, 1980).





a)



b)

Figure 4-16. Geophysical evaluation used by the Finnish National Road Administration for rehabilitation and reconstruction projects showing a) results from road analysis and b) moisture profile beneath the pavement (Tolla, 2002).

Texas DOT has recently developed a guideline, which supports the approach of using GPR and FWD data supported by DCP testing in the rehabilitation/reconstruction project evaluation process, as reported by Wimsatt and Scullion (2003). Computer programs have been developed to analyze the GPR and FWD data. GPR data is processed specifically to determine pavement layer thicknesses and the presence of excessive moisture or excessive air voids in pavement layers. FWD data is processed to generate remaining life estimates and pavement and subgrade layer moduli values. The DCP data is then used as required to verify the results of FWD data analysis, such as measuring base, subbase, and stiffness, or determining the depth to a stiff layer. Cores are generally collected at locations based on the GPR results (*e.g.*, in suspect areas).

#### **4.6 IDENTIFY SOURCE FOR OTHER GEOTECHNICAL COMPONENTS**

As indicated in section 4.1, the next subsurface exploration step is to evaluate conceptual designs and determine sources for other geotechnical components (*e.g.*, base and subbase materials). The requirements for subsurface drainage and subgrade stabilization, as well as construction material properties, should also be determined. Sampling of construction materials was discussed briefly in Section 4.5.6. The detailed requirements for these components will be covered in Chapter 7.

#### **4.7 SUBGRADE CHARACTERIZATION**

The last step in the exploration process is to characterize the subgrade through 1) an evaluation of the field data, 2) performance of classification tests to support the field-identified subsurface stratigraphy, 3) develop stratigraphic profiles of the site, and 4) use that information to select representative soil layers for laboratory testing. Evaluation of the field data includes compiling and examining the stratigraphic information from the field investigation steps (*i.e.*, existing information, geophysical results, in-situ tests and borings), and the generation of final boring logs. The final logs are generated using classification tests to establish and support stratigraphy in relation to the design parameters. Soil profiles and plan views along the roadway alignment can then be created and examined to determine resilient modulus or other design testing requirements for each influential soil strata encountered.

### 4.7.1 Boring Logs

The boring log is the basic record of almost every geotechnical exploration and provides a detailed record of the work performed and the findings of the investigation. A boring log is a description of exploration procedures and subsurface conditions encountered during drilling, sampling, and coring. The field log should be written or printed legibly, and should be kept as clean as is practical.

Boring logs provide the basic information for the selection of test specimens. They provide background data on the natural condition of the formation, on the groundwater elevation, appearance of the samples, and the soil or rock stratigraphy at the boring location, as well as areal extent of various deposits or formations. The subsurface conditions observed in the soil samples and drill cuttings or perceived through the performance of the drill rig (for example, rig chatter in gravel, or sampler rebounding on a cobble during driving) should be described in the wide central column on the log labeled “Material Description,” or in the remarks column, if available. The driller's comments are valuable and should be considered as the boring log is prepared. All appropriate portions of the logs should be completed in the field prior to completion of the field exploration. Following is a brief list of items, which should be included in the logs.

- Topographic survey data, including boring location and surface elevation, and bench mark location and datum, if available.
- An accurate record of any deviation in the planned boring locations.
- Identification of the subsoils and bedrock, including density, consistency, color, moisture, structure, geologic origin.
- For rehabilitation and reconstruction projects, an accurate thickness (+/- 2 mm {0.1 in.}) of each existing pavement layer should be carefully documented.
- The depths of the various generalized soil and rock strata encountered.
- Sampler type, depth, penetration, and recovery.
- Sampling resistance in terms of hydraulic pressure or blows per depth of sampler penetration. Size and type of hammer. Height of drop.
- Soil sampling interval and recovery.
- Rock core run numbers, depths/lengths, core recovery, and Rock Quality Designation (RQD).
- Type of drilling operation used to advance and stabilize the hole.
- Comparative resistance to drilling.
- Loss of drilling fluid.

- Water level observations with remarks on possible variations due to tides and river levels.
- The date/time that the borings are started, completed, and of water level measurements.
- Closure of borings.

A wide variety of drilling forms are used by various agencies, with some agencies using computerized logs entered on hand-held computers in the field. The specific forms to be used for a given type of boring will depend on local practice. A typical boring log is presented in Figure 4-17. A key or legend should be established by the agency for use by either in-house or outsource drilling in order to maintain uniformity in boring log preparation. A representative legend for soil boring logs and for core boring logs is included in Appendix D.

