



# GEOTECHNICAL POLICIES AND PROCEDURES MANUAL



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## **Introduction**

The purpose of this manual is to provide Geotechnical Engineers guidance to expected procedures when performing geotechnical investigations, analysis, and reporting for the Nebraska Department of Roads (NDOR). This manual is intended to provide general guidelines for the geotechnical duties of a typical project, as each project has unique considerations and requires engineering judgment based on knowledge of an individual situation. Design and construction of roadway projects is complex and involves the contribution of many department units and therefore requires good communication and coordination between the Geotechnical Engineer and other units. This interaction is important in designing a reliable and cost-effective project.

## **Chapter 1**

### **Review of Available Data**

The inherent complexity of projects and varying soil types makes it difficult if not impossible to establish a fixed format for geotechnical investigations within the State of Nebraska. However, there are basic guidelines that should be considered for any project. A review of data available will indicate what information has already been collected and what information will have to be obtained through surface or subsurface investigations at the project site.

#### **1.1 Assessment of Project Requirements.**

The first step in performing any geotechnical investigation is a thorough review of the basic physical and engineering parameters of the proposed project. This review should include the project location, orientation of the project, locations of all structures, loads on structures (as appropriate), delineation of project cut and fill areas and any restrictions on construction activities based upon natural conditions, flora or fauna at the project site. Preliminary plans (also known as plan-in-hand or P.I.H. plans) consisting of a location map, typical section, site layout, drainage plans, plan & profile sheets, 2L sheets (geometric, construction and removal plans) wetland delineation plans, standard cross sections and culvert cross sections may serve as a basis for this review. In addition, geologic cross sections are available upon request for this review.

Specific information concerning the geotechnical aspects of many projects is readily available before venturing into the field for preliminary reconnaissance. The most useful sources of geotechnical data are briefly outlined below.

1.2a Sources of Geotechnical Data.

o **Topographic Maps.**

Topographic maps are prepared by the U.S. Geological Survey (USGS) and are available as a uniform map series covering the entire area of the United States. The best-known USGS topographic maps are the 1:24,000 scale series, also known as the 7.5-minute quadrangles. Topographic maps portray physical features, elevation and relief of the ground surface, some vegetation data, surface water and some man-made features. Topographic maps are commonly used to determine distances, directions and slopes. The Soil Survey Section of Materials and Research Division maintains topographic map coverage of the entire State of Nebraska.

o **Aerial Photographs.**

Aerial photographs are available from various state and federal agencies. Current aerial photographs can be used to gain an up-to-date picture of the area of interest or to supplement maps for current use interpretations. Most man-made features including roadways, buildings, quarries, railroads, and drainage structures are readily visible on aerial photography. Experienced interpreters can determine considerable information concerning soil types and textures using only aerial photographs. Historical aerial photographs such as those archived in the NDOR Roadway Design Vault may be useful in determining the natural topography prior to construction of existing man-made features. Historical aerial photographs may also reveal remnants of previously existing man-made structures, some of which could adversely affect proposed structures.

o **Geologic Maps and Reports.**

Information on geologic formations and structures that lie below the ground surface, including the strike and dip of beds, can be obtained from geologic maps and reports. Geologic maps show the location and relative position of different geologic strata and contain information concerning the characteristics of various layers. This information can be used to evaluate the characteristics of the rock along proposed routes as well to indirectly evaluate soil characteristics, as parent material is one of the factors significantly influencing soil characteristics. Geologic maps and reports can be obtained directly from the United States Geologic Survey (USGS). The Soil Survey Section of Materials and Research Division maintains geologic maps that pertain to the State of Nebraska.



- o **Soil Conservation Service (SCS) and USDA Surveys.**

USDA and SCS soil surveys are compiled by the U.S. Department of Agriculture, usually in the form of county soil maps. SCS Soil surveys show the extent of soil units classified on the basis of the characteristics of different soil horizons and the texture of the surface soil. Soil surveys can provide extensive data on surface soils, including composition, grain size distribution, drainage characteristics, geologic origin, and depth to bedrock. Soil maps are often used in conjunction with geologic maps, as when the two are used together they can provide exceptional clarity concerning soil conditions both at and below the ground surface. The Soil Survey Section of Materials and Research Division maintains those published USDA and SCS soil maps that pertain to the State of Nebraska.

- o **Adjacent Projects.**

Geotechnical data may also be available from nearby NDOR, county, city or federal government projects. Geotechnical data from adjacent projects is most commonly found in the form of boring logs. A boring log is continuous record of the soil or rock types encountered as a shaft is extended downward through subsurface layers. A brief description of the classification of the various soil and rock types encountered as well as changes in rock/soil type and level of water table are considered minimal information. Data such as soil color, consistency, strength and compressibility are included in some boring logs.

Boring logs maintained within Nebraska Department of Roads are stored in three separate locations. The Bridge Foundation Unit stores written boring logs for most bridge foundation location on Nebraska roads, dating from approximately 1927 to the present. The Bridge Foundation Unit also stores pile-driving records associated with pier and bent construction at specific bridges, dating from approximately 1932 to the present. The Soil Survey Unit stores boring data associated with grading operations for some projects, with the earliest records dating from the early 1950's. Older records are not usually as complete as more recent information.

The Materials and Research Division Geotechnical Section recently began storing boring logs in electronic format using in-house boring log software. This information is available on the NDOR intranet at <http://soils/webssel>. The Soil Survey Unit also maintains some borings logs from specific sites in paper format, dating from approximately 1958 to the present.

A final source of geotechnical data may be as-built drawings from adjacent projects. As-built drawings may contain soil conditions and properties encountered during excavation or when creating cut sections. Data of this

type can prove invaluable for identifying problem areas or for establishing preliminary boring locations and depths for subsequent borings. Maintenance records for existing nearby roads and structures may provide insight into surface soil conditions for some proposed projects. As-built plans are available from Road Design and Bridge sections and through the Communication's Records Management Center.

- o **Field Reconnaissance**

After the review of the existing data, the Geotechnical Engineer should visit the project site. This visit will enable the engineer to obtain knowledge of existing field conditions, confirm the observations with the data obtained in the office, and determine the feasibility of boring locations.

## Chapter 2

### Geotechnical Subsurface Investigation Methods

#### 2.1 Administrative Requirements

This section provides project managers, field boring supervisors and consultants with guidance concerning the various requirements for obtaining subsurface data in support of NDOR projects. Requirements contained herein may not be all inclusive, especially when hazardous materials are encountered. General requirements for all borings include:

- o Check to ensure that the drilling equipment is adequately powered and tooled to drill and sample all of the anticipated rock and soil strata. Check to determine if special drilling or sampling procedures will be required.
- o Observe and comply with federal, state and local laws, ordinances and regulations that in any way affect the work being conducted.
- o Obtain all applicable permits and licenses from the appropriate agencies. Notify landowners of any work done on private land.
- o Determine if environmental or archeological clearances are required if there is sufficient evidence to suspect this may be a concern.
- o Contact the Nebraska Digger Hotline at least 48 hours prior to starting any drilling and/or sounding operations. Obtain a list of the underground utility owners or administering organizations contacted by the Digger Hotline and note if any of these organizations have indicated that their underground lines are “clear” of the proposed drilling and/or sounding locations. Provide this information to the field crew so that they can determine if all remaining underground utilities have been marked at the field location. The field crew must positively identify all underground utilities in the immediate area and maintain a safe working distance from buried and overhead utility lines.
- o Avoid clearing and grubbing operations if possible. If clearing and/or grubbing is required, determine the minimum extent of clearing/grubbing to provide access and working space at each boring location.
- o Take reasonable precautions against damage to public and private property. Document damage and promptly repair or make arrangements or pay for any damage in accordance with NDOR administrative procedures.
- o Ensure proper closure of all bore holes, according to applicable laws and regulations of the State of Nebraska and local agencies.

## 2.2 Field Exploration Methods

### 2.2a Hand Equipment/ Shallow Exploration

- o **Hand Probe.** Hand probes are typically used to obtain consistency information in wetland areas and streams where culverts may be proposed. Thickness and lateral extent of soft, compressible soils are determined with small diameter steel rods pushed by hand to firm underlying material.
- o **Soil Recovery Probe.** Soil probes may be used much like hand probes and can be used in more competent material. The open-slot sampler is able to retrieve soil material for classification and consistency identification.
- o **Hand Auger.** A variety of hand augers or post-hole diggers can be used to retrieve disturbed samples of near surface soil materials. A variety of sizes and styles of cutting heads are used depending on material types. Extensions may also be added for greater depth penetration.
- o **Test Pits.** Test pits and trenches may be excavated by hand or with typical excavation equipment such as a back-hoe. Test pits are used to provide details of the shallow sub-surface geology including geological contacts, slip-planes, ground water, and the retrieval of bulk samples.

### 2.2b Mechanically Powered Borings

- o **Continuous Flight Auger.** Flight auger borings are performed by rotating an auger while advancing it into the ground and then withdrawn. Disturbed samples are removed from the auger and the depth of material changes are estimated. This method is used to determine soil strata through material identification and water table elevations. Stand Penetration Tests (SPT), Shelby tube samples, or other in-situ testing may be performed once the auger is removed from the borehole, however it is typically ineffective in loose or soft soils below the water table.
- o **Hollow Stem Auger.** Hollow-stem auger consists of continuous flight auger with a hollow drill stem. The advancement is similar to continuous flight methods except the auger is not removed once sampling depth is made. Cuttings can be retrieved for field identification as the auger advances, again with only an estimate of depth of sample. SPT and Shelby tube sampling are obtained through the hollow drill stem, which holds the borehole open.
- o **Rotary Borings.** Rotary drilling consists of a cased or uncased borehole with rapid rotation and pressure on the drill bit causing cutting and grinding of sediments at the borehole into small cuttings. The cuttings are removed by pumping water, drilling mud, or air through the drill rods to the bottom

of the borehole. Drilling mud or casing is typically used to keep a borehole open in soft or loose soils. Cuttings are observed at the surface for identification and SPT or Shelby tubes can be obtained in an open borehole.

- o **Rock Coring.** A core barrel is advanced through the rock through simultaneous rotation and down-pressure. Circulation of water cools the bit and removes cuttings. The rate of advancement depends on rock materials with the desire to obtain maximum core recovery.

## 2.2c Soundings/ In-Situ Testing

A sounding is a type of exploration where a static or dynamic force is applied to a rod tipped with a testing device with various sensors to penetrate soil. Information obtained is measured and used to correlate with various soil properties.

**Standard Penetration Test (SPT).** The most widely used test in the geotechnical field, this test is a simple test that obtains a sample and data for correlations. The sampling device is 2.5 feet long (0.762 m), with an outside diameter of 2 inches (51 mm) and an inside diameter of 1.375 inches (35 mm). The device consists of a drive shoe, a split barrel and a head, which attaches to drill rods. The SPT is normally conducted at 5-foot (1.5 m) vertical intervals. The sampler is driven 6 inches (152 mm) below the bottom of the hole to insure proper seating. It is then driven two additional 6-inch increments, recording the blow count for each interval in a field log. (Note that a calibrated auto-hammer is required for all SPT tests on NDOR projects). The number of blows required to advance the split-spoon for each of three 6-inch increments is recorded. The blow sum for the last 12 inches of the test is called the N-value (blows per foot). Representative samples of subsurface soils are obtained for purposes of identification, classification, moisture or density testing or to obtain a measure of the relative density of subsurface soils. The results of a SPT test can be correlated with the relative density of granular cohesionless soils and somewhat less accurately with the compressive strength of fine-grained cohesive soils. A correlation of N with the relative density and friction angle of granular soils is shown in Table 1. The sum of the latter two increments is the N value. The SPT is conducted in accordance with AASHTO T 206 or ASTM D 1586.

Table 2 illustrates the correlation of N with the unconfined compressive strength of cohesive soils. Correlations are somewhat less accurate for cohesive soils due to variations in the overconsolidation ratio, moisture content, and fluid pressures below the water table surface.

**Table 1 – Relationship of N Value to Relative Density and Friction Angle for Granular Soils**

<b>N Value</b>	<b>Relative Density</b>	<b>Friction Angle</b>
0 - 4	Very Loose	26 - 30
4 - 10	Loose	28 - 34
10 - 30	Medium Dense	30 - 40
30 - 50	Dense	33 - 45
Over 50	Very Dense	≤50

**Table 2 – Relationship of N Value to Strength and Consistency for Cohesive Soils**

<b>N Value</b>	<b>Consistency</b>	<b>Strength, <math>Q_u</math>, kPa (tsf)</b>
< 2	Very Soft	< 25 (0.25)
2 - 4	Soft	25 - 50 (0.25 - 0.50)
4 - 8	Medium Stiff	50 - 100 (0.50 - 1.0)
8 - 15	Stiff	100 - 200 (1.0 - 2.0)
15 - 30	Very Stiff	200 - 400 (2.0 - 4.0)
> 30	Hard	400 - 800 (4.0 - 8.0)

- o **Cone Penetration Test (CPT).** The CPT is a quasi-static penetration test in which a rod with a conical point is advanced through soil at a constant rate while tip and friction resistance is measured. The resistance readings can be measured at increments or continuously depending on the type of cone being used. If used the CPT data should be presented in both graphical and tabular form in the Geotechnical report showing tip and sleeve resistance, friction ratio, and pore pressure if measured. Cone penetration tests shall be performed according to ASTM D 3441 (mechanical cones) and ASTM D 5778 (electronic friction cones and piezocones).
  
- o **Dilatometer Test (DMT).** The dilatometer is a 3.75-inch wide and 0.55-inch thick stainless steel blade with a thin 2.4-inch diameter expandable metal membrane on one side (see Figure 1). The blade is pushed with the membrane flush to a desired depth. Pneumatic and electrical lines are connected from the membrane to the surface through drill rods. A pressurized gas expands the membrane and the pressure is recorded at pressures of initial membrane movement and at movement of 0.04 inches. Correlations have been developed to estimate several soil properties including material type, void ratio, shear strength and consolidation parameters. Tests shall be performed according to ASTM D 6635.

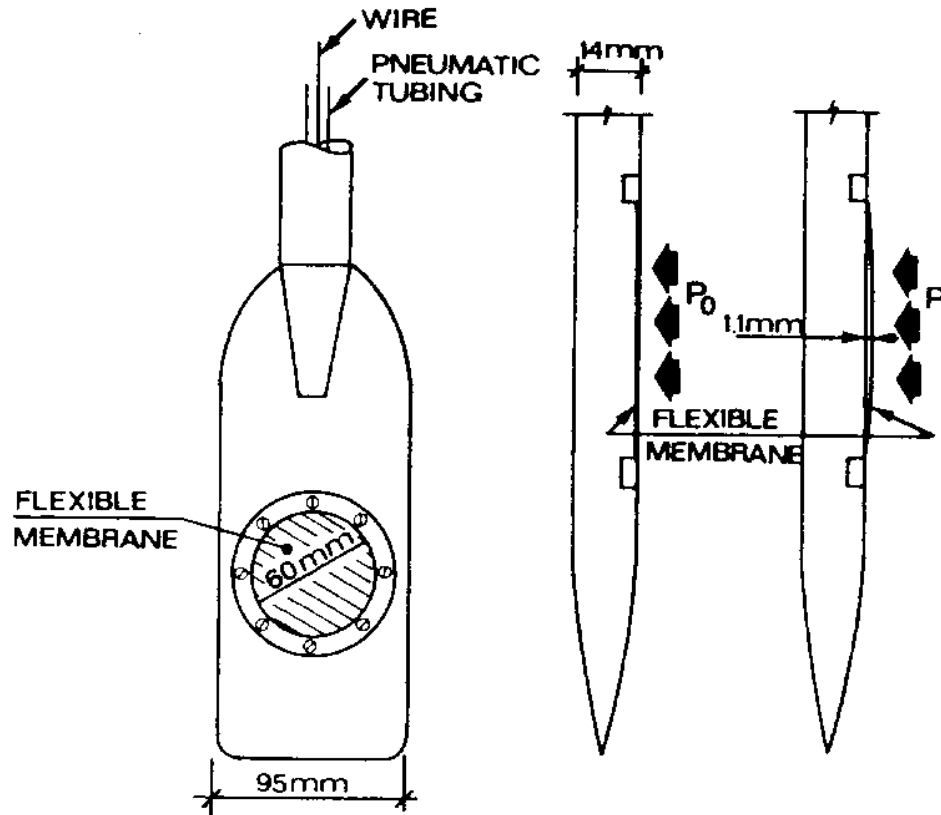
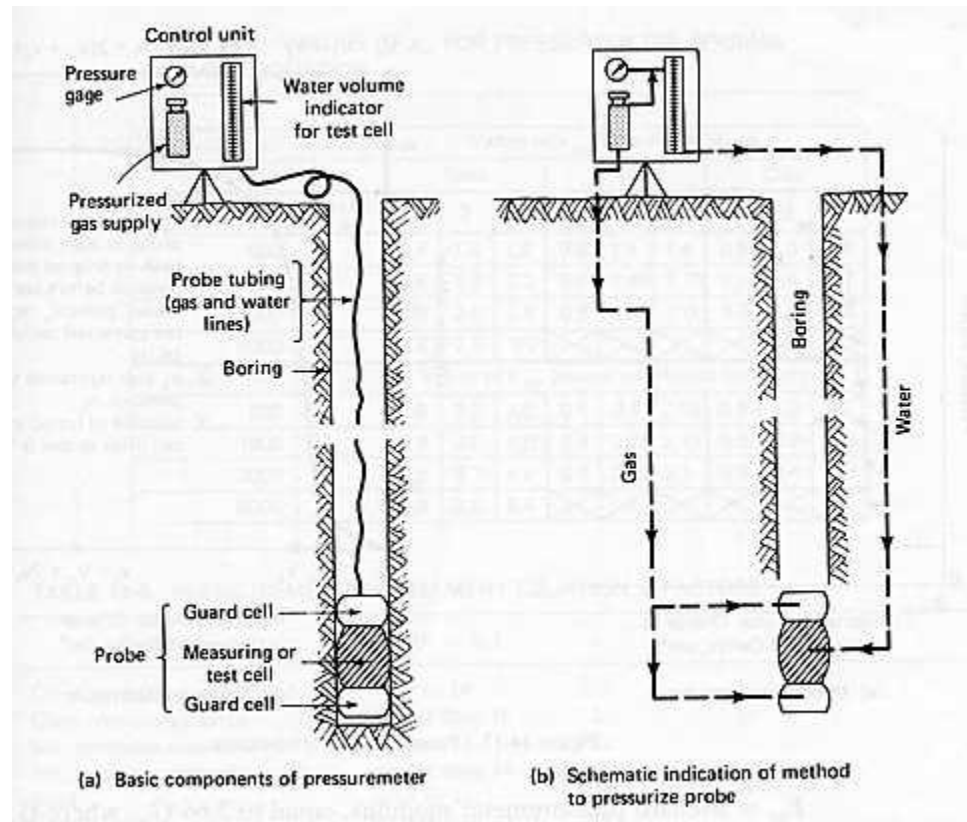


Figure 1. Dilatometer (Marchetti 1980)

- o **Pressuremeter Test (PMT).** The pressuremeter test (PMT) employs a device designed to determine specific in situ properties of subsurface materials. The pressuremeter consists of a cylinder, whose volume can be increasing by expanding in lateral directions only. To run a pressure meter test, the pressuremeter cylinder, called a probe, is lowered to the desired depth in a borehole and its internal pressure is increased, causing the cylinder to expand laterally into the surrounding soil. Pressure is increased in measured increments that are held for a period of time, typically one minute, and resulting changes in the volume are recorded. The test continues until the soil has failed, a condition inferred from a large change in volume resulting from a small increase in internal pressure or when the total expanded volume of the test zone reaches twice the volume of the original cavity. A plot of pressure versus volume is then made to obtain parameters useful for foundation design. Setup for a pressuremeter test is illustrated in Figure 2.



**Figure 2 – Setup for a Pressuremeter Test.**

- o **Field Vane Test.** Soft to medium stiff, saturated, clay soils are easily disturbed by conventional sampling methods, so obtaining an estimate of shear strength of these soils can be difficult. The vane shear test was developed specifically to determine the in situ shear strength of this type of soil. The vane shear test consists of pushing a thin four-bladed vane into undisturbed soil and subsequently rotating the vane to determine the torque required to cause a cylindrical failure surface along the edge of the blades. The torsional force needed to rotate the blades is measured and subsequently converted into shearing resistance acting over the cylindrical surface. After the test on undisturbed soil is completed, a remolded strength can be obtained by turning the vane rapidly through several revolutions and then measuring the torsional force required to shear the remolded soil. The vane shear test has the distinct advantage of causing very little disturbance in the soil before testing. The type of soil being tested is usually unknown until after the vane shear test has been completed and the boring advanced beyond the elevation being tested. The vane shear test, for obvious reason, does not work well in soils that contain pebbles or stones. Soils that drain or dilate during testing will yield inconsistent results. The vane shear test is conducted in accordance with AASHTO T 223.



- o **Quick Shear Tests (Pocket Penetrometer and Torevane).** The pocket penetrometer and Torevane tests represent quick approximations of the unconfined compression test. The first is performed using a hand held penetration device called a pocket penetrometer. The device consists of calibrated spring and 0.25 inch (6.4 mm) diameter piston, both contained within an external metal casing. The test is performed in the field, commonly on split spoon samples or on auger cuttings. When the piston is forced (by hand pressure) to penetrate into a soil sample, the calibrated spring is compressed providing an indication of unconfined compressive strength,  $Q_u$ , on the scale. The values obtained from the pocket penetrometer test are generally not accurate enough for design recommendations. The extremely small area of the piston, the skill of the operator, and the specific point on the sample to which the piston is applied influence the soil strength value obtained during this test. If small pebbles are present in the sample, vastly different strength values may be obtained from the same sample depending upon where the piston is inserted. Several different penetrometer readings should be taken from the same and different specimens and averaged before test results are reported. The pocket penetrometer test provides the most accurate readings when used on soft to medium stiff clays.

### 2.3 Soil Sampling

Common methods of sampling for Nebraska soil types are described below. All samples should be properly preserved, labeled, and transported carefully to the laboratory to ensure the integrity of the samples are maintained according to ASTM D 4220.

- o **Bulk Bag Samples.** These samples are disturbed samples obtained from auger cuttings or test pits. The amount of sample obtained depends on the intended testing to be performed but can be as little as 200 grams to 50 lbs or more. Typically testing on these samples includes moisture content, index tests, and moisture-density relations.
- o **Split-Barrel.** Also known as split-spoon samples from the Standard Penetration Test, the sampler is 2 inches in outer diameter and driven 18 inches. The sample is removed after driving, logged, and placed in an air-tight container for later moisture content and index testing but are considered disturbed samples and should not be used for strength of consolidation testing. Refer to ASTM D 1586.
- o **Shelby Tube.** This sampler is a thin-walled steel tube, typically 3 inches in O.D. and 30 inches in length. The sampler is pushed into the soil in a fairly rapid smooth motion, retracted and sealed on both ends for transportation. Shelby tube samples are relatively undisturbed and are suitable for strength and consolidation tests. Refer to ASTM D 1587 (AASHTO T 207).

## **2.4 Rock Core Sampling.**

Rock cores shall be obtained in accordance to ASTM D 2113 Standard Practice for Diamond Core Drilling for Site Excavation using either a double or triple wall core barrel utilizing diamond or tungsten-carbide tipped bits. There are basically three types of core barrels: Single tube, double tube, and triple tube. Single tube core barrels generally provide poor recovery rates and are not preferred. Refer to ASTM D 5079 for preserving and transporting rock core samples.

- o Double Tube Core Barrel. This core barrel consists of inner and outer tubes equipped with a diamond or tungsten-carbide drill bit. As coring progresses drilling fluid circulates downward between the inner and outer tubes to cool the bit and wash ground up rock to the surface. The inner tube protects the core sample from erosion of the drilling fluid. In a rigid type core barrel both the inner and outer tubes rotate. In a swivel type core barrel the inner tube remains stationary while the outer tube rotates.
- o Triple Tube Core Barrel. This core barrel is similar to the double tube except it has an additional liner that is either a clear plastic solid tube or a thin metal split tube in which the core is retained. This barrel is preferred when coring in fractured and poor quality rock.

## **2.5 Logging Requirements**

Exploratory borings and soundings and in-situ testing are the main resource of subsurface information which describes the geologic constraints pertaining to a specific project and is the basis from which design decisions are made from. Those involved in the subsurface investigation are responsible for obtaining accurate information on which later engineering analysis will rely on. The “Logger” is the term used for the person who records the data in the field.

### **2.5a Duties of the Logger**

- o Acquire accurate subsurface information needed for the proper evaluation of the geology of the project site.
- o Observe, describe, record and evaluate all subsurface information, exploration methods, and other operations performed as part of the investigation.
- o Examine drilling and sampling equipment for defects and that required materials are available.
- o Maintain a production summary of each boring that describes location, elevation, depth, tests, start and finish date.

- o Complete all logs using the established classification and testing criteria. (the more information the better)
- o Ensure proper sampling, preservation, labeling, storage, and transportation methods.
- o Communicate with the geologist or geotechnical engineer.

**2.5b Field Identification of Soils (ASTM D 2488).**

Tentative field identification of soil is based upon basic manual and visual tests. Field identification should only be considered approximate. Field identification of soil should always be confirmed by laboratory testing before this information is used for design.

As soil samples are obtained from borings, test pits, or excavations, each sample should be identified in terms of color, texture and field classification. Boulders, cobbles, and gravels are large enough to allow visual identification. Table 3 shows the size limits for soil particles. Figure 3 is a flow chart that aids in field classification of clayey, silty, sandy and organic soils.

**Table 3 – Size Limits for Soil Particles.**

Description	Size Range	
	mm	U.S. Sieve
Cobble / Boulder	> 76	> 3 in.
Coarse Gravel	19 to 76	0.75 to 3 in.
Fine Gravel	4.75 to 19	No. 4 to 0.75 in.
Coarse Sand	2.00 to 4.75	No. 10 to No. 4
Medium Sand	0.425 to 2.00	No. 40 to No. 10
Fine Sand	0.075 to 0.425	No. 200 to No. 40
Silt	0.002 to 0.075	< No. 200
Clay	< 0.002	-

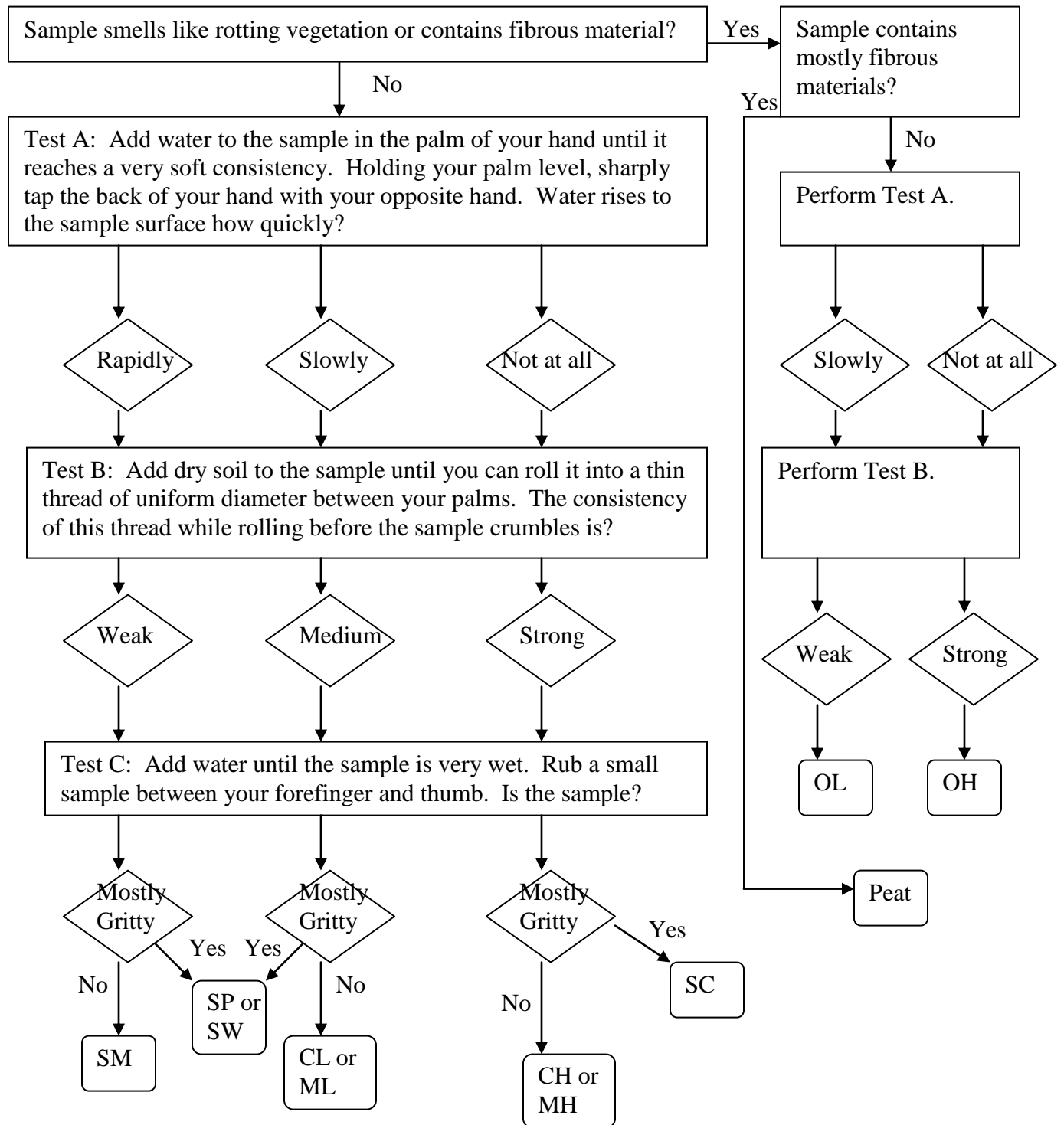


Figure 3 – Flow Chart for Field Identification of Nebraska Soils.

### 2.5c Field Identification of Rock.

Rock is the parent material of soil and is normally more coherent and consolidated than soil. Rock is classified into three broad categories, igneous, sedimentary and metamorphic. Igneous rocks result from volcanism, either at the earth's surface or below. Sedimentary rocks result from the debris of physical and chemical weathering processes being deposited in sedimentary basins, compacted and then uplifted. Metamorphic rocks result from some other type of rock being exposed to temperatures and pressures commonly found inside the earth's crust. Only sedimentary rocks are commonly encountered in Nebraska.

At minimum, field identification of rock should include:

- o Rock type, if possible (shale, sandstone or mudstone)
- o Color (which may change with weathering/moisture)
- o Moisture condition (wet or dry)
- o Grain size and shape (if visible)
- o Texture (stratified, foliated, thin-bedded, massive, etc.)
- o Noticeable weathering or alteration of sample

When core samples of rock are obtained, core recovery and rock quality designation (RQD) should be measured. The core recovery ratio is the length of rock core recovered from a core run, divided by the total length of the core run. The core recovery ratio provides information regarding the presence of weathered zones within the rock mass.

The RQD is the sum of the lengths of all pieces of sound core over 4 inches (100 mm) in length from a core run divided by the length of a core run. To illustrate, if the core run length is 48 inches, and there are 12 rock pieces, 8 of which have lengths less than 4 inches and 4 pieces with lengths of 4.1 inches, 5.0 inches, 5.5 inches and 6.1 inches respectively, the RQD for this rock is  $(4.1 + 5.0 + 5.5 + 6.1)/48 = 43.1\%$ . The length of each piece is an average measured from the midpoints of each end. Several correlations have been developed that relate the RQD with the strength and quality of a rock mass. RQD is related to rock quality as illustrated in Table 4. Table 5 provides a summary of some identifying field characteristics of the principle types of sedimentary rocks found in Nebraska.

**Table 4 – Relationship Between RQD and Rock Quality.**

RQD, %	Rock Quality
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

**Table 5 – Field Characteristics of Nebraska Sedimentary Rocks.**

Type of Rock	Grain Size	Hardness	Breaks Into	Reacts with HCl
Sandstone	Up to 0.25"	Varies	Pieces	No
Siltstone	Fine Powder	Varies	Pieces	No
Shale	Not Visible	Varies	Layers	No
Mudstone or Claystone	Not Visible	Soft to Hard	Pieces	No
Limestone	Not Visible	Hard	Pieces	Rapidly
Dolomite	Not Visible	Hard	Pieces	Slowly

**2.5d Log Format**

- o Project Name, Project # including Control #
- o Client Name (Nebraska Department of Roads)
- o Boring Log #
- o Start and Finish Date
- o Names of Driller and Logger
- o Elevation at top of hole
- o Location of hole, station and offset. County, Reference Post, Township and Range information.
- o Depth of hole and reason for termination.
- o Diameter of hole and any casing used.
- o Description and size of sampler
- o Type of hammer used (i.e. Autohammer)
- o Blow count for each 6 inches of SPT
- o Length of each core run and amount of time to core each run.
- o Length of each core run and amount of core recovered per run.
- o Notes of observations during drilling such as groundwater conditions, hard drilling, fluid circulation loss, obstructions, changes to drilling mud, odor of recovered sample.
- o The more information the better.

## **2.5e Backfilling Borings.**

Recommended procedures for backfilling geotechnical borings contained in the paragraphs that follow pertain to typical situations only. In atypical situations, additional seals or plugs may be required to prevent contamination of adjacent aquifers. AASHTO PP33-96 “Standard Guide for Decommissioning Geotechnical Exploratory Boreholes” and AASHTO R 21-96 “Standard Recommended Practice for Drilling for Subsurface Investigations – Unexpectedly Encountering Suspected Hazardous Material” provide additional details for closing boreholes in atypical situations.

Where no aquifers are encountered during drilling, borings may be backfilled with auger cuttings. Borings in alluvial valleys should be backfilled with an impervious grout seal or a bentonite clay plug. The plug should be emplaced as the casings are extracted from the completed boring. The plug should extend upward from the top of water table elevation a minimum of three feet (one meter). The remainder of the borehole may be backfilled with auger cuttings.

Borings that intersect multiple aquifers should be backfilled with impervious grout seals or bentonite clay plugs as the casing is extracted from the completed borehole. The plugs should extend upward from the top of each aquifer for a minimum of three feet (one meter). The remainder of the borehole may be backfilled with auger cuttings.

Impervious grout seals and bentonite clay plugs are emplaced to prevent surface water or water from shallow water pockets from entering lower elevation aquifers. Seals and/or plugs also prevent migration of water between adjacent vertical aquifers via the borehole.

## Chapter 3

### Subsurface Investigation Guidelines for Highways and Related Structures

A subsurface investigation should be performed at locations of new structures, roadways, widening, extensions, and repair locations as directed by the Geotechnical Engineer. This chapter provides guidance on planning a subsurface investigation for typical roadway construction. The amount and type of information obtained during a subsurface investigation are often limited by time, manpower, equipment, access, or funding. However, an investigation should provide at a minimum, sufficient data for the Geotechnical Engineer to recommend the most efficient design.

#### 3.1 General Requirements

The extent of an exploration varies based on the nature of the project and the complexity of the existing geology. The following standards apply to all investigations for the specific project agreed upon by the Geotechnical Engineer.

- o Preliminary exploration depths should be estimated from the review of available data, the field reconnaissance, and local experience. The borings should penetrate through unsuitable foundation materials (soft clays, loose sands, etc.) and terminate in competent material capable for support of the proposed foundations.
- o Borings for bridges over waterways shall extend a minimum of 30' below estimated scour depth.
- o Each boring, sounding, hand boring, or test pit shall be given a unique identification number for easy identification. Bridge borings shall have the prefix of "BR", Soil Mechanics borings a prefix of "SM", and Soil Survey borings a prefix of "SS".
- o The ground surface elevation and location shall be located accurately for each boring either by survey using GPS or project stationing.
- o A sufficient number of samples shall be collected for each soil layer encountered in order to provide sufficient testing capability.
- o Water table observation within each boring or test pit should be recorded at first encounter and after sufficient time has allowed for the water table to stabilize.



- o Unless used as an observation well, each borehole, sounding, hand boring, and test pit shall be backfilled according AASHTO PP33-96 “Standard Guide for Decommissioning Geotechnical Exploratory Boreholes”.

### **3.2 Minimum Guidelines for Investigations**

The following guidelines represent the minimum extent of exploration and testing expected for most typical projects and shall be adapted to the specific requirements of each project. It should be noted that these guidelines only discuss the use of conventional borings. The Geotechnical Engineer may consider it appropriate to use additional methods such as soundings, geophysical methods, test pits, or in-situ testing to supplement or substitute borings for some of the conventional borings noted in the following sections.

#### **3.3a Soil and Subgrade Survey Borings for Roadways.**

Soil and subgrade surveys are an essential part of a preliminary engineering survey for location and design purposes. Information on the distribution of soils and groundwater conditions must be obtained before a reasonable and economic design can be developed for a highway project. The information contained within these surveys is also useful for construction inspectors, as it provides a method of checking construction practices.

A soil survey is generally conducted prior to final preparation of the grading plans for roads on which the ultimate surface will be rigid or flexible pavement. A soil survey is conducted by drilling a row of holes into the proposed excavation, usually along the centerline or offset along either side of the centerline within the limits of construction. When drilling into rock layers that are not level or when one row of holes will not provide the required information, additional rows of holes may be drilled. Soils are examined visually and by “feel” as they emerge from the auger. A description of each soil and depth of each soil layer change is recorded in the drilling log. In addition, samples are retrieved for laboratory testing. If a water table or wet zone is encountered during the survey, its location and extent are recorded. Additional supplemental borings should be taken to determine the source and extent of the water.

A subgrade survey is generally conducted on previously graded roads for which rigid or flexible pavement is being designed. Reasons for conducting a subgrade survey include: 1) divide the project into sections based upon type of soil in the upper subgrade, 2) identify and locate any problems with moisture in the existing subgrade, 3) provide information for the design of the pavement section. A subgrade survey is similar to a soil survey with the exception that the boreholes for a subgrade survey are on or near the road centerline or within the proposed traffic lanes. Areas showing “frost boil” on the existing road surface are generally bored in detail to determine the cause

and possible methods of mitigating boiling. Requirements for spacing, depth and sampling when conducting soil and subgrade surveys are contained in the following paragraphs.

o **Spacing Requirements.**

Borings should be spaced at intervals of 500 ft (150 meters) or less depending upon degree of variation in soil properties. Boring intervals may be reduced to as little as 25 ft (8 m) in areas where a high water table exists or where a complex subsurface profile exists. The soil surveyor will determine all drilling locations. Sufficient borings will be completed to determine the cause, extent and possible mitigation for wet zones and water tables, as well as other potential problem soils.

o **Depth Requirements.**

Borings should be deep enough to penetrate the major soil types at each location. Normally a depth of 5 feet (1.5 meters) beneath the grade or below the base elevation of the deepest excavation will be sufficient for soil surveys. A depth of 4 ft (1.2 m) will generally be sufficient for subgrade surveys. The soil surveyor will determine if greater depth is necessary at certain locations or for specific projects.

o **Sampling Requirements.**

Sampling and soil testing requirements are primarily dependent upon pavement design. Samples should normally be taken at every change in soil type or once for every five borings. If the soil type in a particular boring is similar to that represented by a sample taken previously, this is indicated in the notes in lieu of taking another sample. Thus the total number of samples submitted for testing is held to a reasonable number. Large samples (60 lbs/35 kg) of each soil type encountered are initially collected for testing. Two smaller samples (20 lbs/10 kg) are collected per linear mile to confirm that the soil type has not changed. Moisture content samples are collected when soils appear to be excessively wet and as significant changes in moisture content are noticed.

**3.3b Borings for Borrow Pits.**

Investigation of borrow pit locations is primarily directed toward the stability properties of soil for use as subgrade material or within embankments. Since excavation and remolding tend to intermix soil units, thin seams of soil within thicker units may not require separate testing. However, suspected deleterious properties of a soil seam of any thickness should be noted in the drilling log as boring progresses.

The number of borings required at a particular location is highly dependent upon the stratigraphy, layout and depth of the borrow site. Borings should be

spaced close enough to accurately determine all soil types and the thickness of each soil unit within the borrow area. Representative samples should be obtained from the proposed borrow area and tested for Atterberg limits, percent silt, percent clay, particle size distribution, in-situ moisture content, group index, USCS soil classification, percent organic material, moisture-density relationship and remolded compressive strength.

### **3.3c Borings for Wetlands.**

Borings for wetlands are performed primarily to determine depth to the ground water table. However, samples should be taken of each soil type encountered and lab tests conducted to determine its engineering properties. NDOR Planning and Project Development Section may specify the pattern and location of boreholes or this may be left to the discretion of field personnel. A geologist or soil surveyor should determine distance between boreholes. Distance between boreholes depends upon variations in the soil or geologic profiles encountered at each project location. In locations where little to no variation in profiles exists, one borehole may suffice for the entire project.

### **3.4a Borings for Soil Mechanics**

Soil Mechanics borings are performed for embankments, retaining walls, culverts, and slope failure locations with an emphasis on settlement, bearing capacity, and slope stability.