In addition to the description of individual samples, the boring log should also describe various strata. The record should include a description of each soil layer, with solid horizontal lines drawn to separate adjacent layers. Soil **description/identification** is the systematic, precise, and complete naming of individual soils in both written and spoken forms (ASTM D-2488, AASHTO M 145). During progression of a boring, the field personnel should only describe the soils encountered. Group symbols associated with classification should not be used in the field. Samples are later returned to the lab where samples may be classified. Soil **classification** is the grouping of the soil with similar engineering properties into a category based on index test results; *e.g.*, group name and symbol (ASTM D-2487, AASHTO M 145). A key part of classification of soil classification is the assignment of group symbols, which should only be assigned after supporting laboratory tests have been performed.

**It is important to distinguish between visual identification (*i.e.*, a general visual evaluation of soil samples in the field) versus classification (*i.e.*, a more precise laboratory evaluation supported by index tests) in order to minimize conflicts between field and final boring logs.** Some agencies have assigned symbols in the field based on visual observation and later corrected them on final boring logs based on lab tests. This practice leads to discrepancies between the field logs and the final logs that have on several occasions been successfully used to support contractor's claims in litigation procedures. In order to avoid these problems, it is recommended that group symbols not be included on field logs, but be reserved only for classification based on lab tests. Some states have avoided these problems by using lower-case symbols for field logs and upper-case symbols for lab-supported classification results, with the lower-case symbols clearly defined on the logs as based on visual observation only.

<b>Project:</b> <b>Project Location:</b> <b>Project Number:</b>	<b>Log of Boring</b> ____ Sheet 1 of ____
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Date(s) Drilled	Logged By	Checked By
Drilling Method	Drill Bit Size/Type	Total Depth Drilled (meters)
Drill Rig Type	Drilled By	Hammer Weight/ Drop (N/m)
Apparent Groundwater Depth ____ m ATD ____ m after ____ hrs ____ m after ____ hrs		Surface Elevation (meters)
Comments		Borehole Backfill
		Elevation Datum

Depth, meters	SAMPLES				MATERIAL DESCRIPTION and other remarks	Elevation, meters	Pocket Pen., kPa	Water Content, %	Liquid Limit	Plasticity Index	Other Tests
	Location	Type	Number	Sampling Resistance							
0											
1											
2											
3											
4											

Template: Proj ID:

Printed:

Figure 4-17. Subsurface exploration log.

The stratigraphic observations should include identification of existing fill, topsoil, and pavement sections. Visual descriptions in the field are often subjected to outdoor elements, which may influence results. It is important to send the soil samples to a laboratory for accurate verification of visual identification, classification tests, and the assignment of appropriate group symbols, as discussed in the next section.

Data from the boring logs are combined with laboratory test results and other field information (*i.e.*, historical logs, soil survey and geological information, geophysical and in-situ tests) to identify subgrade profiles showing the extent and depth of various materials along the roadway alignment. Detailed boring logs, including the results of laboratory tests, are included in the geotechnical investigation report. Guidelines for completion of the boring log forms, preparation of soil descriptions and classifications, and preparation of rock descriptions and classifications are covered in detail in FHWA NHI-01-031, Subsurface Investigation manual.

#### **4.7.2 Soil Classification**

All soils should be taken to the laboratory and classified using the AASHTO (or Unified) soil classification system (see Figure 4-18). As previously indicated, final identification with classification can only be appropriately performed in the laboratory. This will lead to more consistent final boring logs and will avoid conflicts with field descriptions. The Unified Soil Classification System (USCS) Group Name and Symbol (in parenthesis) appropriate for the soil type in accordance with AASHTO M 145, ASTM D 3282, or ASTM D 2487 is the most commonly used system in geotechnical work and, more recently, highway subgrade material. It is covered in detail in this section. The AASHTO classification system has been often used for classification of highway subgrade material, and is shown in comparison with the USCS in Figures 4-18 and 4-19. While both methods are based on grain size and plasticity, USCS groups soils with similar engineering properties.

Table 4-13 provides an outline of the laboratory classification method. Table 4-14 relates the Unified soil classification of a material to the relative value of a material for use in a pavement structure.