#### **o Embankment Spacing Requirements.**

New Roadway Alignments: If embankment height will be greater than or equal to 20 ft (6 m), boring interval should range from 300 ft (90 m) to 1,000 ft (300 m). Typically borings will be situated along the centerline of a single pavement or along the median if the embankment will support multiple pavements. Larger intervals can be used when drilling boreholes for smaller embankments.

Roadway Widening: Boreholes should be located along the shoulders and in the roadway ditch for embankments associated with roadway widening or slope flattening projects. For long, tall embankments with heights greater than 20 ft (6 m) or longer than 500 ft (150 m), the boring interval should range from 300 ft (90 m) to 1,000 ft (300 m). When an embankment will support the entire roadway width, borings should be alternated between sides of the roadway at the same interval.

#### **o Depth Requirements.**

While two-thirds of embankment height is the minimum requirement, borings typically extend 1.5 times the height of the proposed embankment. If a competent material such as dense alluvial gravel, sand, silt or very firm

glacial till is encountered at a depth of less than 1.5 times embankment height, the borehole should extend a minimum of 15 ft (4.6 m) into that layer. If bedrock consisting of sandstone, siltstone, claystone or limestone is encountered at a lesser depth, the boring may be terminated there if that material cannot be drilled with a standard auger equipped with finger bits. Otherwise, the borehole should be continued until it reaches the maximum length of the auger already in the borehole or until the borehole extends a minimum of one foot into the bedrock.

o **Sampling Requirements.**

A geotechnical engineer should determine sampling requirements for each project based on the information already known about the site from previous projects and the type and extent of data required. In general, thin-walled tube samples should be collected in accordance with ASTM D 1587 at 5 ft (1 m) intervals beneath the ground surface. Additional samples should be collected from each borehole at a depth of 2.5 ft (0.75 m) below existing grade for all pavement projects. At locations where a mechanically stabilized earth (MSE) wall will be constructed, additional samples should be obtained from each borehole at a depth of 7.5 ft (2.25 m). A split spoon sampler should be used to collect samples of materials (such as saturated sands) that cannot be collected using thin-walled tube samplers. The split spoon sampler may be either hydraulically pushed or driven as part of the SPT test.

**3.4b Borings for Retaining Walls.**

Typically, two borings per retaining wall location should be made directly beneath the proposed wall face. Additional borings should be considered behind the wall face if the need exists to define the soil profile in the direction transverse to the wall face.

For walls less than or equal to 20 ft (6 m) in height, maximum boring spacing should range from 100 to 200 ft (30 to 60 m). For walls greater than 20 ft (6 m) in height, maximum boring spacing range from 50 to 100 ft (15 to 30 m). At least one boring should be located near the maximum expected height of the retaining wall.

While two-thirds of retaining wall height is a minimum requirement, borings typically extend 1.5 times the height of the proposed wall. If a competent material such as dense alluvial gravel, sand, silt or very firm glacial till is encountered at a depth of less than 1.5 times retaining wall height, the borehole should extend a minimum of 15 ft (4.5 m) into that layer. If bedrock consisting of sandstone, siltstone, claystone or limestone is encountered at a lesser depth, the boring may be terminated there if that material cannot be drilled with a standard auger equipped with finger bits. Otherwise, the borehole should be continued until it reaches the maximum length of the auger already in the borehole or until the borehole extends a

minimum of one foot into the bedrock. The boring depth for sheet piling should be at least twice the minimum exposed wall height.

### **3.4c Borings for Culverts.**

#### **o Borings for Concrete Box Culverts.**

A concrete box culvert relies on the soil beneath its base to support its weight and to provide structural stability. Because most box culverts are located in stream or riverbeds, subsurface deposits at proposed box culvert locations often consist of alluvial materials that may not have sufficient stability to adequately support the proposed structure. At least one boring or other type of subsurface investigation (SPT, CPT, etc.) is recommended at each proposed box culvert location where the height of embankment will be in excess of 12 ft (3.5 m) above stream channel level or 10 ft (3 m) above the top of the culvert. The information collected will enable a geotechnical engineer to anticipate subsurface conditions and recommend prudent subgrade improvement.

#### **o Borings for Pipe Culverts.**

NDOR currently does not require any subsurface investigation prior to installation of pipe culverts. Pipe culverts are similar to box culverts, except pipe culverts are generally smaller, round versus rectangular in shape, and are commonly precast versus cast-in-place. Their smaller size, round shape and precast construction make pipe culverts much less susceptible to problems resulting from poor soil conditions than traditional box culverts. At least one type of subsurface investigation (boring, SPT, CPT, etc.) is recommended at each proposed location where problems associated with differential settlement are anticipated. If surface soils are found to be unsuitable at a proposed location, the subsurface investigation will provide information that will enable a geotechnical engineer to recommend a suitable method of subgrade improvement.

### **3.5a Borings for Bridge Foundations**

A single boring at the location of a proposed structure will cost less than a single pile, but the knowledge obtained from that single boring might allow elimination of all piles beneath a structure. Without boring data, the design engineer is unable to utilize his knowledge or experience to design a safe but economical foundation. He must instead use an extremely conservative design characterized by a high factor of safety, which is always more expensive.

If general knowledge of local subsurface conditions is available from geological studies, earlier investigations or records from nearby existing

structures, the scope of a foundation investigation may be detailed in advance. Otherwise, the extent of work is normally established as the investigation proceeds. The number, depth, spacing and specific tests required in a subsurface investigation are so dependent upon the type of structure and specific site conditions that no general rules are applicable in all situations.

- o **Spacing Requirements**

A minimum of one boring is commonly required for each structural abutment or pier, and at the end of any wingwall that measures over 30 ft (9 m) in length. The pattern should be staggered so that borings are at the opposite ends of adjacent footings. Piers or abutments over 100 feet (30 m) in length require one boring at the extremities of each abutment. For spread footing designs on sloping rock surfaces, additional borings are recommended.

- o **Boring Depths**

Boring depths must consider the most likely foundation type for the bridge based on the existing geology and design loads. The depth of boring required can be estimated from earlier investigations, from adjacent projects, or from specified boring resistance data such as “The borings for structural foundations shall be terminated when a minimum resistance criteria of 20 blows per foot on the sample spoon has been achieved for 20 feet of drilling”. The minimum resistance criteria is commonly modified depending upon the foundation capacity required at the site.

- o **Sampling Requirements**

Split spoon samples are normally obtained at 5-foot (1.5 m) intervals or when changes in material are encountered. Continuous split spoon samples are recommended for the top 15-foot (5 m) when the footings will be placed on natural soils. Split spoon samples are generally “disturbed” when obtained and thus are not suitable for laboratory determination of strength or consolidation parameters. Undisturbed Shelby tube samples should be obtained at 5-foot (1.5 m) intervals when working with cohesive soils. For cohesive soils greater than 30-foot (10 m) in depth, Shelby tube sample intervals can be increased to 10 feet (3 m). In soft clay soils, in-situ vane shear strength tests are recommended at 5 to 10-foot (1.5-3.0 m) intervals. Split spoon samples must be carefully sealed in plastic bags and placed in jars before being sent to the laboratory for analysis. Shelby tube samples must be sealed and stored upright in a shockproof container for transportation to the laboratory.

Standard penetration test (SPT) data should be recorded for each boring in accordance with ASTM D 1586 and placed in the drill log. The drill crew

should also continuously perform a rough visual analysis of soil samples and record their observations in the drill log.

Cone Penetration Test (CPT) soundings may be performed in accordance with ASTM D 3441-94 to supplement borings to help identify the depth and thicknesses of hard and soft layers, sampling selecting, and provide information between borings.

The water level in each borehole should be recorded along with data on when the observation was made. Artesian pressure can be measured by extending the drill casing above ground level until flow stops. An erroneous indication of water level may result when water is used as a drilling fluid and adequate time is not allowed after hole completion for the water level to stabilize. In clay soils, one week or more is required before an accurate reading can be obtained.

To avoid confusion, a unique number should be used to identify each borehole on a project. One solution to avoid duplication is to designate that all boreholes for bridge piers or abutments begin with the letter “BR”, and a sequential number from a series of numbers assigned to that specific project. For example, the first borehole on a bridge project might be designated BR-100. Drill holes for embankments begin with the letters “SM” while drill holes for cut sections could begin with the letter “SS”.

The guidelines listed in previous paragraphs will provide minimum data on the soil types, their relative density and the position of the groundwater table required by the design engineer to create a safe and economical foundation. Extremely soft or otherwise unusual soil conditions may require testing in addition to what has been specified above.

**3.6a Borings for Other Structures**

**3.6b Borings for Buildings.**

The number of borings and spacing between borings for a building project is directly related to the type and size of the planned structure along with the associated live and dead loads. Variations in soil conditions will affect the extent to which the design engineer feels comfortable interpolating subsurface conditions between borings. Demands of municipal building codes and the funds available for the boring program may also affect the number of borings completed for a building.

Most building projects are unique to some degree, so it is difficult to establish a set of rules which will answer all of the designer’s or contractor’s questions under all circumstances. A minimum of two borings or a combination of one boring and one subsurface test (SPT, CPT, shear vane test, etc.) should be taken at the proposed site of any building. Larger buildings will require more data. Building corners are typically selected as borehole/subsurface test

locations. Borehole/subsurface test spacing should not exceed 200 feet. For buildings with critical components requiring small settlement tolerances or high load capacity or where the subsurface conditions are extremely variable, boring/subsurface test spacing should be reduced accordingly. Borings /subsurface test locations should be selected to investigate known or suspected special conditions, such as filled-in basements, covered drainage pathways or historic dump sites.

Consideration should be given to performing a preliminary investigation to obtain information about general subsurface conditions. From the information obtained during the preliminary investigation, a final subsurface exploration program that answers most questions can be planned.

Borings/subsurface test depths will vary according to the type of soil present at the project location. For cohesive soils, test holes should extend to a depth where loads imposed on the soil surface have dissipated to approximately ten percent of the surface value. This depth is approximately three times the spread footing width below the base of the footing. Test holes should not be terminated in cohesive soils where the consistency is less than medium stiff (unconfined compressive strength is less than 0.5 tsf).

In granular soils, boreholes should extend to a depth at least three times the footing width below the base of the footing, or 1.5 times the height of emplaced fill, whichever is greater. When boreholes extend through stratified layers of both cohesive and granular materials, depth should be determined by the more stringent of the above criteria. If bedrock is encountered before the above criteria are met, the borehole can be stopped at that point.

A geotechnical or soil mechanics engineer should provide the driller with an estimate of the type and depth of materials expected. The driller should contact the engineer if significant differences are encountered. Additional depth, additional sampling frequency or additional boreholes may be required.

Sampling frequency is dependent upon the type of subsurface testing being performed in conjunction with the drilling program. More sophisticated subsurface testing (i.e. electronic CPT testing) may allow for significant reduction in the number of samples versus would be required without testing. Sampling frequency (with no additional subsurface testing) should be no greater than 2.5 feet of depth, with samples taken in cohesive soils using thin-wall tubes while SPT samples are collected for granular soils. Samples should be taken to a minimum depth corresponding to the footing width or to a depth at least five feet below the base of the footing whichever is greater. If the borehole extends beyond this depth, sample frequency can be reduced to one sample for every five feet of borehole.



### **3.6c Borings for Traffic Control Structures.**

Responsibility commonly rests with the contractor to investigate soil conditions, emplace the foundation for and erect traffic control structures. The major concern is to have adequate foundation depth to resist the overturning moment resulting from wind loads acting near the opposite end of the structure. If the structure is a single support cantilever design, rotational forces resulting from the weight of the structure itself must also be considered.

Foundation designers often complete an initial design based upon assumed minimum soil strength. If soil strength is questionable, a split-spoon or Shelby tube boring can be made at the proposed location to obtain a soil sample for testing. The test will either verify the assumed minimum soil strength or provide the foundation designer with additional data that can be used to modify the design.

### **3.6d Borings for Light Poles.**

Light poles are similar to traffic control structures, except that a section of the pole generally serves as the foundation for the length of pole extending above the ground surface. The length of pole beneath the soil surface must be sufficient to resist overturning moment resulting from wind loads near the top of the structure. Depth of embedment is generally fairly constant based upon experience or extended practice. Few light poles fail from insufficient depth of embedment.

Light pole failure generally results from soils having insufficient shear strength to resist lateral wind forces. In some instances, the wind can exert sufficient lateral force to move the pole from a nearly vertical orientation to a more severely inclined orientation. In locations near where this situation has occurred in the past, one or more subsurface borings will provide data necessary for a geotechnical engineer to provide suggestions on how to alleviate the problem.

## Chapter 4

### Laboratory Testing

Laboratory testing is a vital part of the geotechnical investigation and must be done correctly and according to AASHTO and ASTM procedures. Consequently all firms performing geotechnical laboratory testing for NDOR must be certified by AMRL or equivalent agency. An efficient and accurate lab testing program should provide the engineer with sufficient information to complete an effective and economical design.

This chapter briefly describes typical laboratory tests, their purpose, and the use of the data obtained from the tests. Not every test is described may be applicable to every project. Engineering judgment must be used to set up a testing program to provide the information required for each individual project.

#### 4.1 Soil Classification Systems (AASHTO and USCS).

##### o AASHTO M145 Soil Classification System.

The primary purpose of a soil classification system is to allow construction personnel to recognize and utilize specific types of soil under field conditions. The most widely recognized system of soil classification associated with roadways was devised by the Public Roads Administration (now the Federal Highway Administration) for classification of subgrade soils. In this system, known as the AASHTO M145 standard, soils are classified into one of seven groups, designated A-1 through A-7, according to their general load carrying capacity. The AASHTO M145 classification standard for highway subgrade materials is illustrated in Figure 4.

An AASHTO soil classification is expressed as a group classification followed by a group index in parenthesis. For example, a soil with a group classification of A-4 and a group index of 20 would be reported as A-4 (20). The group index is computed using the following equation:

$$\text{Group Index} = (F-35)[0.2 + 0.005(w_L-40)] + 0.01(F-15)(I_P-10)$$

where: F = fines content (percentage passing #200 sieve)

$w_L$  = liquid limit

$I_P$  = plasticity index

The group index value is always expressed as a whole number. There is no upper limit for the group index value. Increasing values of group index within a group classification reflect the effects of increasing liquid limit and plasticity index, which coupled with a decreasing percentage of coarser material, combines to reduce the bearing capacity of a specific subgrade.

Computed group values of less than zero are reported as zero. Under conditions of good drainage and thorough compaction, the bearing capacity

of the subgrade material may be assumed to be inversely proportional to its group index. Thus a group index of zero represents a subgrade material with a relatively high bearing capacity while a group index of 20 or more represents subgrade material with a very low bearing capacity.

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (More than 35% passing No. 200)			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
Group classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percent passing:											
No. 10	50 max.	—	—	—	—	—	—	—	—	—	—
No. 40	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40:											
Liquid limit	—	—	—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.	—	NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

\*Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Figure 4 - AASHTO M145 Soil Classification System.

NDOR uses a revised group index chart that indicates the relative desirability of a soil for use as a subgrade material. As with the AASHTO group index, a higher number indicates a less desirable soil. Charts for determining the Nebraska Revised Group Index are shown in Figure 5. Group index values using the Nebraska Revised Group Index commonly range from -4 to 32.

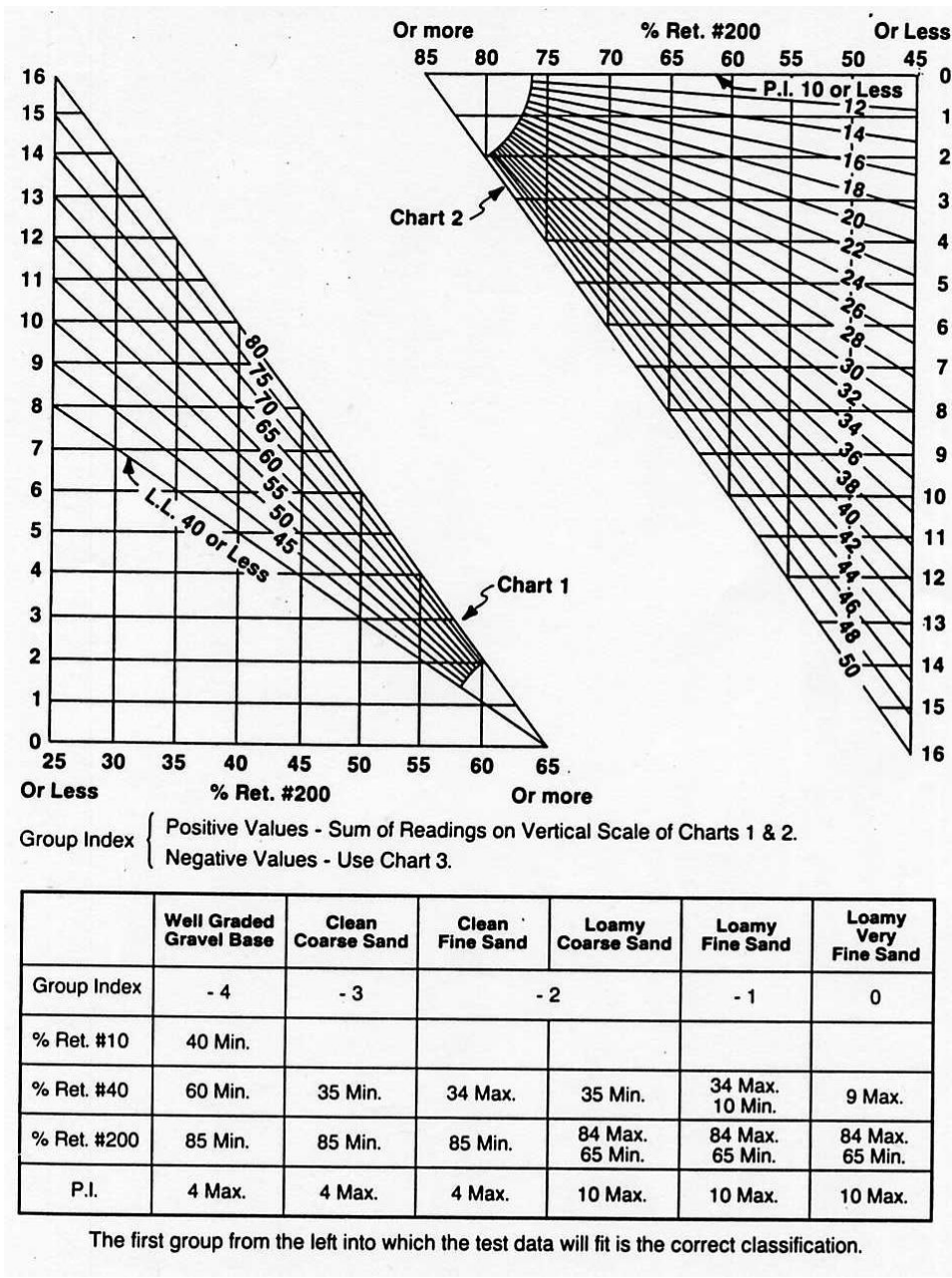


Figure 5 – Nebraska Revised Group Index Charts.

o **Unified Soil Classification System (USCS).**

The Unified Soil Classification System (USCS) is based upon a system developed by Dr. Arthur Casagrande of Harvard University for the U.S. Army Corps of Engineers during World War II. The original system was expanded and revised in cooperation with the U.S. Bureau of Reclamation (USBR), the Tennessee Valley Authority (TVA) and the Federal Aviation Administration (FAA). The USCS is the classification system used for construction and engineering evaluation of soil properties and is the standard referenced in ASTM D2487.

The USCS identifies soils according to their texture and plasticity qualities with respect to their performance as engineering construction materials. Soil identification is based upon the relative percentages of gravel, sand and fines, the shape of the particle size distribution curve and the plasticity characteristics of the soil. Each soil is given a descriptive name and a two-letter symbol, as shown in Figure 6.

Soils are initially divided into coarse-grained or fine-grained soils, depending upon the percentage passing the No. 200 sieve. If a soil has a dark color and an organic odor when moist and warm, a second liquid limit test should be performed on a sample that has been oven-dried in a 110 degree Centigrade (230° F) oven for 24 hours. If the liquid limit after drying is less than seventy-five percent of the liquid limit of the original sample, the soil is classified as organic silt or organic clay.

Coarse-grained soils are subdivided into gravels (G) and sands (S), based upon the percentage passing the No. 4 sieve. Gravels have 50% or more of the fraction retained on the No. 200 sieve also retained on the No. 4 sieve while sands have 50% or more of the fraction retained on the No. 200 sieve passing the No. 4 sieve. Four secondary classifications within each group depend upon the type and amount of fines and the shape of the particle size distribution curve.

Depending upon the liquid limit and plasticity index, fine-grained soils are subdivided into silts (M) and clays (C). Silts are fine-grained soils that plot below the A line on Figure 6, while clays plot above the A line. Silts and clays have secondary divisions based upon whether the soils have relatively high (H) or low (L) liquid limits. Soils in the crosshatched area of Figure 7 have borderline characteristics and require dual symbols.

The U line represents the upper limit of plasticity index and liquid limit for naturally occurring soils. If a soil plots above the U line, one or more the tests used to classify the soil generally contains errors.

Unified Soil Classification System (ASTM Designation D-2487)				
Major division	Group Symbols	Typical Names	Classification Criteria	
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	GW Well-graded gravels and gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
		Gravels with fines	GP Poorly graded gravels and gravel-sand mixtures, little or no fines	Not meeting both criteria for GW
			GM Silty gravels, gravel-sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4
			GC Clayey gravels, gravel-sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
			SW Well-graded sands and gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean sands	SP Poorly graded sands and gravelly sands, little or no fines	Not meeting both criteria for SW
		Sands with fines	SM Silty sands, sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4
			SC Clayey sands, sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
			ML Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Check plasticity chart
			CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	OL Organic silts and organic silty clays of low plasticity			
	MH Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
Fine-grained soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less	CH Inorganic clays of high plasticity, fat clays	Check plasticity chart	
		OH Organic clays of medium to high plasticity		
	Silts and Clays Liquid limit greater than 50%			
Highly organic soils	Pt	Peat, muck and other highly organic soils	Fibrous organic matter; will char, burn, or glow	

Figure 6 – Unified Soil Classification System (USCS).

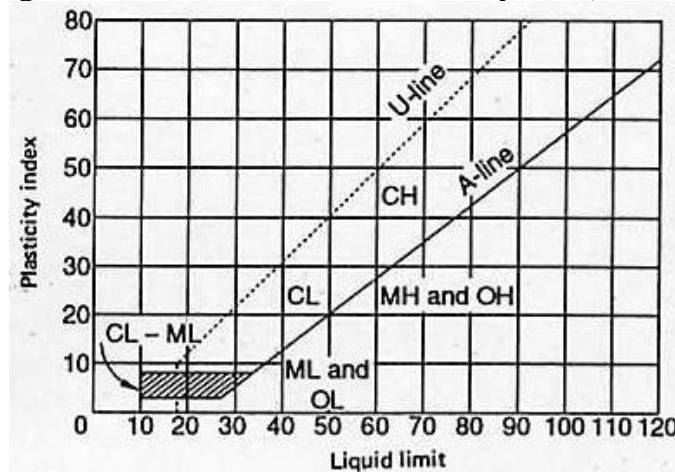


Figure 7 – Plasticity Chart for Classification of Fine-Grained Soils.

Soils containing a high percentage of organic material are usually highly compressible and have other undesirable engineering properties. These soils

are classified into one category, Pt. Soils of this type include peat, humus and many swamp soils.

#### **4.2 Particle Size.**

Particle size analysis is a quantitative determination of the distribution of particle sizes in a sample of soil. Complete particle size analysis requires two tests, a sieve analysis and a hydrometer analysis. The sieve analysis is conducted in accordance with AASHTO T 27/T11 while the hydrometer analysis is conducted in accordance with AASHTO T 88.

Sieve analysis is normally conducted on soil samples where most particles will be retained on the No. 200 (0.075 mm) while the hydrometer test is conducted on soil samples where a majority of particles will pass the No. 200 sieve. In the sieve analysis, the soil sample is shaken through a stack of wire screens with standard size openings. The side dimension of a square hole thus becomes the definition of particle diameter. Hydrometer analysis is based upon Stokes equation for the velocity of freely falling spheres. The diameter of a sphere of the same density that falls at the same velocity as the particle being measured thus becomes the definition of particle diameter for the hydrometer test.

Results of both sieve and hydrometer analysis are often presented on a single particle size distribution curve. Particle size distribution curves can be used for soil classification, determination of hydraulic conductivity, identification of frost-susceptible soils and assessment of soil strength.

#### **4.3 Specific Gravity.**

Specific gravity of a soil is determined in accordance with AASHTO T 100. Specific gravity is the ratio of the mass in air of a given volume of soil at a specific temperature compared to the mass in gas-free, distilled water of the same volume of soil at the same temperature. The specific gravity of most soils lies within the range of 2.60-2.85. Soils with high organic content or porous particles may have a much lower specific gravity, while soils containing an appreciable quantity of heavy minerals may have much higher values of specific gravity.

#### **4.4 Moisture Content (Atterberg Limits).**

Soil moisture content is measured in accordance with AASHTO T 265 or ASTM D 2216-87/ASTM D 4643-87. Moisture content is defined as the ratio of mass of the water in a specimen to the mass of solids in the dry sample. The equation used to calculate moisture content is:

$$W (\%) = M_w/M_s (100\%)$$

The difference in weight between the wet and dry sample is the mass of water,  $M_w$  while the weight of the dry sample is the mass of the soil,  $M_s$ . Note that the equation defining water content differs from standard equations for determining the percentage of constituent materials. A specimen containing 25 grams of water and 25 grams of dry soil has a moisture content of 100%, but water comprises only 50% of the sample by weight.

The moisture content test requires only a scale and a means of drying the sample. The soil can be dried at a constant temperature of 110° C using a conventional oven for 15-16 hours (ASTM D 2216-87), or by using a microwave oven requiring only a few minutes (ASTM D 4643-87). Moisture content is an important soil property, which has been correlated with shear strength, hydraulic conductivity, compressibility and unit weight of the soil. Moisture content is important for interpretation of moisture-density relationships and forms the basis of Atterberg Limit testing.

Albert M. Atterberg defined five different water contents describing soil consistency, now referred to as the Atterberg limits. Starting from a very wet state and then drying, the five water contents defined by Atterberg include the liquid limit, the plastic limit, the shrinkage limit, the sticky limit and the cohesion limit. Only the liquid limit and plastic limit tests are performed on a routine basis to aid in soil classification.

The liquid limit (LL) is the moisture content of the soil at the boundary between the liquid and plastic states. At moisture contents greater than the liquid limit, the soil has little or no shear strength. The plastic limit (PL) is the moisture content of the soil at the boundary between the plastic and semi-solid states. The plasticity index (PI) is the range in moisture content between the liquid limit and the plastic limit, and represents the range of moisture contents over which the soil exhibits plastic deformation. The shrinkage limit (SL) is the moisture content below which an unloaded soil will not change in volume.

#### **4.5 Liquid Limit.**

The liquid limit test requires a Casagrande liquid limit device and a specifically designed grooving tool. The liquid limit of a soil sample is determined by measuring the moisture content at which two halves of a soil mass will flow together over a distance of 0.5 inches (13 mm) along the bottom of a uniform groove separating the two halves, when a bowl containing the soil is dropped 0.4 inches (10 mm) at a rate of two impacts per second. At least three tests at different moisture contents are conducted and the results plotted on semi-log paper. The liquid limit corresponds to the moisture content interpolated to 25 blows. Detailed parameters for this test can be found in ASTM D 4318 and AASHTO T 89.



#### **4.6 Plastic Limit.**

The plastic limit of a soil sample is ascertained by determining the minimum moisture content at which a sample of soil can be consistently rolled into threads 0.125 inches (3.3 mm) in diameter without the material crumbling. Detailed parameters for this test can be found in ASTM D 4318 or AASHTO T 90.

#### **4.7 Shrinkage Limit.**

The shrinkage limit is defined as the water content at which the soil no longer decreases in volume as the degree of saturation decreases. The shrinkage test is primarily performed on soils that may undergo large volume changes as water content increases or decreases.

#### **4.8 Unit weight.**

The unit weight of a soil is represented by the symbol  $\gamma$ . Unit weight is commonly expressed in pounds per cubic foot or kilonewtons per cubic meter. Unit weights can be reported as wet unit weight,  $\gamma_{\text{wet}}$  or dry unit weight,  $\gamma_{\text{dry}}$ . Wet unit weight is calculated by dividing the total weight of a mass of soil containing water by its total volume. Dry unit weight is calculated by dividing the weight of dry soil by its total volume. Wet unit weight thus includes the weight of water as well as the soil particles while dry unit weight includes only the weight of the soil particles. Wet unit weight can be converted to dry unit weight by dividing wet unit weight by one plus the water content.

#### **4.9 Moisture Density Relationship.**

Projects have specifications that indicate the soil density and the range of moisture content that must be achieved to be considered satisfactory. These requirements are normally based upon the results of laboratory compaction tests (more properly described as moisture-density tests). Moisture density tests determine the maximum dry unit weight for a specific soil and the range of moisture contents over which a specified degree of compaction can be achieved.

The most widely used procedure for moisture density testing consists of compacting soil layers in a cylindrical mold using a drop hammer (AASHTO T 99, AASHTO T 180, ASTM D 698, or ASTM D 1557). For each procedure, a mold with uniform dimensions is specified. The number of layers used to fill the mold plus the weight and drop height of the hammer is also specified. To determine the moisture density relationship for a particular soil, separate samples must be compacted at different water contents. Each sample is compacted using the same procedure (identical volume, same number of layers, equal compaction energy). Weighing the mold,

determining the weight of the soil within determine the wet density of the soil. A small piece is cut from the center of the sample and used to obtain water content. The dry unit weight is determined by dividing the wet unit weight by one plus the water content.

A comparison of the results at different water contents reveals that maximum dry density varies with water content. If all results are plotted on dry density versus water content coordinates, a moisture density curve similar to Figure 8 is developed. Maximum dry density corresponds to the peak of the curve. The water content corresponding to the maximum dry density is referred to as the optimum moisture content (OMC). The optimum moisture content is the best possible water content for achieving high density within a specific soil when compaction machinery analogous to the particular test method is used.

Standard practice is to determine the maximum dry density of a soil in the laboratory and then compare this density to the actual dry density achieved during compaction in the field. Specifications to control field compaction are commonly written as a percentage of the maximum dry density between specified water contents.

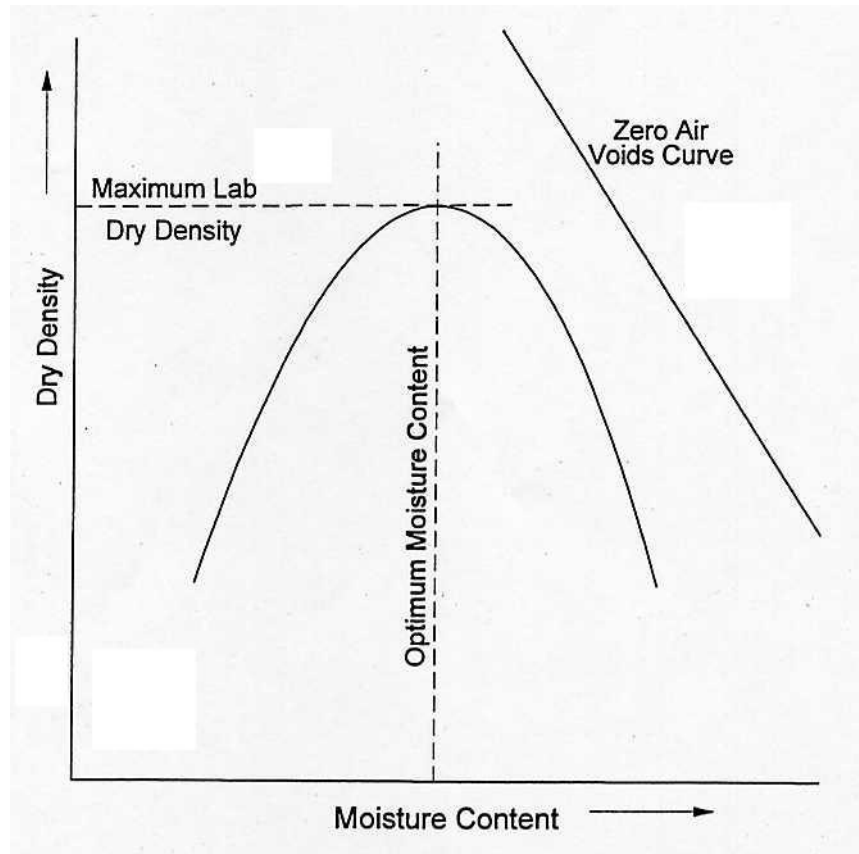


Figure 8 – Moisture Density Curve.

Knowledge of soil OMC is important to both the contractor and the inspector. Informed decisions often must be made relative to the treatment of the soil prior to or during compaction. If the soil has actual moisture content vastly different from OMC, continued compaction will prove uneconomical to achieve the desired results. If the soil is below OMC, moisture can be added by a variety of systems and mixed with the soil by blading or disking. If the soil is above OMC, the contractor may remove water from the soil by scarifying and allowing the surface to dry. In extreme cases, treatment with desiccating mixtures such as lime or removal of the excessively wet soil mass and replacement with drier soil have been used.

#### **4.10 Consolidation/Swell/Collapse Tests.**

##### **o One-Dimensional Consolidation Test.**

The one dimensional consolidation test (AASHTO T 216 / ASTM 2435) can be used to determine the rate and amount of both total and differential settlement for a structure or embankment. The term consolidation refers to the phenomenon of transfer of applied load from the pore water pressure to the soil particles. The results of the consolidation test are normally more accurate if performed on relatively undisturbed samples, which are often obtained by Shelby tube.

A sample is fitted into a ring or cylinder designed to confine the sample against lateral displacement. A vertical load of known magnitude in the range of anticipated design loads is then imposed on the sample. The amount of compression and time required for compression to occur are recorded. The test usually consists of a series of increasing vertical loads, followed by a shorter series of decreasing vertical loads. Each load increment is held for 24 hours or until the linear portion of the secondary settlement curves appears.

The readings from consolidation tests for the various pressure readings are plotting as height versus time and height versus square root of time on separate plots. From this data, the void ratio ( $e$ ) versus log of pressure ( $\log P$ ) curve is plotted. The shape of this curve is significant in that a relatively straight line indicates the sample has been disturbed while a line with two distinct straight line segments with different slopes indicate a relatively undisturbed sample. One result obtained from the  $e$ - $\log P$  curve is determination of the compression index,  $C_c$ , which is defined as the slope of the lower portion of the  $e$ - $\log P$  curve. The compression index is used to calculate the amount of primary settlement expected.

One-dimensional consolidation tests are normally performed only on relatively insensitive normally consolidated clays. This test overestimates the magnitude of settlement for overconsolidated clays and for silty/sandy soils. For sensitive clays, the results of the one-dimensional consolidation test yield settlements that may be much too low. The test gives no indication of

embankment or structural settlement caused by bearing capacity failure or by secondary compression. Consolidation resulting from vibration or earthquake loading will not be included either. Engineering judgment should be judiciously applied to results obtained from consolidation tests.

o **One-Dimensional Swell Test.**

Swelling or expansive soils exhibit behavior opposite to consolidation. Heavily overconsolidated tills and desiccated clays tend to rebound or swell when some of their overburden is removed. These types of soils sometimes absorb water from the atmosphere or ground water, and then exhibit a marked increase in volume.

The one-dimensional swell test is outlined in ASTM D 4546. The same apparatus as utilized in the one-dimensional consolidation test is used to provide a curve of specimen height versus time. The slope of this curve is then analyzed to determine a rate and magnitude of swell.

o **Collapse Potential Test.**

The collapse potential of a particular soil can be determined from any test method that generates an e-log P curve (AASHTO T 216, ASTM D 2435 or ASTM D 4545). Sensitive soils are normally characterized by a nearly vertical segment of the e-log P curve over an extended range of void ratio. This vertical segment indicates that the soil being tested undergoes a dramatic change in void ratio in response to a very small change in load. Soil with these characteristics is referred to as “sensitive”.

Under field conditions, a soil has the potential to collapse if its saturated moisture content is greater than its liquid limit. Based upon dry unit weight and liquid limit calculations, a soil with a specific gravity of 2.67 may collapse if:

- LL = 45 and dry unit weight < 75.7 pcf
- LL = 40 and dry unit weight < 80.5 pcf
- LL = 35 and dry unit weight < 86.1 pcf
- LL = 30 and dry unit weight < 92.5 pcf .

**4.11 Shear Strength Tests.**

o **Unconfined Compression Test.**

The unconfined compression (AASHTO T 208/ASTM D 2166) test is a simple form of triaxial compression test where the confining pressure is zero. Axial force is the only external pressure imposed on the sample. Axial force begins at zero and increases until failure occurs in the sample. The soil sample must be capable of standing in the test apparatus under its own internal strength, so the unconfined compression test is limited to soils having

some cohesion. More information on standard triaxial tests is contained in the paragraphs immediately below.

o **UU Triaxial Test.**

For common triaxial tests, a cylinder of soil, (typically obtained from boring) is wrapped in a membrane to protect it and placed in a closed chamber where a confining pressure can be applied, normally by a fluid, around the outside circumference of the soil sample. The sample sits on a fixed pedestal while a cap attached to a vertical piston rests on top of the sample. During testing, a confining pressure, which is usually held constant, is applied all around and to the top of the sample. A vertical axial load is applied to the sample by piston through the top of the chamber. The axial load is increased until failure occurs. Figure 9 shows a schematic diagram of a triaxial test apparatus.

The test procedure designated “UU” (AASHTO T 296 / ASTM D 2850) is shorthand for an unconsolidated, undrained triaxial test. A sample is placed on the pedestal and the cavity around and above the sample filled with fluid. The drainage valve to the chamber is then closed and the vertical axial stress increased until failure occurs. The results of a UU triaxial test provide the undrained shear strength for a fine-grained soil that has been disturbed.

o **CU Triaxial Test.**

For a CU (Consolidated, Undrained) triaxial test (AASHTO T 297/ ASTM D 4767) the drainage valve is opened and cell pressure is increased until the sample is consolidated to its normal consolidation pressure consistent with its overburden. The drainage valve to the chamber is then closed and the vertical axial stress increased until failure occurs. Since drainage during shear is restricted, excess pore water pressure often develops. Part of the stress imposed on the soil is supported by the pore fluid, a temporary condition that changes as water is forced out of the pore spaces. The results of the CU test are used to evaluate the strength of fine-grained soils under short-term or undrained loading conditions.

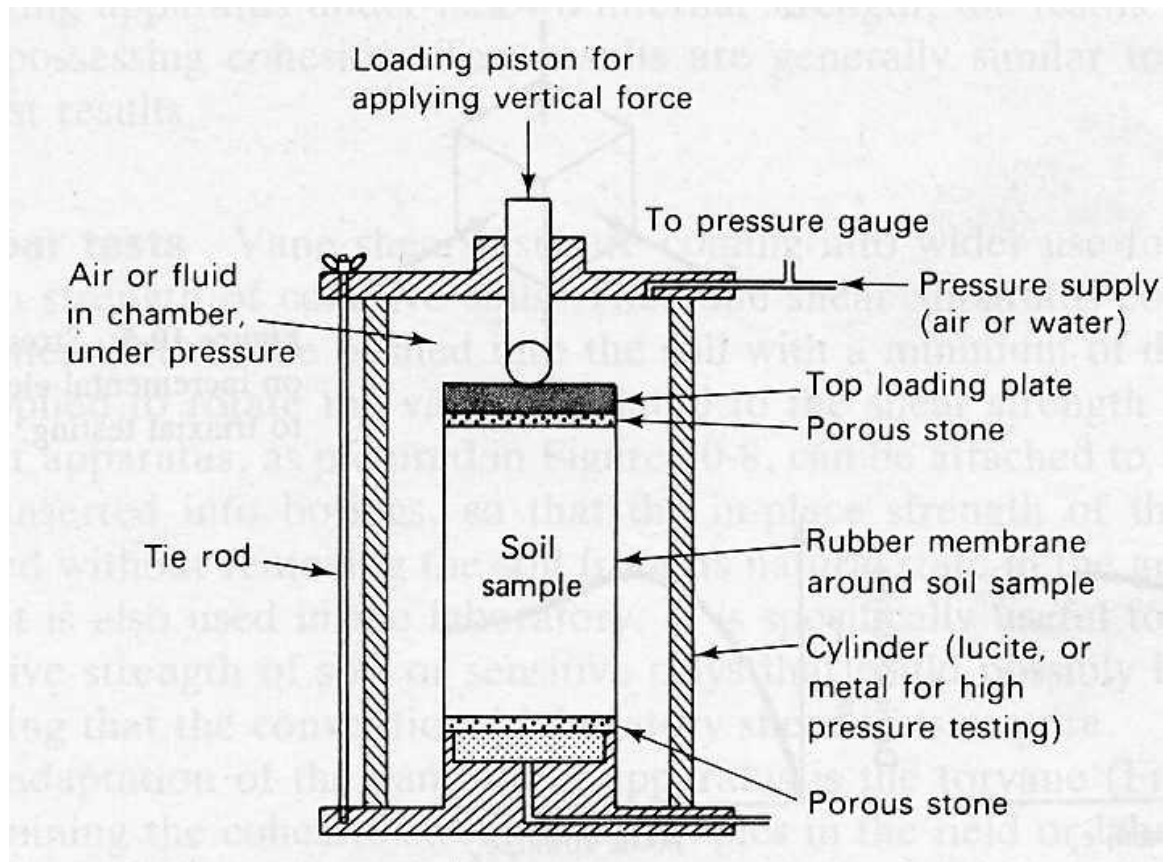


Figure 9 – Schematic Diagram of Triaxial Test Apparatus.

o **Direct Shear Test.**

The direct shear apparatus used for performing the direct shear test (AASHTO T 236/ ASTM D 3080) is a rectangular or circular box, separated into lower and upper halves. After a sample is loaded within the box, a compressive load is applied to compact the soil. The upper half of the apparatus is then forced to move laterally by a shear force that is continuously measured and recorded. The horizontal force causes the sample to shear across the plane between the upper and lower halves of the apparatus. The compressive force is kept constant during the test, while the shear force starts at zero and increases until the sample fails. A record of the magnitude of shearing force and resulting lateral translation is simultaneously maintained so that a volume change versus a shear stress or strain curve can be calculated. Typical test results plot shearing stress versus shearing displacement, as shown in Figure 10.