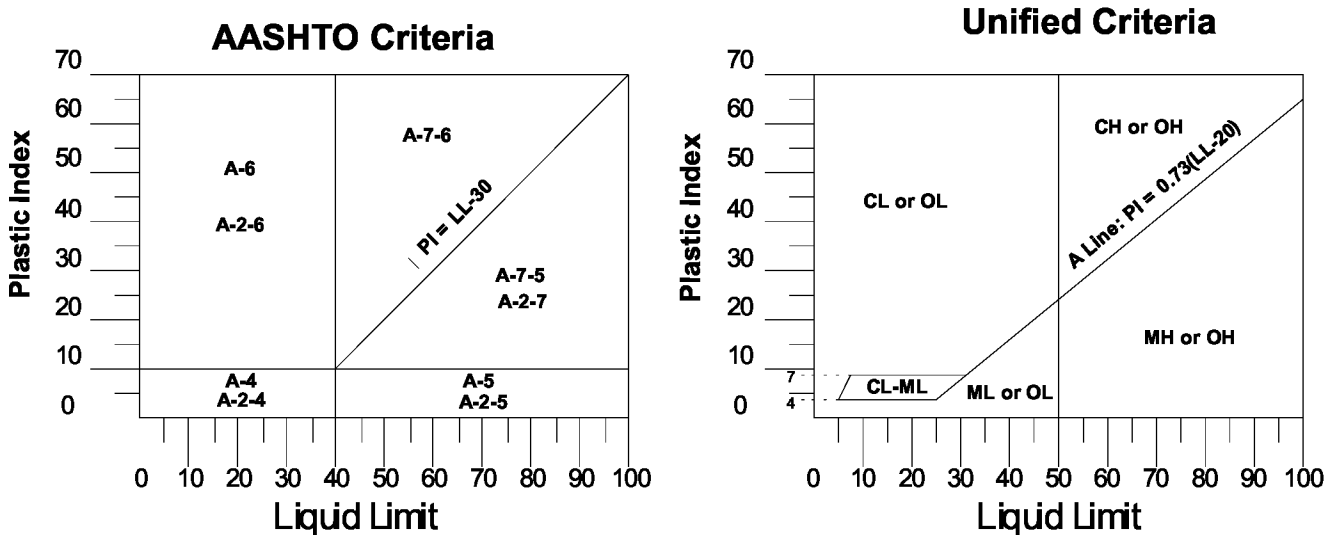
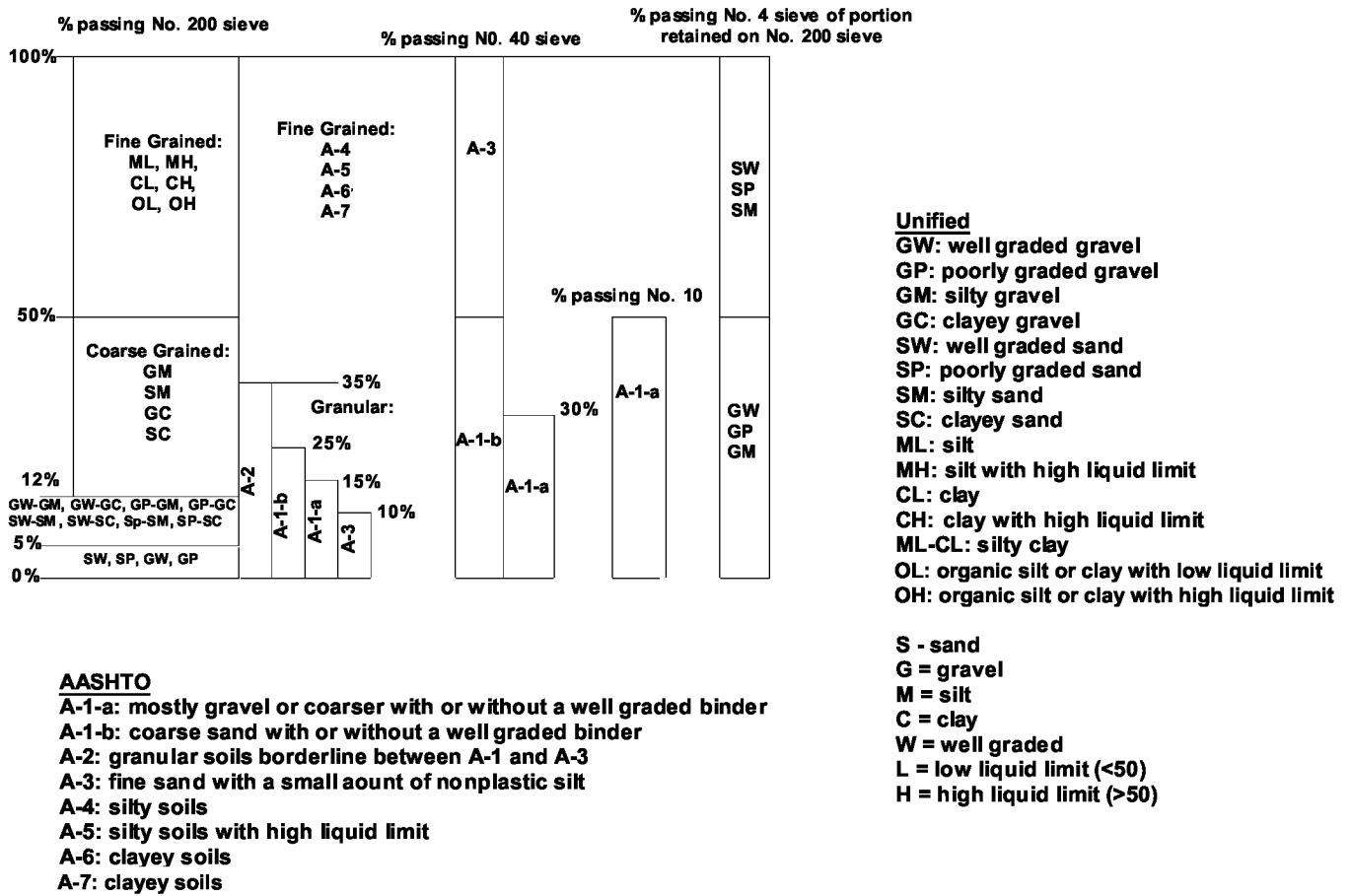