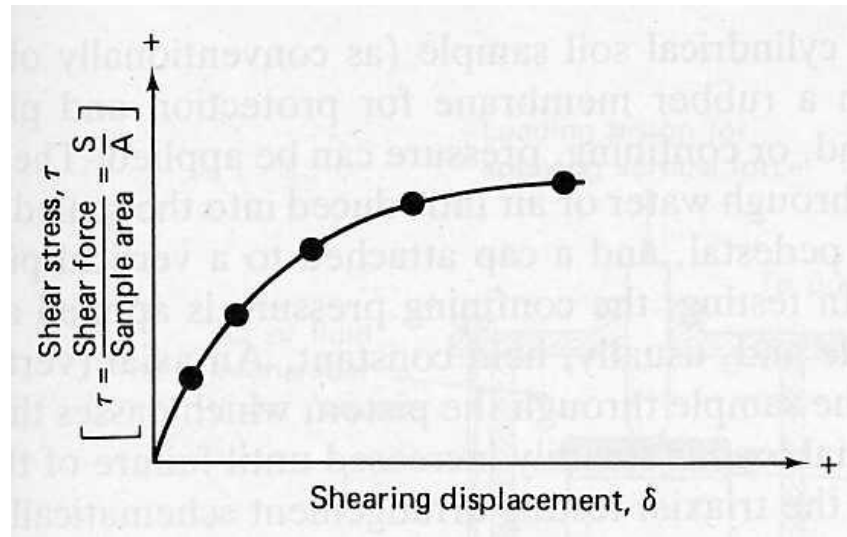


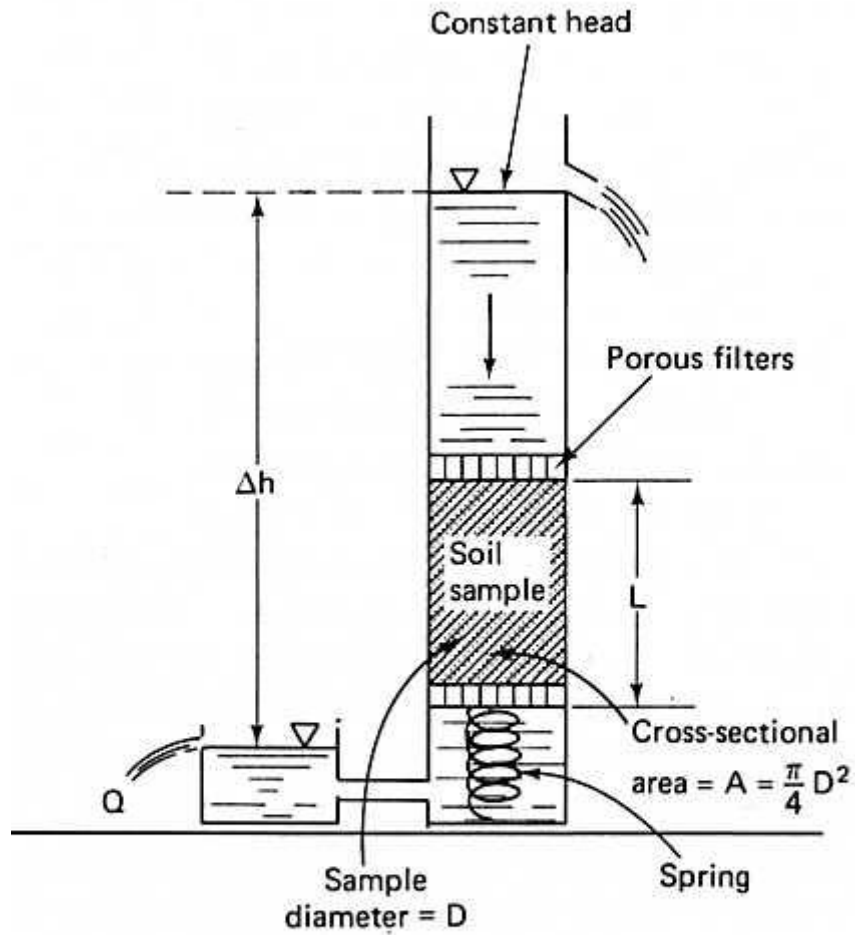
Figure 10 – Plot of Direct Shear Test Data.

Two types of direct shear test are commonly used. In a stress-controlled test, the magnitude of shearing force is controlled. The stress is increased at a uniform rate or in established increments. As each increment of shearing force is applied, it is held constant until no further shearing deformation occurs. In a strain-controlled test, the shearing deformation (lateral displacement) occurs at a controlled rate, usually at a constant speed. The strain controlled shear test is the most widely used.

#### 4.12 Hydraulic Conductivity Tests.

##### o Constant Head Test.

The constant head test (AASHTO T 215/ ASTM D 2434) is used to measure the hydraulic conductivity of a soil. Two reservoirs are used, a higher reservoir on the upstream side and a lower reservoir on the downstream side of the permeameter. The difference in the surface water elevations provides a total driving head, causing water to flow downward through the sample in the permeameter. The volume of water in the lower tank is measured after a given period of time. The time, volume of water collected, length and cross-sectional area of the permeameter and driving head are substituted into the equation shown in Figure 11 to determine hydraulic conductivity for the sample. The constant head test works best when used to determine the hydraulic conductivity of coarse-grained soils.



Constant head test:  $\frac{Q}{t} = kiA$   
 $k = \frac{Q}{t} \cdot \frac{1}{iA} = \frac{Q}{t} \cdot \frac{L}{A(\Delta h)}$

Figure 11 – Constant Head Hydraulic Conductivity Test.

o **Falling Head Test.**

With fine-grained soils, the hydraulic conductivity is generally so low that the time required to obtain a reasonable volume of water through use of the constant head test could days, weeks or months. The falling head test reduces the amount of time required to obtain this information. A standpipe is used to provide an upstream head of water while a lower tank is used on the downstream side as shown in Figure 12.



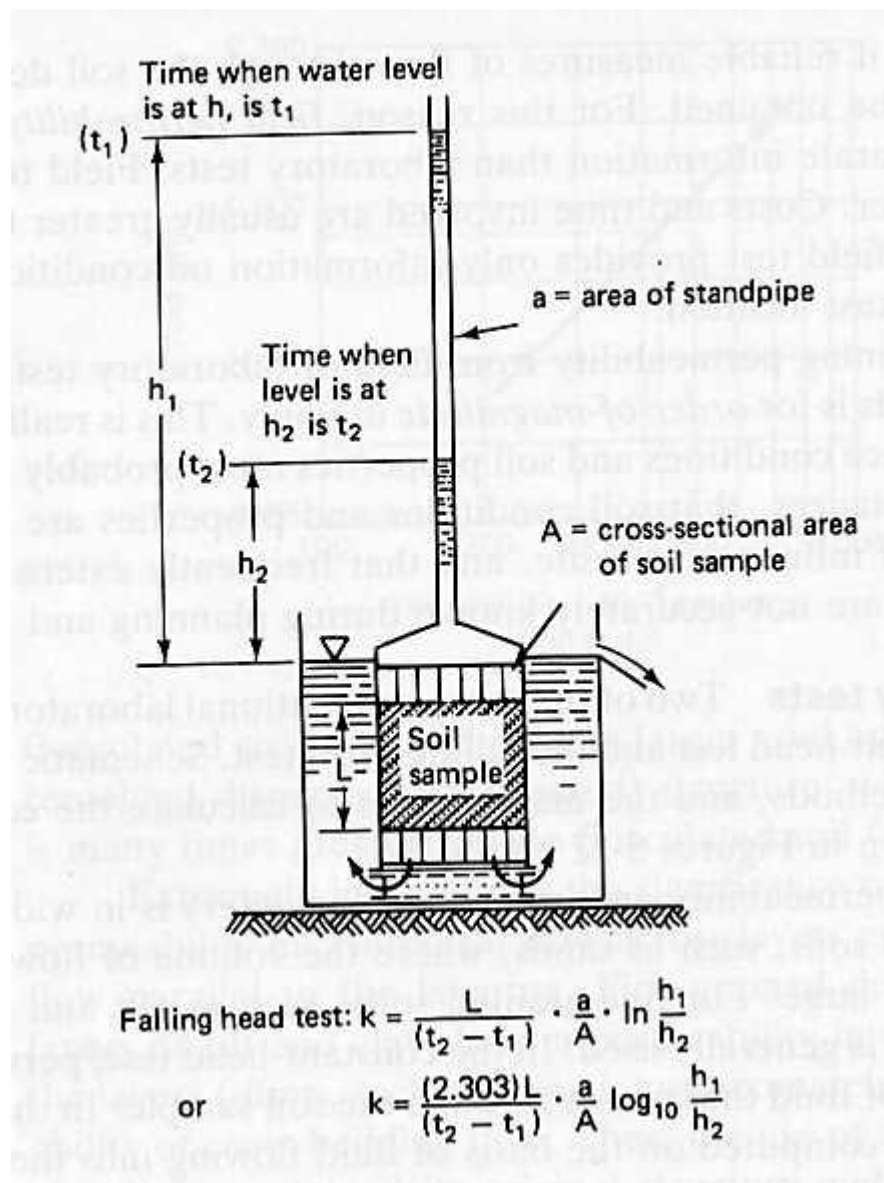


Figure 12 – Falling Head Permeability Test.

The difference in elevation between the two water surfaces is designated as  $h$ , which is the driving head at any time  $t$ . As the test starts, the high head  $h_1$ , initiates water flow. No additional water is added to the standpipe, so the water level drops throughout the test. As the water level falls, both head and flow rate decrease. The test is run for a period of time,  $t$ , to a second head,  $h_2$ , which is above the lower tank elevation. The volume of water that has entered the tank during any time,  $dt$ , is equal to the change in head in the standpipe,  $dh$ , times the cross-sectional area of the standpipe,  $a$ . These values are substituted into the equation shown in Figure 8 to obtain the hydraulic conductivity.

- o **Flexible Wall Permeameter Test.**

Certain limitations are inherent in the procedures used in the constant head and falling head permeability tests, some of which are created by the physical constraints of confining a soil within a fixed diameter cylinder. Permeameters are commonly constructed of some type of plastic, so the sides of the permeameter enclosing the sample may be relatively smooth compared to the average particle size within the sample. Relatively large void spaces can develop next to the sides of the permeameter, allowing water to flow around the sample at a rate well in excess of its true permeability.

Flexible wall permeameters have been created to address this problem. A flexible wall permeameter consists of an elastic tube used as a container for the sample. Water is forced through the sample while it is suspended within this flexible tube. Use of a flexible tube allows the walls of the permeameter to conform to bumps and depressions along the sides of a sample, reducing voids and limiting flow along the sides of the container. A flexible wall permeameter can be used as the sample container for either a constant or falling head test.

#### **4.13 Field Density Tests.**

- o **Nuclear Moisture Density Testing.**

The wet field density of a soil can be determined by the nuclear gauge method (AASHTO T 238) using the direct transmission procedure. The source of gamma radiation is placed at a known depth while the detector remains at the surface. Attenuation of radiation received at the detector is displayed as wet density by the gauge. A calibration curve must be developed for each gauge to correlate the intensity of radiation registered with actual wet density of the soil.

The moisture content of the soil can be determined by the nuclear gauge method (AASHTO T 239) using the backscatter procedure. The neutron source and neutron detector both remain at the surface for this test. When fast neutrons collide with hydrogen nuclei within water molecules, they slow down. The detector measures the quantity of slow neutrons resulting from these collisions. The moisture content is proportional to the total hydrogen content of the soil and is directly related to the water content per unit volume.

The accuracy of nuclear gauge measurements of moisture contents is subject to certain chemical interactions. Organic hydrocarbons such as road oil and asphalt will appear as moisture to the nuclear gauge, which will result in a measured moisture content that is higher than actual. Chemically bound water (such as that found in gypsum) will be included as free water in nuclear gauge observations, resulting in a higher than actual moisture content as well.

Soils containing iron or iron oxides will have higher fast neutron capture rate, which will indicate a lower than actual moisture content on the gauge.

- o **Rubber Balloon Method (ASTM D 2167).**

The rubber balloon method measures in-place moist unit weight of a soil. A hole, approximately six inches in diameter, and semi-spherical in shape, is dug at the desired test location. All of the removed soil is collected so that its total weight and water content can be determined. The volume of the hole is then determined by measuring the volume of water that can be pumped into a rubber balloon filling the hole. Weight of the excavated soil divided by volume of the hole provides the unit weight of the material excavated.

- o **Sand Cone Method (ASTM D 1556).**

The sand cone method is similar to the rubber balloon method. A hole is excavated at the desired test site and the material removed is collected to determine its total weight and water content. A volume of uniform sand with a known unit weight is carefully weighed. Sand is poured into the hole until the hole is filled level with the original ground surface. The weight of sand required to fill the hole is divided by the unit weight of sand to calculate the volume of the hole. The wet (or dry) weight of material removed divided by the volume of the hole determines the corresponding unit weight for the soil.

## Chapter 5

### Soil Modification

When a construction project encounters inadequate soil conditions, four possible alternatives exist. These include:

- o Avoid the site completely. Relocate the planned highway or structure to some other location.
- o Design the planned structure according to limitations imposed by the soil on site. The solution will depend upon performance criteria specified, which may include bearing capacity, embankment stability, subgrade stability, settlement and/or seepage.
- o Remove and replace the unsuitable soil.
- o Attempt to modify the existing soil.

Similar options must be considered when good quality material for construction of embankments, roads, or dams is lacking. This chapter is concerned with the last alternative mentioned, modification of the existing material. Modification of existing soil may take the form of mechanical, electrical, thermal or hydraulic, modification of physical or chemical properties, by addition of inclusions or by confinement.

#### **5.1 Surface and/or Subgrade Treatment.**

##### **5.1a Topsoil.**

Nebraska has many areas where only minor topographic relief is encountered, particularly along roadways that parallel river valleys. In these locations, topsoil may be the primary construction material available. Topsoil, the layer of natural soil found at the ground surface, generally contains varying quantities of organic matter and humus (decaying organic matter) in addition to natural soil particles. Topsoil is often removed and set aside for use when establishing vegetation on slopes or embankments. When topsoil extends to a depth below the shallow root zone (approximately 24 inches or 600 mm), its suitability for use as a construction material or as a fill material should be evaluated.

Similar requirements apply when using topsoil for a construction material as apply when using any soil as a fill material or when selecting any material for use in the top layers of a pavement subgrade. AASHTO M 57 specifies that for construction of embankments and subgrades, AASHTO soil classifications A-1, A-2-4, A-2-5 and A-3 (corresponding to USCS soil

classifications of gravels or sands) are preferred while AASHTO classifications A-2-6, A-2-7, A-4 and A-5 (corresponding to USCS soil classifications of silts and clays) are generally unsuitable without of some type of design or soil modification.

AASHTO M 147 (Table 6) specifies various particle size gradations for material that can be used for construction of subbases, base courses and surface courses. This information is presented in graphical form as Table 8. Gradations A-F are recommended for subbase material and base courses, while surface courses should be composed of material meeting the specifications of gradations C-F.

Properties of unsuitable topsoil can be modified by various methods and procedures, many of which are discussed later in this chapter. Topsoil that does not meet all criteria for use as a subgrade material need not always be removed to its full depth. Removal of a layer of topsoil equal to the thickness of the base course is often sufficient to mitigate most problems.

Table 6 – AASHTO M147 Grade Requirements for Soils Used as Subbase Materials, Base Courses and Surface Courses.

AASHTO M147-65 (80)		Percentage Passing by Mass					
Sieve	Size	Grades					
(mm)	(in)	A	B	C	D	E	F
50	2	100	100				
25	1		75-95	100	100	100	100
9.5	3/8"	30-65	40-75	50-85	60-100		
4.47	No. 4	25-55	30-60	35-65	50-85	55-100	70-100
2.00	No. 10	15-40	20-45	25-50	40-70	40-100	55-100
0.425	No. 40	8-20	15-30	15-30	25-45	20-50	30-70
0.075	No. 200	2-8	5-20	5-15	5-20	6-20	8-25

### 5.1b Unsuitable Soils.

As a general rule, soil used in most highway construction applications should have a minimum dry unit weight of 90 pounds per cubic foot, an organic content less than 2%, a liquid limit of less than 50% and 70% maximum by weight passing the No. 40 sieve. Soils outside of these limits will normally require some type of modification to alleviate adverse characteristics.

If a soil with a liquid limit higher than 50% is present in the subgrade or must be used as fill material, both soil treatment and drainage options are available. Examination of existing pavement in the immediate area of the

project may reveal if high plasticity soils are unstable. If the soil is stable, no treatment is necessary. If treatment is necessary, it generally consists of various methods of drainage, removal and replacement of the soil, treatment of the soil with additives to reduce its plasticity or some combination of two or more of these procedures.

Some soils have a natural structure that may become unstable and collapse under certain impact loading and moisture conditions. The natural structure of loess and other slightly cemented soils may collapse when water infiltrates the soil layers. Placing a pavement directly over unaltered loess will often trap moisture beneath the pavement, allowing the loess to accumulate moisture from the bottom upward. Vibration of traffic on the roadway over a loess deposit may then cause the soil structure to collapse. Normal construction practices result in sufficient densification of loess that collapse will not occur.

Dispersive soils represent another problem. Dispersive is a term applied to soils containing clay minerals that are composed of a high percentage of sodium montmorillonites. These particular clay minerals break down to form a suspension when exposed to water. The suspended clay particles can be transported away when exposed to moving water, leaving voids in the soil structure. This phenomenon is known as piping when the voids assume a cylindrical shape with the long axis in the direction of water movement. Piping often occurs along foundations, and if allowed to proceed, can result in loss of bearing and ultimately structural failure.

ASTM D 6572 - Standard Test Methods of Determining Dispersive Characteristics of Clayey Soils by the Crumb Test can be used to determine if a soil has dispersive qualities. Compaction with proper equipment at the specified range of moisture greatly reduces problems with dispersive soils. Cement, gypsum, fly ash and lime have all been used to treat dispersive soils with varying levels of success.

## **5.2 Soil Modification Procedures.**

Some natural soils do not possess adequate strength and stiffness to support a roadway. When materials to remove and replace these soils are unavailable and the roadway cannot be moved, soil modification procedures must be used. A variety of soil modification procedures are available at various costs. It is not always evident which procedure is optimal for a given situation.

### **5.2a Surface and/or Subsurface Drainage.**

Soil drainage may have several objectives, including lowering a water table, redirecting seepage away from a cut section or reducing the water content of a soil mass. Surface drainage techniques have traditionally been based upon

gravity drainage with gravity flow or pumping to remove collected water from sumps or ditches. These techniques are relatively inexpensive and work well for relative shallow excavations in coarse granular soils. Slopes consisting of fine-grained soils can be gravity drained by constructing a toe drain with gravel filled slots.

Subsurface drainage is accomplished using a combination of granular or geotextile filters, slotted pipe, trenches, sumps, wells, and/or drainage fabric. Design of subsurface drainage systems is more complex than design of surface drainage systems. Internal drainage of pavement systems was discussed in Chapter 3.

### **5.2b Modification of Surface Soil Moisture Content.**

The strength and stiffness of a cohesive soil are primarily dependent upon moisture content and degree of compaction. Soil with moisture content significantly greater than optimum is inherently unstable, and will prove difficult to use as a platform for operation of construction equipment. Disking, tilling or scarifying and allowing the soil to dry naturally are effective only for the top 8-12 inches (200-300 mm). Actual reduction in moisture content for surface soil is very dependent upon weather conditions while the soil is being worked.