Figure 4-18. The AASHTO and the Unified Soil Classification System (after Utah DOT, 1998).

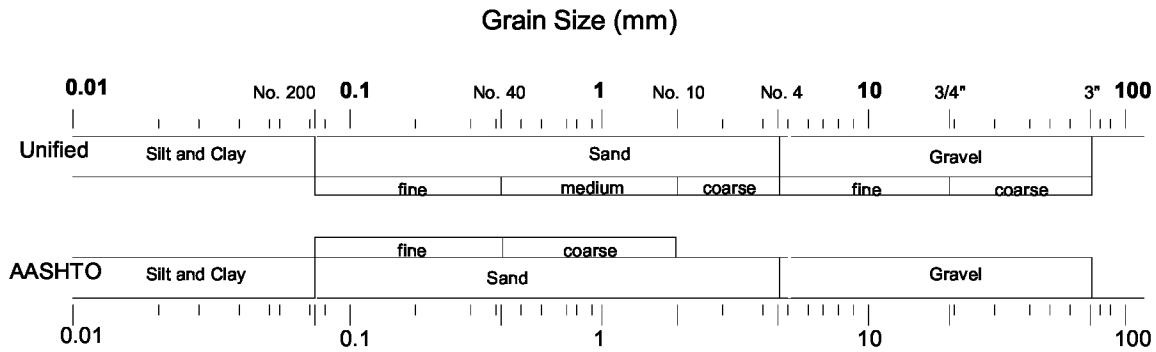


Figure 4-19. Particle size limit by different classifications systems.