If a soil remains moist or becomes wetter with depth, drying the surface may not suffice. Heavy repeated loading of soil layers where a drier layer is located above a wetter layer causes the moisture content in the surface layer to increase with a corresponding decrease in strength. Granular soils that drain relatively rapidly can be stabilized by the installation of subdrains alone. Cohesive soils require application of external loads after the drainage system has been installed to drain effectively.

If a soil has been compacted drier than optimum moisture content, the soil may have sufficient strength but fail to satisfy density requirements. Low-density soils tend to absorb greater moisture when exposed to water, which is accompanied by a corresponding decrease in strength. The significance of this loss of strength in the subgrade depends upon overall pavement design. The most common method of increasing soil moisture content during construction is use of a water distributor and disc to mix water into the surface layer immediately before compaction.

If a clay soil has been compacted dry of optimum and at high density, the soil has significant strength, but fails the moisture requirements. High density soils (especially clays) with low moisture contents will eventually absorb moisture, especially beneath pavements. This absorption can cause the clay soils to swell and heave the pavement structures causing damage to the pavement and a rough surface. Again, the most common method of

increasing soil moisture content during construction is use of a water distributor and disc to mix water into the surface layer immediately before compaction. It is important that proper field testing is performed to ensure that the moisture content and density requirements are met.

### **5.2c Use of Soil Admixtures.**

There are a variety of soil stabilizing agents available, which are commonly divided into two categories, active and passive agents. Active agents produce a chemical reaction with specific soil minerals, which in turn produces desirable changes in the engineering characteristics of the soil. Lime is one example of an active agent. When lime is added to medium to fine-grained soils, it produces numerous desirable changes in soil properties.

Passive stabilizers do not react chemically with the soil, but instead bind together natural aggregates within the soil. Bituminous admixtures, cement and lime-fly ash mixtures are common examples of passive stabilizers. Passive stabilizers are more commonly applied to coarse-grained soils.

#### **o Lime Stabilization.**

During periods of excessive precipitation, the physical condition of a roadway construction site on cohesive soils may be so soft and wet as to prevent construction activities. If the soil cannot be dried out by aeration within an acceptable period of time, consideration should be given to treating the soil with an additive that will improve its strength. Lime is the most commonly used additive in these situations. A small quantity of lime may be added to the soil to dry out the subgrade material. If a greater quantity of lime is added to the same soil, the lime stabilized soil mixture will gain sufficient strength to serve as the roadway base course. This process is known as lime stabilization. Practical lime admixtures vary from 2% to 8% by weight. The optimal percentage of lime to be used for each project should be determined by triaxial or other specified tests.

Lime treatment has several inherent advantages. Removal and replacement of material below the subgrade is minimized, saving time and money. Lime stabilized soil has improved workability, resulting from a decreased plasticity index due to an increased plastic limit. Lime treatment increases the strength of a clay soil as measured by an unconfined compression test. Increased strength confers improved durability under cyclic loading and improved resistance to wind, water and freeze-thaw cycles.



- o **Soil Cement Stabilization.**

The most commonly used admixture for soil stabilization is Portland cement. The reaction of cement and water in the soil forms cementitious calcium and aluminum hydrosilicates, which bind granular soil particles together. Hydration of the cement results in slaked lime,  $\text{Ca}(\text{OH})_2$ , which in turn reacts with the clay components of the soil to improve strength. Hydration is independent of the soil type, so cement stabilization is effective for a wide range of soil types. Soil cement stabilization results in increased strength and stiffness, better volume stability and increased durability of the soil being treated.

The benefits of soil cement stabilization are dependent upon the degree of mixing and compaction achieved under field conditions. Good mixing and good compaction result in a dense, strong subbase. Typical cement contents vary from 2% to 10% by weight. Cement stabilization reduces the plasticity index of most soils, improving their workability. The unconfined compressive strength of soil increases directly in proportion to the percentage of cement used during the treatment process.

- o **Calcium Chloride Stabilization.**

Calcium chloride is a common salt with properties that make it particularly suitable for certain geotechnical engineering applications. Calcium chloride is hygroscopic, meaning that it attracts and absorbs moisture from the atmosphere.  $\text{CaCl}_2$  is highly soluble in water, raises the surface tension of and a lowers the freezing point of water.

Calcium chloride replaces the  $\text{Na}^+$  ions within the diffuse double layers of sodium montmorillonites with  $\text{Ca}^{++}$  ions, reducing the thickness of that layer, thereby decreasing the plasticity and increasing the strength of the soil.  $\text{CaCl}_2$  reduces evaporative water loss from soils, facilitating moisture control during construction. Its hygroscopic properties make calcium chloride an ideal substance to help control dust on unpaved roads at construction sites.

- o **Fly Ash Stabilization.**

Fly ash is a waste product resulting from the combustion of coal. It is transported out of the combustion chamber by flue gasses and extracted by electrostatic precipitators and filter bags. Fly ash is composed primarily of silt sized particles and is usually dark to light tan in color.

Under a microscope, fly ash appears to be glassy spheres surrounded by shards of crystalline material. The principle components of fly ash are silica ( $\text{SiO}_2$ ), alumina ( $\text{Al}_2\text{O}_3$ ), ferric oxide ( $\text{Fe}_2\text{O}_3$ ) and calcium oxide ( $\text{CaO}$ ). ASTM C618 divides fly ash into two categories, class F and class C. Class F

fly ash is produced by burning anthracitic or bituminous coal, while class C fly ash is produced by burning subbituminous or lignite coal. Class F fly ash is pozzolonic while class C fly ash is both pozzolonic and cementitious.

Fly ash (F) is commonly mixed with lime (L), cement (C) and/or aggregate (A) to create LFA, CFA or LCFA bases and subbases for roadways. Guidelines for the relative percentages of constituents for various types of soils are available from either the FHWA or from NDOR. Fly ash mixed with either cement or lime can also be used to stabilize a variety of soils that may serve as the surface layer for light traffic roadways. Stabilization of a sandy base with fly ash/cement mix (versus cement alone) creates a stiffer base with less hydraulic conductivity. Fly ash/cement mixtures used to stabilize soils exhibit less shrinkage and surface cracking than mixtures containing cement alone. Fly ash by itself can be used to dry out wet subgrade soils and increase soil strength.

- o **Bitumen Stabilization.**

Bitumen refers to the product obtained by processing the residue that remains after distillation of crude oil. Bitumen is generally mixed into the soil in the form of an emulsion or cutback, and only rarely applied as “foamed” bitumen. In an emulsion, small drops of bitumen are dispersed in water and prevented from coagulating by chemical emulsifiers. When applied as a cutback, a volatile solvent that evaporates after placement temporarily reduces the viscosity of the bitumen. Foamed bitumen is generally applied to the soil by a process where steam is blown through the hot bitumen using special nozzles, forming thin film bubbles with excellent coating ability.

Bitumen is generally added to a soil to reduce water absorption or to add cohesion to granular soils. Strength of compacted bitumen stabilized soil increases with the quantity of binder added until a maximum stability is reached; thereafter increasing the bitumen quantity decreases strength. The effectiveness of bitumen toward imparting cohesion and water absorption depends primarily upon the type of soil. Emulsions are said to be most effective when applied to well-graded sands with a fines content of 8-20%. Sands with greater fines content will have improved strength and better water resistance if bitumen is applied as a cutback rather than as an emulsion.

The soils most suitable for bituminous admixtures include sandy gravels, sands, clayey and silty sands, and fine crushed rock. Bitumen is not as common as other soil admixtures, primarily because of its relatively high cost. Considerable expertise is required in controlling viscosities, choosing correct proportions and mixing times for emulsions and cutbacks and in optimizing curing rates.

## **5.2d Over-Excavation and Replacement of Soil.**

Removal of a weak subgrade soil and replacement with more suitable material is a commonly used method of treatment. If consolidation is not a problem, relatively shallow cuts may be sufficient. When deep deposits of expansive clays are encountered, extensive removal and replacement may be required to alleviate problems with consolidation.

Another often-utilized solution is to cover a soft subgrade with a predetermined depth of granular material or to remove a predetermined depth of soft material immediately below the finished grade line and replace it with granular material. The granular material distributes the wheel loads over a much larger area of the subgrade, thereby stabilizing the roadway.

The removal and replacement method is simple and does not require equipment other than that normally available on most construction projects. If suitable fill material is available near the project, this method can be quite inexpensive. Costs associated with this method include excavating and disposing of the unsuitable material plus purchasing, placing and compacting the replacement material.

Several problems may be encountered when using the removal and replacement method. If subgrade material lacks strength because of a high water table, the properties of the material used as backfill may also be adversely affected by the water conditions. If high water table conditions exist at the project site, the backfill selected should be relatively unaffected by changes in water content. Unless some type of separation membrane is used between the subgrade and granular layer, material from the soft subgrade may migrate into the granular material, significantly reducing the effectiveness of the granular layer over time.

## **5.2e Soil Reinforcement.**

Reinforcement of a soil mass by strips, bars, meshes, or fabrics imparts a greater than normal tensile strength to a mass of soil. Structures designed and constructed using reinforcing strips, bars, meshes, or fabrics are referred to as reinforced earth structures. The most common type of reinforced earth structure consists of horizontal layers of soil interspaced with reinforcing strips, bars, grids or fabrics. The reinforcing members may or may not be attached to the wall face.

Backfill criteria and construction specifications for reinforced earth structures are relatively stringent. Percentage of fines (particles <0.08 mm in diameter) are normally less than 15% of the backfill material by weight. If fines compose greater than 15% of a material, it may still be suitable for use as backfill, but special tests must be performed to determine that sufficient

pullout resistance can be developed between the reinforcement and the backfill material before it can be used.

Backfill must be placed and compacted at less than optimal moisture content. Backfill on FHWA projects is restricted to soils falling within AASHTO soil classification A-1-a (USCS GW or SW). Current backfill requirements are designed to produce a freely draining structure with a soil reinforcement friction factor ( $\tan \delta$ ) not less than 0.3. Backfill material restrictions are derived from measurements of undrained shear strength of granular materials contaminated by clay and upon direct shear tests on reinforcing materials performed using a standard shear box.

Reinforced soil failure modes are characterized as either internal or external. If the major failure plane lies outside of the reinforced earth mass, the failure mode is external (also known as global failure). External failure modes consist of bearing failure, sliding and overturning; these failure modes are analyzed using traditional retaining wall analyses. Internal failure modes consist of rupture of the reinforcement, slippage between the reinforcement and the surrounding soil, failure of reinforcement by excessive deformation or by buckling of the face elements.

Reinforcing strips were initially composed of galvanized metal in various configurations and sizes. Many different shapes and types of materials are now used for reinforcement, including mats, grids and meshes. Mats, grids and meshes perform the double function of strengthening the soil surface while acting as reinforcement for the soil mass. Meshes, mats and grids consist of flexible sheets of varying thickness with relatively large openings in relation to the size of the connecting segments. Extrusion, stretching, or fabric welding processes are used to create these materials.

In recent years, a wide variety of synthetic materials have become available that have rapidly gained acceptance. Synthetic materials have proven easy to transport and to place, exhibit predictable properties once emplaced, and are able to withstand degradation under subsurface conditions. Synthetic fabrics are commonly referred to as geotextiles, a broad classification encompasses numerous materials developed for specific geotechnical engineering applications, including geonets, geogrids and geocomposites. The term “geotextile” commonly refers to a synthetic fabric that has the general appearance of cloth but has no attached accessories, such as a reinforcing mesh.

Geotextile fabrics are anisotropic with regard to many of their material properties. Fabric properties are listed with regard to the machine direction, the direction in which the fabric was manufactured and the cross machine direction, which is orthogonal to the machine direction. Geotextile fabrics are classified according to the way in which the threads were linked together,

with woven, non-woven and knitted fabrics representing the most common types. Non-woven fabrics commonly have a random orientation of strands within the fabric itself. To produce non-woven fabrics, filaments of material are spread on a conveyor belt and then bonded by the addition of resins or by heating.

The introduction of geotextile fabrics into the U.S. market in the 1970s prompted development of many different forms of geosynthetics, which were subsequently combined with other materials to form composites tailored to specific applications. One example is a geotextile envelope constructed around a synthetic core that is incompressible enough to hold the geotextile sheets apart, allowing water to flow easily within the plane of the combined materials. This combination is known as a geocomposite. Geocomposites allow a single item to be ordered, transported to the site and inserted as a drain. Geocomposites can provide an excellent drainage system at considerable savings when compared to the cost of using natural materials to construct a similar drainage system.

A whole series of geosynthetic products are now available which can perform specific functions in addition to soil reinforcement. These functions include separation of material, filtration, and drainage. Table 7 lists some of the more common uses of geosynthetic products within the transportation industry.

American Society for Testing and Materials (ASTM) procedures for testing the mechanical, hydraulic and durability properties of geotextiles are detailed in Table 8.

Geotextiles are extremely versatile, adapt readily to site circumstances, and can be combined without adverse effects with most traditional construction materials. The key to design with geotextiles lies in understanding the various functions of geotextiles and relating these functions to improvement in soil properties.

**Table 7 – Common Uses of Geotextiles**

<b>Application</b>	<b>Use(s) of Geotextile</b>
Pavement on soft soil	Increase subgrade stability; decrease rutting
Pavement overlays	Inhibit crack transmission to surface layer
Structures	Reinforce soils to increase bearing capacity for foundations
Embankments	Provide stability; provide drainage
Natural slopes	Provide drainage; reinforce soil; erosion control
Retaining structures	Reinforce and/or separate backfill
Rivers and streams	Erosion control; replace/improve filter layers
Water pollution	Extract/collect granular pollutants; relieve pore water pressure on fine soils

**Table 8 – ASTM Procedures for Geotextile Testing**

<b>Topic</b>	<b>ASTM</b>
Basic physical properties, sampling	D 4354
Determination of tensile properties – wide strip	D 4595
Determination of tearing strength – trapezoidal	D 4533
Test method for breaking load and elongation	D 4632
Determination of seam strength	D 4884
Determination of pore size distribution	D 4751
Determination of permittivity	D 4491
Determination of transmissivity	D 4716
Determination of durability	D 4355

AASHTO M288 provides three different strength classifications (Table 9) that helps the designer determine the minimum requirements necessary for installation and long-term survivability based on the application intended and the soil type. Separation is achieved if the geotextile fabric prevents the mixing of two adjacent soils. The principle property of a geotextile necessary to achieve and maintain separation is strength. Most fabrics will act as natural separators if their integrity is not compromised. Design criteria for separation therefore reference the mechanical properties of the fabric, particularly tensile properties, tearing strength, breaking load and elongation. If water is present on one or both sides of the fabric, the fabric must also be evaluated as a filter, as water movement will transport some particles as it makes its way to and through the fabric. Moving particles can collect against the fabric, causing excessive pore water pressure buildup, ponding of water and ultimately mechanical failure of the separator.

**Table 9 – AASHTO M288 Table 1**

	Test Methods	Units	Class 1		Class 2 <sup>a,b</sup>		Class 3	
			Elongation <50% <sup>c</sup>	Elongation <50% <sup>c</sup>	Elongation <50% <sup>c</sup>	Elongation <50% <sup>c</sup>	Elongation <50% <sup>c</sup>	Elongation <50% <sup>c</sup>
Grab Strength	ASTM D 4632	N	1400	900	1100	700	800	500
Sewn Seam Strength <sup>d</sup>	ASTM D 4632	N	1260	810	990	630	720	450
Tear Strength	ASTM D 4533	N	500	350	400	250	300	180
Puncture Strength	ASTM D 6241	N	2750	1925	2200	1375	1650	990
Permittivity	ASTM D 4491	sec <sup>-1</sup>	Minimum property values for permittivity, AOS, and UV stability are based on geotextile application. Refer to Table 2 for subsurface drainage, Table 3 and Table 4 for separation, Table 5 for stabilization, and Table 6 for permanent erosion control.					
Apparent Opening Size	ASTM D 4751	mm						
Ultraviolet Stability (retained strength)	ASTM D 4355	%						
a	Required geotextile class is designated in Table 2,3,4,5, or 6 for the indicated application. The severity of installation conditions for the application generally dictates the required geotextile class. Class 1 is specified for more severe or harsh installation conditions where there is greater potential for geotextile damage, and Classes 2 and 3 are specified for less severe conditions.							
b	All numeric values represent MARV in the weaker principle direction (See Section 8.1.2).							
c	As measured in accordance with ASTM D 4632.							
d	When sewn seams are required. Refer to Appendix for overlap seam requirements.							
e	The required MARV tear strength for woven monofilament geotextiles is 250 N.							

Where water exits from an earthen structure or moves from a relatively fine to a coarse layer, fine particles may be carried along with the water, leading to internal (piping) or external erosion, instability due to buildup of pore water pressure, or fine particle accumulation in the drainage pipe, trench, or layer. Traditional methods of alleviating this process have included one or more graded filter layers, increasing in grain size and hydraulic conductivity in the direction of flow toward the collection system. A granular filter must have significantly more hydraulic conductivity than the soil it is supposed to protect, but should not have voids big enough to allow soil particles from the protected material to pass through. The specifications for granular filters to

prevent migration of protected soil into the filter without impeding flow of water were discussed in Section 3.5 of this manual. Similar criteria apply for geotextile filter design, with the respective criteria commonly referred to as permeability and retention. Common applications of geotextile fabrics used as filters are shown in Figures 13 and 14.

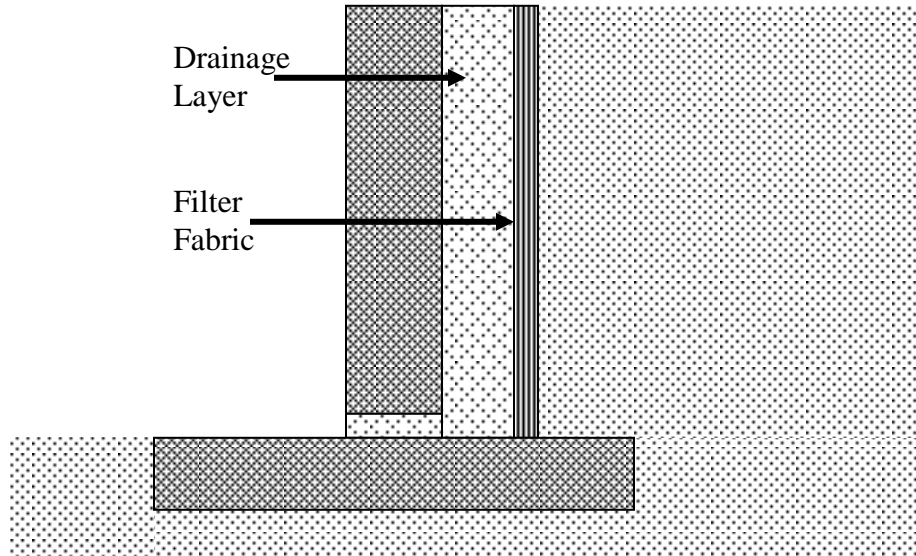


Figure 13 – Geotextile Used as a Filter Fabric Behind a Retaining Wall.

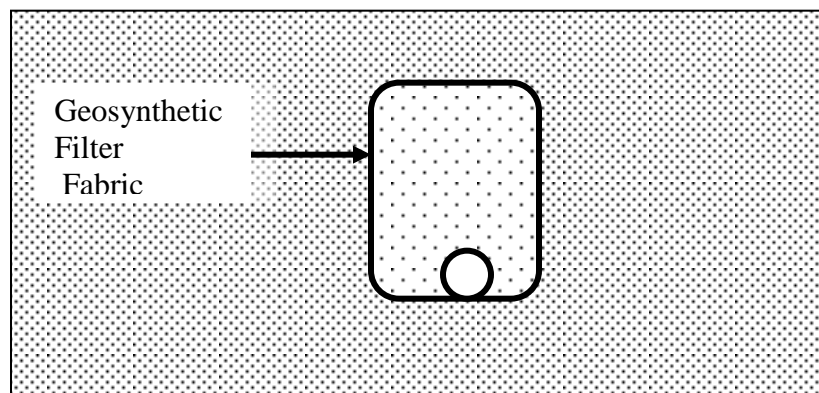


Figure 14 – Geotextile Filter Fabric Used in Trench Drain.

Removing water from soil has many beneficial effects including reduction of pressure on retaining walls, increase in subgrade stability, and increase in the stability of slopes. The availability of geotextiles and geocomposite drainage materials has made the solution to many drainage problems easier and much more economical. Geocomposite drains, consisting of a geosynthetic core



wrapped in a geotextile, are readily available in strip and sheet configurations. A geocomposite strip used to drain fill behind a vertical retaining wall is shown in Figure 15.