**Table 4-13. Classification of soils.**

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests <sup>a</sup>			Soil Classification	
			Group Symbol	Group Name <sup>b</sup>
GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines	$C_U \geq 4$ and $1 \leq C_C \leq 3^c$	GW	Well-graded Gravel
		$C_U \leq 4$ and $1 \geq C_C \geq 3^c$	GP	Poorly-graded Gravel <sup>f</sup>
	GRAVELS WITH FINES More than 12% of fines <sup>c</sup>	Fines classify as ML or MH	GM	Silty Gravel <sup>f,g,h</sup>
		Fines classify as CL or CH	GC	Clayey Gravel <sup>f,g,h</sup>
SANDS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN SANDS Less than 5% fines <sup>d</sup>	$C_U \geq 6$ and $1 \leq C_C \leq 3^e$	SW	Well-graded Sand <sup>i</sup>
		$C_U \leq 6$ and $1 \geq C_C \geq 3^e$	SP	Poorly-graded Sand <sup>i</sup>
	SANDS WITH FINES More than 12% fines <sup>d</sup>	Fines classify as ML or MH	SM	Silty Sand <sup>g,h,i</sup>
		Fines classify as CL or CH	SC	Clayey Sand <sup>g,h,i</sup>
SILTS AND CLAYS Liquid limit less than 50%	Inorganic	$PI > 7$ and plots on or above "A" line <sup>j</sup>	CL	Lean Clay <sup>k,l,m</sup>
		$PI < 4$ or plots below "A" line <sup>j</sup>	ML	Silt <sup>k,l,m</sup>
	Organic	<u>Liquid limit - oven-dried</u> <u>Liquid limit - not dried</u> <0.75	OL	Organic Clay <sup>k,l,m,n</sup>
			OL	Organic Silt <sup>k,l,m,o</sup>
SILTS AND CLAYS Liquid limit more than 50%	Inorganic	PI plots on or above "A" line	CH	Fat Clay <sup>k,l,m</sup>
		PI plots below "A" line	MH	Elastic Silt <sup>k,l,m</sup>
	Organic	<u>Liquid limit - oven-dried</u> <u>Liquid limit - not dried</u> <0.75	OH	Organic Silt <sup>k,l,m,p</sup>
			OH	Organic Silt <sup>k,l,m,q</sup>
Highly fibrous organic soils	Primary organic matter, dark in color, and organic odor	Pt	Pt	Peat and Muskeg



NOTES:

- a. Based on the material passing the 75-mm sieve.
- b. If field sample contained cobbles and/or boulders, add “with cobbles and/or boulders” to group name.
- c. Gravels with 5 – 12% fines require dual symbols:
  - GW-GM well-graded gravel with silt
  - GW-GC well-graded gravel with clay
  - GP-GM poorly graded gravel with silt
  - GP-GC poorly graded gravel with clay
- d. Sands with 5 – 12% fines require dual symbols:
  - SW-SM well-graded sand with silt
  - SW-SC well-graded sand with clay
  - SP-SM poorly graded sand with silt
  - SP-SC poorly graded sand with clay
- e.

$$C_U = \frac{D_{60}}{D_{10}} = \text{Uniformity Coefficient (also UC)}$$

$$C_C = \frac{(D_{30})^2}{(D_{10})(D_{60})} = \text{Coefficient of Curvature}$$

- f. If soil contains  $\geq 15\%$  sand, add “with sand” to group name.
- g. If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.
- h. If fines are organic, add “with organic fines” to group name.
- i. If soil contains  $\geq 15\%$  gravel, add “with gravel” to group name.
- j. If the liquid limit & plasticity index plot in hatched area on plasticity chart, soil is a CL-ML, silty clay.
- k. If soil contains 15 – 29% plus No. 200, add “with sand” or “with gravel”, whichever is predominant.
- l. If soil contains  $\geq 30\%$  plus No. 200, predominantly sand, add “sandy” to group name.
- m. If soil contains  $\geq 30\%$  plus No. 200, predominantly gravel, add “gravelly” to group name.
- n.  $PI \geq 4$  and plots on or above “A” line.
- o.  $PI < 4$  or plots below “A” line.
- p.  $PI$  plots on or above “A” line.
- q.  $PI$  plots below “A” line.

**FINE-GRAINED SOILS** (clays & silts): 50% or more passes the No. 200 sieve

**COARSE-GRAINED SOILS** (sands & gravels): more than 50% retained on No. 200 sieve

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**Table 4-14. Summary of soil characteristics as a pavement material.  
(from NCHRP 1-37A Pavement Design Guide)**

Major Divisions		Name	Subgrade Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
Gravel And Gravelly Soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight	Almost none	Excellent
	GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	None to very slight	Almost none	Excellent
	*d GM --- u	Silty gravels, gravel-sand silt mixtures	Good to excellent	Slight to medium	Very slight	Fair to poor
			Good	Slight to medium	Slight	Poor to practically impervious
	GC	Clayey gravels, gravel-sand-clay mixture	Good	Slight to medium	Slight	Poor to practically impervious
Sand and Sandy Soils	SW	Well-graded sands or gravelly sands, little or no fines	Good	None to very slight	Almost none	Excellent
	SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight	Almost none	Excellent
	*d SM --- u	Silty sands, sand-silt mixtures	Fair to good	Slight to high	Very slight	Fair to poor
			Fair	Slight to high	Slight to medium	Poor to practically impervious
	SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious

\*Division of GM and SM groups is based on Atterberg Limits (See Chapter 6) with suffix d used when L.L. is 28 or less and the PI is 6 or less. The suffix u is used when L.L. is greater than 28.