Geocomposite strip drains have all but replaced sand and waxed cardboard “wick” drains in surcharge applications on cohesive soils, as illustrated in Figure 16. Drains used in this application serve the temporary function of accelerating the consolidation of a clay layer under a surcharge load.

Depending upon the properties of the geosynthetic fabric selected, the capillary rise of water within a fabric with small voids may lead to the siphoning effect, which can be advantageous in specific water removal applications. Alternately, a geosynthetic fabric with large voids may be used to break the capillary head, thereby preventing frost heave or problems with moisture sensitive soils.

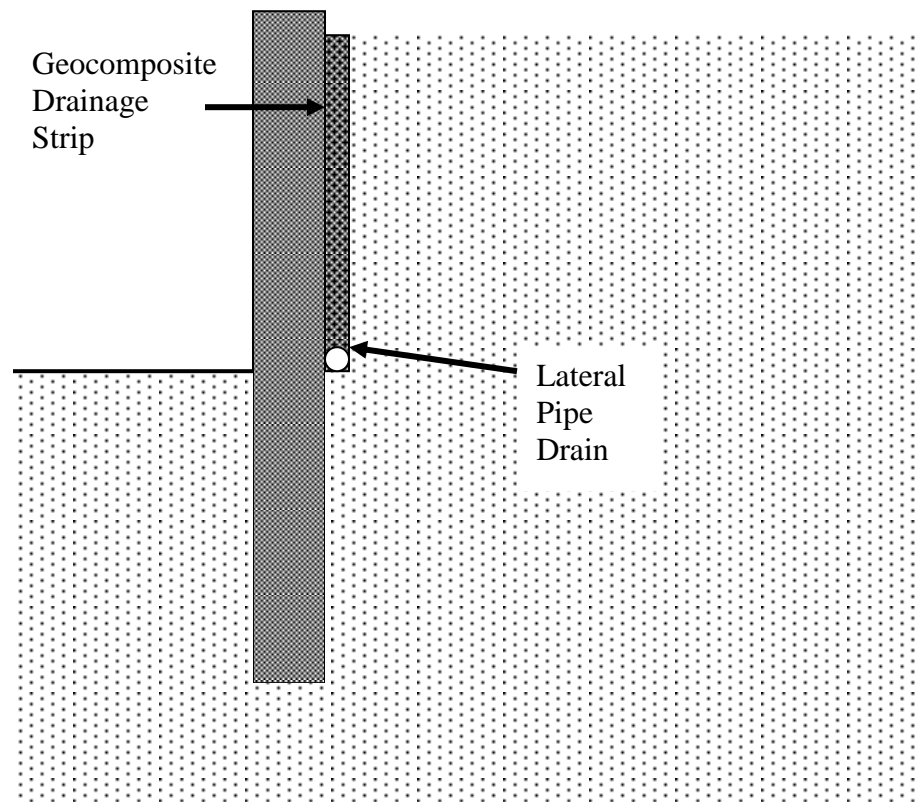


Figure 15 – Geotextile Used as Drain Behind a Retaining Wall.

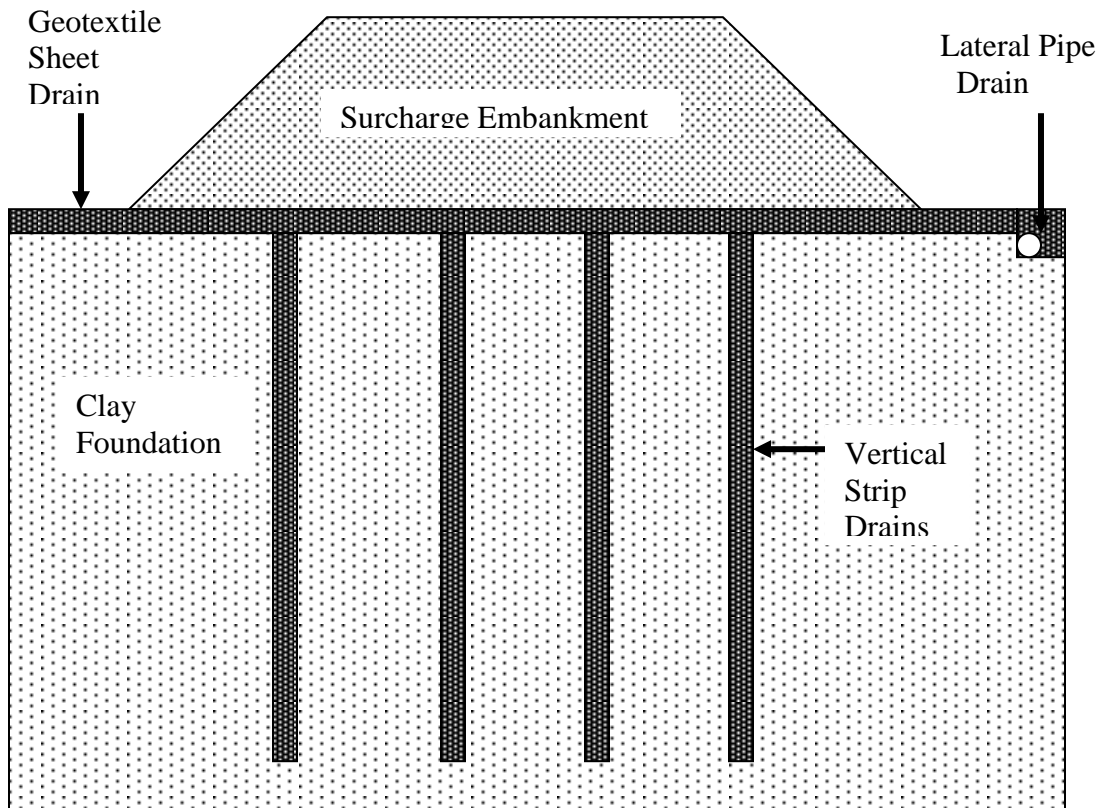


Figure 16 – Consolidation Surcharge Loading Using Strip and Sheet Geocomposite Drains.

Placing geosynthetic fabric or grid over a soft subgrade and covering it with a granular material can increase the stability and structural strength of most subgrades. The fabric or grid maintains the soil beneath separate from the granular material above as it aids in distributing loads over the subgrade surface. The fabric or grid may also allow water to flow from the subgrade upward into the granular layer, providing an upward and outward drainage path for water from the saturated soil below.

Several geosynthetic manufacturers have developed granular layer thickness design software that is generally available free from the manufacturer. This software is relatively easy to learn to use. A reduction in thickness of the granular material layer of one-fourth to one-third of the originally required thickness can commonly be achieved with the use of geosynthetics. A reduction in the required granular layer thickness results in a reduction in the depth of cutting required, as well as the quantity of material that must be

purchased and transported to the project site. The design engineer must determine whether the geosynthetic fabric and installation costs are offset by the reduced cost of cutting to a shallower depth and a reduced thickness of aggregate. Subgrade strength, extent of traffic loading and properties of the geosynthetic material all influence the thickness of granular layer required.

Open mesh type geotextile fabrics in conjunction with straw, mulch or wood shavings and seeds have been used to provide temporary stability to cut slopes until vegetation is established. Open mesh geotextiles can be used to create sand fences for dune management. Denser fabrics can be used as silt curtains to prevent floating matter and suspended particles from entering stream channels.

## Chapter 6

### Construction Procedures and Instrumentation

Soil provides the foundation for most of man's structures. Soil is also used extensively as a construction material. The principle reason for using soil as a building material is that soil is available almost anywhere, it is durable and it has comparatively low cost when compared to other building materials.

When soil is used as a construction material, it is typically placed in relatively thin layers to develop a final section and elevation. Each layer is compacted before being covered by the next layer. When each layer has been properly placed and compacted, the resulting soil mass has strength and support properties considerably better than the natural soil strata.

When soil is used as a foundation material, it is desirable for the soil to have certain properties. The soil should possess adequate strength, be relatively unresponsive with regard to volume changes as the water content varies, be durable and not deteriorate over time. These factors can be achieved to some degree at all sites through selection of the proper soil type and by use of proper placement techniques.

Almost any soil can be used for fill, if it does not contain organic or foreign matter that would decompose and undergo volume change after placement. Granular soils are generally the preferred material at construction sites, as these soils are capable of developing high strength with minimal changes in volume after compaction, and can be emplaced under most moisture conditions. Compacted silt is stable, develops fairly high strength, and has a slight tendency to exhibit volume changes with variations in moisture content. Silty soils can, however, prove very difficult to compact when the soil is wet or under rainy conditions. Compacted clay soils can develop high strength, but the assemblage of clay minerals present determines their stability against shrinkage and expansion under varying moisture contents. Compacted clays have low to very low hydraulic conductivity, a factor that can prove beneficial or deleterious depending upon hydraulic conductivity needed for the project. Clay soils can be compacted only with great difficulty when wet.

#### **6.1 Embankments**

Design of an embankment to support a roadway must consider settlement, slope stability and bearing capacity at the base of the embankment. Settlement must be within required specifications, especially when the embankment is located adjacent to a rigid structure such as a bridge. Differential settlement is normally more of a concern than total settlement. When excessive settlement or slope stability is a problem, the most common (and often most economical) solution is treatment of the soil as it is emplaced during embankment construction. For most Nebraska soils, a minimum

design factor of safety of 1.25 will be required against slope or bearing failure.

### **6.1a Settlement**

Embankments constructed over the top of certain types of deposits often experience settlement that varies in magnitude and the length of time required to reach equilibrium. Laboratory tests conducted on undisturbed samples can be evaluated to determine the amount of settlement expected and the period of time that settlement will be a problem. Many treatment methods are available at various costs and durations. The design engineer must compare the economics of each method of treatment while considering the time required to achieve primary settlement. Some of the most commonly used methods of mitigating settlement include:

- Removal and replacement of the soil displaying excessive settlement. Removal/replacement of materials is generally not economical when the depth of removal exceeds 10 ft (3 meters).
- Placing a load, usually in the form of a temporary fill above the compressible layer to accelerate settlement. This is known as preloading.
- Use of sand or wick drains in conjunction with preloading or the planned embankment to accelerate settlement.
- Use of instrumentation and time delays when the settlement problem is located adjacent to structural foundations. Bridge approaches are often constructed as the last sections of pavement constructed to allow additional time for consolidation at these critical locations. Instrumentation is installed and used to monitor settlement, so that construction of the pavement is delayed only until primary settlement is completed.
- Vibrocompaction, if granular materials are the foundation soils are the cause of the settlement.
- Dynamic compaction of material used to densify the compressible foundation soils by dropping a weight from a specified height in a regular pattern over the soil surface.

The rate of settlement depends upon the hydraulic conductivity and thickness of the consolidating layer, the shape and length of the drainage pattern, and the magnitude of excess pore water pressure. An economical option to decrease the rate of settlement is adding a surcharge (typically 5 to 8 feet) of fill to the finished subgrade elevation of the embankment. The additional load from the surcharge often can increase the total amount of settlement but also decreases the time it takes for the settlement that is expected for the embankment without the surcharge. After settlement completion the surcharge is removed. It is important that slope stability analysis be

performed for the embankment with the surcharge to determine if there is a possibility for a safety factor reduction below minimum requirements.

Duration of settlement for an embankment can be significantly reduced through use of a sand blanket to relieve pore water pressure. A sand blanket is a horizontal layer of clean, granular material, not less than 24 inches (600 mm) thick. The sand layer is placed directly upon the original ground surface. The drainage blanket acts as a pervious foundation over which the embankment is constructed. The edges of the drainage blanket may be left exposed and allowed to drain freely, or PVC drainpipes may be spaced within the drainage layer to provide free drainage.

Depending upon the height of embankment, providing drainage pathways for excessive pore water pressure through use of vertical or inclined sand or wick drains may also shorten the duration of consolidation. Design of a successful drainage system requires a detailed subsurface analysis, careful design and meticulous installation of the drains. The nature of the substrata and its influence on drainage must also be considered.

#### **6.1b Stability**

Techniques used to improve slope stability of an embankment include some of the same techniques used to preclude settlement. Slope stability improvement techniques often include one or more of the following actions:

- Removal and replacement of the unsuitable material.
  - Use of a soil berm, usually at the base or sometimes embankment midslope.
  - Installation of some type of a drainage system.
  - Installation of some type of structural support system, such as a retaining wall, soil nails, or micropiles.
  - Soil reinforcement consisting of a geogrid or geotextiles used at the base of or within the embankment fill.
  - Construction of a shear key or toe key at the toe of slope.
- **Reinforcement.**

Construction of a reinforced soil slope should be considered when there is insufficient Row-of-Way for a stable embankment slope. The reinforced soil slope should be evaluated for both internal and external stability and be designed to have an acceptable magnitude of settlement. Special provisions may be necessary to mitigate surface erosion, as the reinforced slope will normally be steeper than adjacent natural slopes composed of similar material.

Proprietary systems or a system designed by a geotechnical engineer for the specific location can be used. If a proprietary system is used, initial plans should be completed showing both line and grade drawings. A special provision should be added to the specifications providing reinforced slope requirements and a list of approved proprietary systems that meet those requirements. The manufacturer of the proprietary system or the contractor installing the system should submit detailed shop drawings and stability analysis for review and approval by NDOR engineers.

If a geotechnical engineer completes a unique design based upon a specific location, his/her design recommendations should include surface treatment, required properties of fill soil, compaction specifications, surface slope angle, specifications for geosynthetic materials recommended, locations and spacing of geosynthetic materials and soil layer thicknesses.

o **Cut Slopes.**

There are several analysis methods available to evaluate the stability of a cut slope. For most types of soils found in Nebraska, a minimum design factor of safety of 1.5, based upon laboratory tests of undisturbed samples, is required. A higher FOS is required for cut slopes (than for embankments), as cut slopes normally weather more adversely when exposed to surface drainage conditions.

Flattening the slope or improving drainage are the principle methods used to improve the stability of slopes that are too steep. Flattening of a slope is commonly accomplished by construction of one or more benches. Benches should be at least ten feet (3 meters) wide to allow tractor mowing. Ground water seepage through the face of a cut slope is normal but it may result in slope failure when the rate of discharge is inadequate to relieve pore water pressure. Seepage also removes fine materials from the slope face resulting in surface instability. Perhaps the most effective way of dealing with groundwater seepage is to extract the water at a higher level up the slope using some form of subsurface drainage system. Interceptor trenches or trench drains can be used to intercept water higher on the slope, rendering the face of a slope more stable.

Constrained rights-of-way may require unique solutions to ensure cut slope stability. Problems in specific locations may be mitigated through the use of various types of retaining walls, sheeting, or by construction of specially designed soil berms.

**6.1c Surface Compaction Methods and Procedures.**

Construction of a fill section consists of two distinct operations, placing and spreading of material to create each layer and the subsequent compaction

process. Compaction is normally the more critical of the two steps and its rate often controls the rate of progress for the entire job. The use of proper and adequate compaction equipment is a matter of economic necessity for the contractor. Various types of specialized equipment have been developed specially for use by the construction industry. Some equipment has been designed for compacting particular types of soil, while other types of equipment are suitable for use on more than one type of soil.

Sheep's foot rollers and other rollers with projecting feet compact by a combination of tamping and kneading action. These compactors consist of a steel drum with small projections welded onto the outside. For most rollers, the drum can be filled with water or sand to increase the weight of compaction. Roller weight is imposed primarily upon the projections, resulting in high compaction pressures in the range of 100-600 psi (700-4200 kN/m<sup>2</sup>), depending upon the size of the roller.

When loose soil is compacted, the drum projections penetrate into the layer and compact the soil near the bottom of the layer first. In subsequent passes, the roller projections sink into the layer less and less, indicative of the fact that the zone being compacted continually is rising upward. Rising of the roller projections through the layer with each pass is referred to as the compactor "walking out" of the lift. The depth of layer that can be compacted is related to the length of the drum projections and the compactor weight. Large, heavy units can compact lifts ranging upward to one foot in thickness in three to five passes, while smaller, lighter units can compact lifts only six inches in thickness for the same number of passes. Sheep's foot rollers are well suited for compacting clay and silt-clay soils. They are not recommended for cohesionless soils, because the projections continuously disturb the surface being compacted.

Pneumatic tire rollers compact by kneading soil between the tires. The number of tires per axle may vary from two to six or more. Some types of pneumatic tire rollers have bent axles for a "wobbly-wheel" effect, resulting in a weaving path being followed by each wheel. Pneumatic tire rollers are normally equipped with a weight or ballast box, to allow easy variation of the roller's weight. These rollers are available in a wide variety of sizes and weights, the most common being 50-ton (450-kN) rollers.

Pneumatic tire rollers are the best equipment for general compaction use. They are capable of compacting both cohesive and cohesionless soils. Lighter rollers (20 tons or 20,000 kg) are generally capable of compacting lifts up to six inches in thickness in three to five passes, while equipment in the 40-50 ton range (40,000-50,000 kg) is capable of compacting layers up to twelve inches in thickness in three to five passes. Pneumatic tire compaction is not limited to specific compaction equipment. Other rubber-tired



equipment (graders, trucks and scrapers) are capable of providing effective compaction, especially under emergency conditions.

Vibratory compactors are available in a wide variety of configurations, including vibrating drum and vibrating pneumatic tire compactors. Vibrating drum equipment has a separate motor that powers a series of eccentric weights, resulting in a high frequency, low amplitude, up and down movement of the drum. Both sheep's foot and smooth drums models are available. A vibratory pneumatic tire compactor has a separate vibratory unit attached to the axle, so that the wheels vibrate while the ballast is not affected. Both types of roller are generally available as either towed or self-propelled equipment.

Many vibratory compactors have a dash control that allows the operator to vary the vibrating frequency. Frequencies available generally range from 1500-2500 cycles per minute. Most soils are composed of particles that oscillate in unison within the above frequency range, allowing repeated impacts from the compactor's weight to shake them into a denser configuration. Vibratory compactors achieve best results when operated at speeds of 2-4 mph (3-6 km/hr). Smooth drum vibratos are effective when compacting granular soils. With little or no silt or clay, a lift thickness of 3 ft (1m) can be compacted to near maximum modified Proctor value. As the percentage of fine material increases, the thickness of layer being compacted must be reduced. Vibratory pneumatic wheel rollers have also been successfully used to compact granular soils, but the maximum lift thickness for effective compaction is limited to about 1 ft (0.3 m).

Conventional (non-vibratory) smooth drum rollers are not well suited for compacting soil because the size of the drum and large contact area result in relatively low compaction pressure. Smooth drum rollers can be used to seal the surface of project at the end of each workday. Sealing provides a smooth surface allowing rainwater to run off, as opposed to rainwater soaking into the upper layers and creating a soft working surface for the next day. Table 10 provides a generalized summary that relates soil types to the characteristics of equipment considered suitable for achieving compaction.

**Table 10 – Recommended Compaction Equipment Based Upon Soil Type**

Soil Description	USCS Class	Recommended Equipment
Sand, sand-gravel mix	SW, SP, GW, GP	Vibratory drum, vibratory pneumatic tire or pneumatic tire equipment
Sand, or sand-gravel with silt	SM, GM	Same as above
Sand or sand-gravel with clay	SC, GC	Pneumatic tire, vibratory rubber tire or vibratory sheep's foot
Silt	ML	Same as above
	MH	Pneumatic tire, vibratory rubber tire, vibratory sheep's foot or sheep's foot
Clay	CL, CH	Pneumatic tire, sheep's foot, vibratory sheep's foot and rubber tire

Surface compaction methods and equipment previously discussed have the capacity to improve soil characteristics only at shallow depth. Techniques have been developed that utilize special equipment to achieve in-situ improvement in the engineering characteristics of very thick layers of soil. These techniques include vibrocompaction, vibroreplacement and dynamic deep compaction. Deep ground treatment techniques may offer practical, economically viable alternatives to the construction of deep foundations.

Vibrocompaction techniques are best when used for compacting thick deposits of loose, granular soil. A cylindrical vibrator is suspended from a crane and jetted to near the bottom of the layer to be compacted. The vibrator is then activated, causing soil to compact in the horizontal direction. The vibrator continues to vibrate as it is slowly lifted to the soil surface. To improve a roadbed over very loose granular material, treatment locations may have to be spaced as closely as ten feet (3 m) apart. Some equipment incorporates water jets directly into the vibrator to assist in penetration and densification of the granular material.

Vibroreplacement works much the same way as vibrocompaction, except crushed stone or gravel is added to the top of the column and vibrated into the soil. This technique works well on cohesive soils as well as on granular

soils. The introduced stone mixes with the in-situ soil only in the area subject to vibration, creating a column partially supported by the stone. This type of stone column can provide a bearing capacity up to 40 tons. The overall capacity of a site treated by vibroreplacement will depend upon the spacing of the stone columns and the bearing capacity of the material beneath the columns.