**Table 4-14. Summary of soil characteristics as a pavement material (continued).**

Major Divisions		Name	Subgrade Strength when Not Subject to Frost Action	Potential Frost Action	Compressibility & Expansion	Drainage Characteristics
Silts & Clays with Liquid Limit Less Than 50	ML	Inorganic silts & very fine sand, rock flour, silty or clayey fine sand or clayey silts with slight plasticity	Poor to Fair	Medium to Very High	Slight to medium	Fair to Poor
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to Fair	Medium to High	Slight to medium	Practically Impervious
	OL	Organic silts & organic silt-clays or low plasticity	Poor	Medium to High	Medium to high	Poor
Silts & Clays with Liquid Limit Greater Than 50	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to Very High	High	Fair to Poor
	CH	Inorganic clays of high plasticity, fat clays	Poor to Fair	Medium to Very High	High	Practically Impervious
	OH	Organic clays of medium to high plasticity, organic silts	Poor to Very Poor	Medium	High	Practically Impervious
Highly Organic Soils	(Pt)	Peat & other highly organic soils	Not Suitable	Slight	Very high	Fair to Poor

### 4.7.3 Subsurface Profile

On the basis of all subsurface information (*i.e.*, from the literature review, geophysical evaluation, in-situ testing, soil borings, and laboratory test data), a subsurface profile can be developed. Longitudinal profiles are typically developed along the roadway alignment, and a limited number of transverse profiles may be included for key locations, such as at major bridge foundations, cut slopes, or high embankments. The subsurface information should also be presented in plan view, providing a map of general trends and changes in subsurface conditions. Vertical and plan view profiles provide an effective means of summarizing pertinent subsurface information and illustrating the relationship of the various investigation sites. By comparing the vertical profiles with the plan view, the subsurface conditions can be related to the site's topography and physiography, providing a sense of lateral distribution over a large horizontal extent. Subsurface profiles should be developed by a geotechnical engineer, as the preparation requires geotechnical judgement and a good understanding of the geologic setting for accurate interpretation of subsurface conditions between the investigation sites.

In developing a two-dimensional subsurface profile, the profile line (typically the roadway centerline) needs to be defined on the base plan, and the relevant borings, projected to this line. Judgment should be exercised in the selection of the borings since projection of the borings, even for short distances, may result in misleading representation of the subsurface conditions in some situations. **Due to the subjective nature of the interpretation required, subsurface profiles and plan views should not be included in either the subsurface investigation report or the construction bid documents.**

The subsurface profile should be presented at a scale appropriate to the depth of the borings, frequency of the borings and soundings, and overall length of the cross-section. Generally, an exaggerated scale of 1(V):10(H) or 1(V):20(H) should be used. A representative example of an interpreted subsurface profile is shown in Figure 4-20, and a plan view profile is shown in Figure 4-21. The subsurface profile can be presented with reasonable accuracy and confidence at the locations of the borings. Generally, however, owners and designers would like the geotechnical engineer 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 becomes questionable. The geotechnical engineer must be very cautious in presenting such data. Such presentations should include clear and simple caveats explaining that the profiles, as presented, cannot be fully relied upon. Should there be a need to provide highly reliable, continuous subsurface profiles, the geotechnical engineer should

increase the frequency of borings and/or utilize geophysical methods to determine the continuity, or the lack of it, of subsurface conditions.

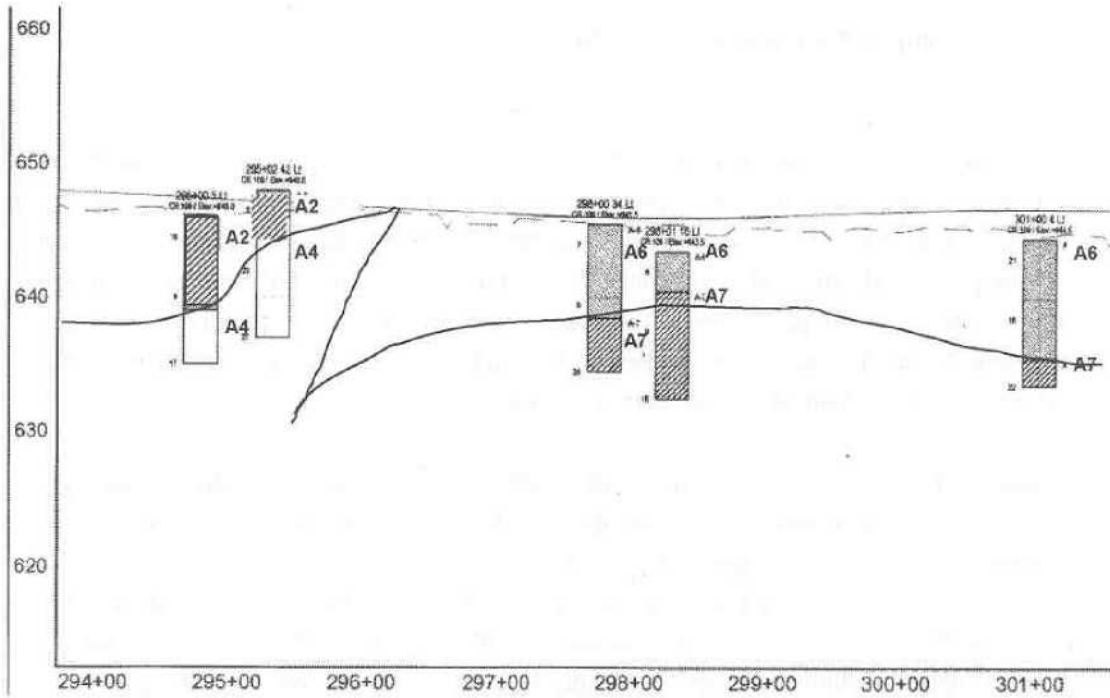


Figure 4-20. Subsurface profile based on boring data showing cross-sectional view.

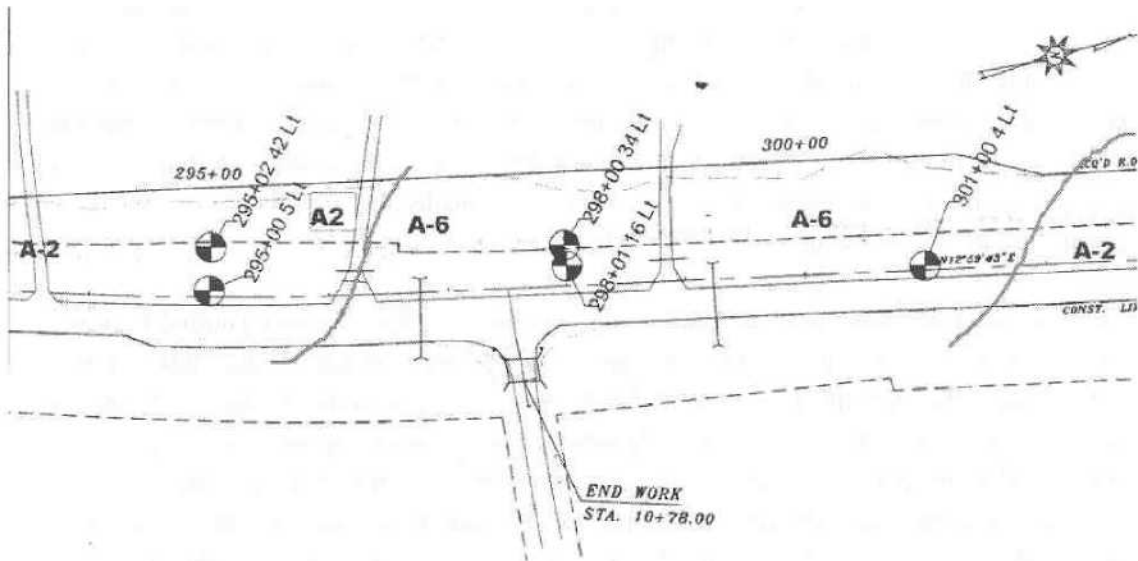


Figure 4-21. Plan view of subsurface information.

#### 4.7.4 Select Samples for Laboratory Testing

A program of laboratory tests will be required on representative samples of the foundation soils or soils to be used as construction materials so that pertinent properties can be determined. The extent of the laboratory program depends on the criticality of the design and on the complexity of the soil conditions. Those laboratory tests and analyses that are typically performed or required for an analysis and selection of the pavement type and thickness are listed in Table 4-15. A deep cut or high embankment, as used in the table, general implies greater than a few meters (6 ft or more).

**Table 4-15. Minimum laboratory testing requirements for pavement designs (NCHRP 1-37A Pavement Design Guide).**

Type of Laboratory Test	Deep Cuts	High Embankments	At-Grade
Moisture Content and Dry Unit Weight	X		X
Atterberg Limits	X	X	X
Gradation		X	X
Shrink Swell	X		X
Permeability	X		
Consolidation		X	
Shearing and Bearing Strength	X	X	X
Resilient Modulus	X	X	X

Representative soil layers are selected for laboratory testing by examining the boring logs, soil profiles, and classification tests. The primary test for design will be either resilient modulus tests, CBR, or other agency-specific design value, as outlined in Chapter 5, along with other properties required for each design level. Where possible, resilient modulus tests should be performed on undisturbed specimens that represent the natural conditions (moisture content and density) of the subgrade. For disturbed or reconstituted specimens, bulk materials should be recompacted to as close to the natural conditions as possible. For rehabilitation projects, the type of distress is also an important consideration, with engineering properties required for structural design of the selected rehabilitation strategy. These tests must also indicate the existing condition of the pavement and highlight any degradation that has taken place during the life of the pavement. Geophysical tests will significantly help in this effort. Tests to evaluate stabilization alternatives typically can be performed on material from disturbed, undisturbed, or bulk samples, prepared and compacted

to the field requirements, as detailed in Chapter 7. Tests will also be required for constructability and performance. These tests can usually be performed on disturbed specimens and/or bulk samples.

The number of test specimens depends on the number of different soils identified from the borings, as well as the condition of those soils. The availability of geophysical and/or in-situ tests will also affect the number and type of tests. Most of the subgrade test specimens should be taken from as close to the top of the subgrade as possible, extending down to a depth of 0.6 m (2 ft) below the planned subgrade elevation. However, some tests should be performed on the soils encountered at a greater depth, especially if those deeper soils are softer or weaker. No guidelines are provided regarding the number of tests, except that all of the major soil types encountered near the surface should be tested with replicates, if possible. Stated simply, resilient modulus tests or other design tests (*e.g.*, CBR) should be performed on any soil type that may have a detrimental impact on pavement performance (NCHRP 1-37A Pavement Design Guide). Other properties, such as shrink/swell and consolidation, will be required for evaluating stabilization requirements and long-term performance (*e.g.*, potential deformation).

For construction, as was discussed in section 4.5.6, moisture-density tests will be required on each soil type that will be used as fill in the pavement section, as well as the roadway embankment. AASHTO T99 should be used for medium to high plasticity fine-grained soils, whereas AASHTO T180 should be used for coarse-grained and low plasticity fine-grained soils. The degree of compaction required for the in-place density should be expressed as a percentage of the maximum density from the specified test procedure. Design tests (*e.g.*, resilient modulus, CBR, etc.) are also required on the compacted subgrade material.

For rehabilitation projects, the number of tests will depend on the condition of the existing pavement. The condition survey as discussed in section 4.2.3, should be analyzed to show where problems may exist and require detailed material property information.

Another important point to remember in selecting the number of specimens to be tested is that the resilient modulus or other design value measured on different soils and soil structures (density, moisture) from repeated load tests can be highly variable. A coefficient of variation exceeding 25 percent for the resilient modulus on similar soils measured at the same stress-state is not uncommon. Repeatability studies indicate that coefficients of variation below 5 percent are not uncommon when testing replicated soil specimens (Boudreau, 2003). The potential high variability in test results requires increased testing frequencies (*i.e.*, many more than two or three resilient modulus tests along a project). As a general guide and suggested testing frequency, three resilient modulus tests should be performed on each major

subgrade soil found along the highway alignment. If the variability of test results (resilient modulus measured at the same stress-state) exceeds a coefficient of variation of 25 percent, then additional resilient modulus tests should be performed to obtain a higher confidence in the data (NCHRP 1-37A).

#### **Student Exercise 4-1.**

Boring logs and a stratigraphic profile from a proposed roadway alignment will be provided and the teams will be asked to 1) determine if the information is adequate, 2) evaluate method(s) for obtaining additional subsurface information, and 3) develop a laboratory testing program. One team will be randomly selected to present the results, followed by a solution discussion with the entire class.

#### **Student Exercise 4-2.**

The students will be asked to provide considerations regarding selection, assignment, and number of laboratory tests. Each item will be noted on a flip chart and, upon completion, reviewed with the list in the Reference Manual. The class will discuss the implications of not running the right test, running too many tests, and incorrectly running tests.

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