Dynamic deep compaction is a method in which a heavy (2-50 ton) weight is dropped from a relatively great height 30-150 ft (10-45 m). The weight and height utilized is dependent upon the equipment available and the depth of soil requiring improvement. A closely spaced grid pattern is commonly laid out on the soil surface and multiple drops are scheduled at each location. This process can be used successfully with most types of soils building problem soils consisting of building rubble and buried garbage fills. A depression is created at each drop location that must be filled in and compacted using normal surface compaction methods.

The intent of deep compaction is to improve marginal surface deposit that already exists at the site to obtain a capacity adequate for roadways or other relatively light surface loads. If used successfully, dynamic deep compaction precludes constructing deep foundations or removal and replacement of a significant thickness of surface material.

#### **6.1d Pile Installation**

Pile installation involves furnishing, driving, trimming, and testing bearing of various types of piles. Specifications for this type of work can be found in Section 703 of NDOR's *Standard Specifications for Highway Construction*. The selection of pile type usually depends on the foundation soil conditions. For NDOR the common practice is:

- H-piles are used for fine grain, clay, or on the rock conditions.
- Concrete pile are used in sands.
- Pipe piles are used in sands and clays.

#### **○ Pile Capacity Field Determination**

Capacity of piles driven through certain types of soil can be directly related to the resistance to penetration developed in the pile during driving. This relationship holds only for soils that do not develop high pore water pressure as piles are driven, including free-draining granular soils and stiff clays. In cohesive saturated soils, high pore water pressures develop because of soil displacement caused by driving. The predicted capacity based upon such a soil's resistance to pile penetration is very different from the capacity developed after the excess pore water pressure dissipates.

Since field personnel must be aware of when driving can be stopped, pile capacity is often expressed as number of blows to drive a pile a specified distance (one foot is common). Driven pile foundations require that records be maintained for every pile installed. An example of a completed pile driving record is shown in Figure 17. The size and driven length of each pile should be recorded, as well as the number of inches per hammer blow needed to drive the pile the last ten blows.

PILE RECORD (English)

PROJECT NO. STPN-TMT-11-2(114)		TYPE AND SIZE OF PILE: Pipe, 12.75x0.375		PILE UNIT WT 49.56 Lbs.		STRUCTURE NO. S011 00919								
PROJECT NAME Wood River West		TYPE OF HAMMER: Delmag D12-42				STATION: 178+14								
COUNTY: Hall		WEIGHT OF RAM: 2820 Lbs.		ABUTMENT, GRADE BEAM, PIER OR BENT NO. A2										
CONTRACTOR: A.M. Cohren		WEIGHT OF CAP: 1820 Lbs.		DESCRIPTION OF STRUCTURE: Abutment #2										
PILE DATA											DRIVING DATA			
Pile No.	Date Driven	Order Length (Ft.)	Additional Length (Ft.)	Length of Pile Driven (Ft.)	Length In Place (Ft.)	Pay Length (Ft.)	Pay Cut-Off (Ft.)	Non-Pay Cut-Off (Ft.)	Pay Splices	Pile Top Elevation (Ft.)	Fall of Hammer (Ft.)	Energy (Kips -Ft.)	Average Penetration of Pile (Inches per Blow)	Calculated Bearing Capacity (Kips)
1W	2/28/2012	0	0	0	0	0	0	0	0	1996.08	6.7	18.89	0.575	70.29
2W	2/28/2012	0	0	0	0	0	0	0	0	1996.17	6.7	18.89	0.575	70.29
13W	2/28/2012	0	0	0	0	0	0	0	0	1994.76	6.7	18.89	0.575	70.29
14W	2/28/2012	0	0	0	0	0	0	0	0	1994.67	7	19.74	0.488	79.92
3	4/18/2012	85	0	85	85	85	0	0	0	1995.11	7.8	22.00	0.1	146.67
4	4/18/2012	85	0	85	85	85	0	0	0	1994.93	7.7	21.71	0.175	128.65
5	4/18/2012	85	0	85	85	85	0	0	0	1995.23	7.6	21.43	0.225	118.23
6	4/18/2012	85	0	85	85	85	0	0	0	1995.2	7.4	20.87	0.238	113.12
7	4/18/2012	85	0	85	85	85	0	0	0	1995.26	7.4	20.87	0.25	111.31
8	4/18/2012	85	0	85	85	85	0	0	0	1995.23	7.5	21.15	0.188	122.97
9	4/18/2012	85	0	85	85	85	0	0	0	1995.04	7.5	21.15	0.125	135.36
10	4/18/2012	85	0	85	85	85	0	0	0	1995.04	7.4	20.87	0.175	123.67
11	4/18/2012	85	0	85	85	85	0	0	0	1996.03	7.6	21.43	0.188	124.59
12	4/18/2012	85	0	85	85	85	0	0	0	1996.11	7.8	22.00	0.156	134.15
TOTALS						850	0		0					
Give driving data on at least one pile out of each group in intervals of 5 feet of penetration all the way down.										DESIGN CAPACITY: Wing 25 Kips Per Pile Abutment 165 Kips Per Pile				
PILE DATA										MINIMUM PENETRATION REQUIRED: Wing 40 Feet Abutment 75 Feet				
DRIVING DATA										AVERAGE BEARING CAPACITY: Wing 72.7 Kips Per Pile Abutment 125.9 Kips Per Pile				
										EFFICIENCY: Wing 290.8 Percent Abutment 76.3 Percent				
										Inspected By: Eric Klein DATE: 4/17/2012 Project Manager: Bob Nordhues				
										REMARKS: pile 6 and 7 were left high. Hammer was warmed up on pile #6, 7.8 hammer fall, 0.15" per blow = 135.38 kips. PDA was done on pile #7 first 10 blows-hammer fall 7.5, 0.338" per blow = 100.95 kips. last 10 blows 7.5 hammer fall, 0.188" per blow = 96.90 kips. 1.73 is the factor to be used.				
										NOTE: Pile Records shall be sent to the Office immediately upon completion of the driving for each individual substructure. Make separate report for each Abutment, Pier or Group.				
8	50	5	2	0										
8	50	10	7	0										
8	50	15	12	0										
8	50	20	17	0										
8	50	25	22	0										
8	50	30	27	0										
8	50	35	32	0										
8	50	40	37	0										
8	50	45	42	0										
8	50	50	47	0										
8	85	55	52	6.4	18.05	0.6								65.64
8	85	60	57	6.4	18.05	0.625								64.18
8	85	65	62	6.7	18.89	0.663								64.97
8	85	70	67	6.6	18.61	0.6								67.67
8	85	75	72	6.9	19.46	0.488								78.79
8	85	80	77	7.2	20.30	0.338								96.90

Figure 17 – Example of a Completed Pile Driving Record.

At NDOR, the pile capacity is verified using two methods: 1) CAPWAP, and 2) the Nebraska Modified ENR Formula.

o **CAPWAP**

The CAPWAP method is based on the data from the Pile Driving Analyzer (PDA). Sensors are attached to the pile in question and the PDA measures the dynamic response of a pile during driving. The Case Pile Wave Analysis Program (CAPWAP) takes the signal inputs of force and velocity from driving collected from the PDA and matches the results with the wave theory to obtain results to simulate a static load test.

o **Nebraska Modified ENR Formula**

The Nebraska Modified ENR Formula is based on the correlation between hammer energy and the resistance of the pile based on Nebraska Soil Types. The original Nebraska Modified ENR Formula was updated in 2007 to incorporate the AASHTO LRFD codes and is shown as:

$$P = \frac{4E}{(S+0.5)}$$

Where P is the factored pile capacity in kips, S is the average penetration in inches of the pile per blow for the last ten blows for steam or diesel hammers, and E is the energy (the ram fall distance in feet times the ram weight) in foot-kips. This pile capacity formula is used for both concrete and steel piles in all Nebraska soil types.

o **Pile Load Tests**

Both cohesive and granular soils will have their properties altered by the installation of a driven pile. In cohesive soils, driving a pile will cause remolding and a temporary loss of strength. With time, much of the lost strength will be recovered, so the testing of piles driven in clay should be delayed until several weeks after driving for the results to be valid. Piles driven through granular soils may exhibit a temporary increase in resistance. In the first week after driving, most of this temporary resistance is lost, so load testing on piles driven through granular soils should be postponed at least five days to make the results are valid.

Loads are applied to the piles during a load test using an anchored platform as illustrated in Figure 18. Measurements of applied load can be taken directly from a load cell and/or from a pressure gauge on the jack. A qualified lab should calibrate both the jack and load cell before testing.

Specifications may be written to require that load test results be verified by deformation of a proving ring or a pressure capsule.

Several methods of performing pile load tests are in current use. The most common is probably the slow maintained-load (SM-L) test described in ASTM D 1143. ASTM D 1143 requires that the load be applied in eight equal increments, until twice the intended design load has been applied. Time and settlement data are obtained after application of each load increment. Each increment of load is maintained until the rate of settlement is less than 0.01 inch/hr (2.5 mm/hr) or two hours, whichever occurs first. The final load is maintained for 24 hours, with the settlement being measured at regular intervals.

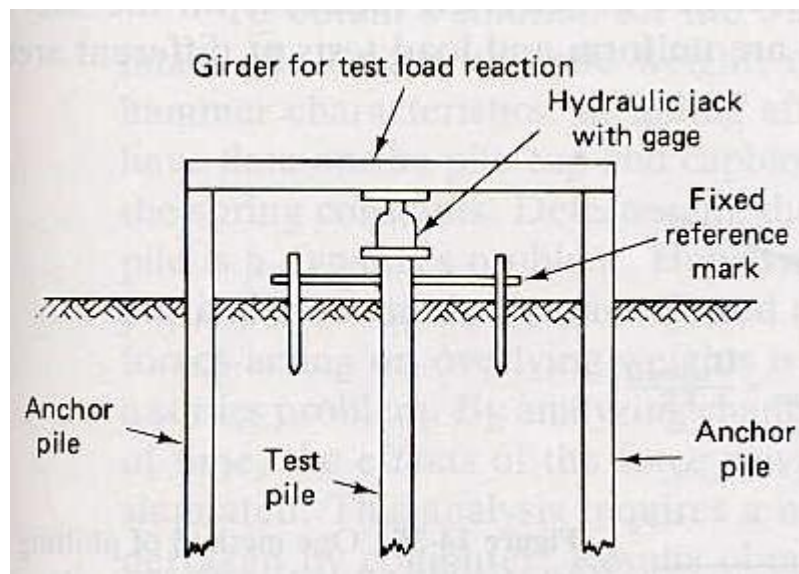


Figure 18 – Anchor piles and Beam Used to Provide Reaction for Testing.

Another method used for load testing piles is the constant rate of penetration (CRP) test. During the CRP test, applied load is varied to force settlement at a specified rate, usually 0.01-0.1 inch/min (0.25-2.5 mm/min), depending upon whether the subsurface conditions are cohesive or granular. Force required to maintain a constant rate of penetration, the depth of penetration and the time is continually recorded, resulting in a load settlement curve similar to that obtained during the SM-L test. Duration of a CRP test is normally 1-4 hours, so it can be conducted in less than one working day.

Other test methods include the cyclic loading and quick maintained load tests. In the cyclic loading test, each increment of a specified load is applied and subsequently removed or reduced. In the quick maintained load test,

load increments are imposed for only a short period of time before the next increment is added (or subtracted). Load applied, time and pile movement for all cycles are recorded, resulting in load settlement curves similar to those obtained from the previously described tests.

Load settlement data is analyzed to determine the design load of a pile. A common method is to apply a factor or safety to the pile failure load. The pile failure load may be designated as the load where the settlement plot becomes no longer linear. Alternative methods of defining failure load include the load where a predetermined amount of settlement has occurred (one-tenth the pile diameter or a specified number of inches is common).

The design factor of safety should be inversely proportional to the information known about subsurface conditions. If soil conditions are uniform and load tests from several different piles are similar, a relatively low factor of safety can be used. If soil conditions and load test results have more variation, a greater factor of safety should be used as protection against unexpected poor soil conditions resulting in lower pile capacity.

#### **6.1e Mechanically Stabilized Earth (MSE) Walls.**

A MSE wall consists of a near vertical face with some type of reinforcement extending through the soil behind. The reinforcement may or may not be connected to the wall face. A MSE wall commonly functions as a retaining wall. A cross-section of a MSE wall is illustrated in Figure 19.

MSE walls can replace many traditional applications of typical gravity retaining walls. MSE walls allow roadways to be built wider with steeper slopes, without having to acquire additional right-of-way. When repairing damage from a landslide, reinforcement placed in the soil during repair may allow slide debris to be used as



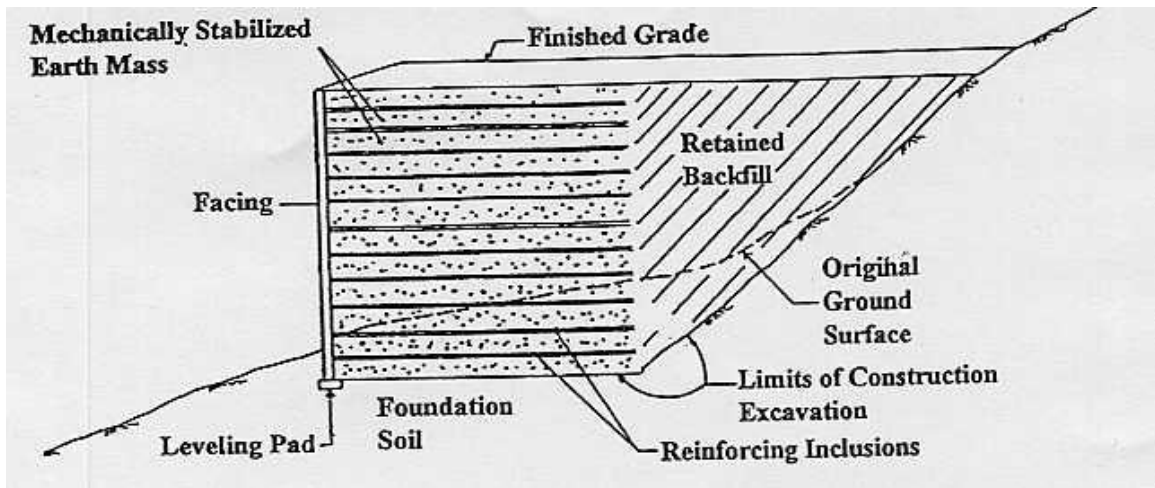


Figure 19 – Cross Section of Mechanically Stabilized Earth Wall.

construction material. Using soil already on site is always less expensive than importing backfill. Many innovative uses of MSE walls have been documented, including use as bridge abutments, wing walls for culverts, and in embankments or excavations where, due to right-of-way restrictions, otherwise stable slopes could not be constructed.

MSE walls are simple and rapid to construct, do not require experienced, skilled craftsmen, reduce right-of-way requirements, require less site preparation than many alternatives, and are relatively insensitive to seismic events. New MSE walls normally require select granular backfill, and often require a large, relatively open area behind the wall for reinforcement to ensure internal and external stability.

Facing elements are the only portion of a MSE wall that is visible. Facing elements provide protection against erosion and sloughing, and often provide a drainage path to prevent water buildup behind the face. The major types of wall facing include precast reinforced concrete panels (in a variety of shapes), modular blocks, welded wire, metallic, wire baskets (gabions) and various geosynthetic materials.

MSE walls can be a very cost effective alternative to reinforced concrete structures. MSE walls offer significant economic and technical advantages over traditional types of retaining walls at sites with poor foundation conditions by completely eliminating the need for pile foundations. A comparison illustrating the costs of various types of retaining walls is shown in Figure 20.

MSE walls for highways currently require select granular materials for backfill. Backfill serves two functions, providing drainage for the soil mass behind the face and providing lateral resistance between the backfill and the

soil reinforcing material. Most MSE wall systems depend upon friction between the reinforcing elements and the backfill to generate lateral force to hold the wall in place.

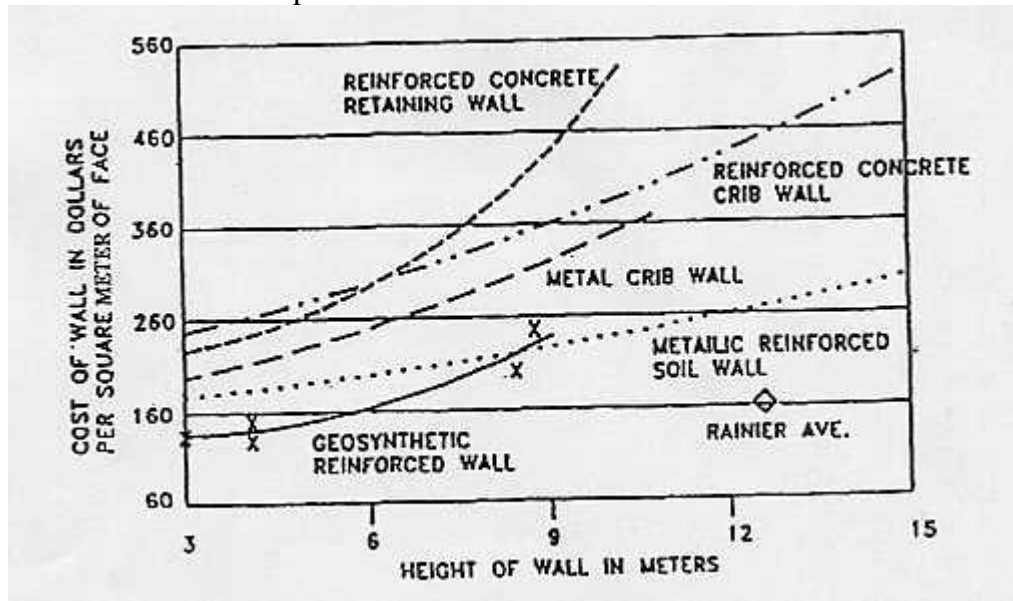


Figure 20 – Cost of Various Types of MSE Walls.

Lower quality backfill could possibly be used, but the engineering characteristics of the lower quality material needs to be taken into account in the design. Granular material offers excellent drainage characteristics, which provide increased life to the reinforcing elements, especially when the reinforcing elements are primarily metallic. The methods used to construct and compact granular backfill also increase the speed of wall construction and decrease variations in alignment as the wall face is constructed upward.

The construction sequence for a MSE wall starts with site preparation. A non-structural leveling pad is then constructed for the wall face, followed by placing the first row of facing panels on the leveling pad. Backfill is then placed on the subgrade up to the level of the first layer of reinforcement. This backfill is compacted and the first layer of reinforcing elements is placed on the compacted backfill. The second layer of backfill is then placed over the reinforcing elements and compacted. This process is repeated until the wall reaches its design height. Specifications for both concrete panel and modular block wall facing materials and construction processes are contained within Sections 714 and 715 of NDOR's *Standard Specifications for Highway Construction*.

Reinforced soil slopes (RSS) can be a cost effective alternative to MSE wall construction in instances where transportation cost for suitable backfill is prohibitive. A reinforced soil slope is usually constructed at angles steeper than could otherwise be safely built and maintained for a natural soil slope.

Reinforcement strengthens the tensile properties of the soil, increasing slope stability during both wet and dry conditions. Reinforcement also improves compaction and tensile properties of the soil immediately adjacent to the slope face, thereby decreasing sloughing.

RSS are usually constructed without facing. Slopes constructed without facing or with flexible facing can more easily adapt to distortion caused by settlement, freeze-thaw cycles, and wet-dry cycles. RSS are relatively unaffected by changes in water content, so RSS have some applications when dealing with saturated soils. Some specific applications include preventing sloughing slopes during periods of saturation, various uses in flood control structures, and slope stability for increasing the height of dams.

RSS increase the factor of safety against sliding, allowing steeper than natural slopes, allow repair of landslides using material from the site and decrease right-of-way requirements. However, maintenance operations (mowing) may become more complicated on steeper slopes. The practice of designing and constructing RSS is still evolving and has yet to be standardized.

## **6.2 Instrumentation.**

Field instrumentation is often used in conjunction with major road projects before, during and after construction. During the design phase, field instrumentation can assist engineers by providing data that allows refinement of the final design. An example of instrumentation used during the design phase might include a small, instrumented test embankment constructed before the project begins. Measurements of consolidation in the test embankment assist in prediction of rates of and magnitudes of settlement on the actual project.

On projects where laboratory tests or instrumentation has indicated potential problems with settlement or embankment stability, instrumentation is often installed to monitor conditions during construction. The locations and orientations of all instrumentation should be included in foundation and earthwork plans. Design notes should also specify all provisions for time and consolidation constraints that the contractor needs to consider (i.e. fill material will be compacted to the extent that settlement will not exceed 1 inch (25mm) per 24 hours) before and during construction.

Instrumentation can also be installed to provide information on existing slopes or embankments. Slope indicators placed within an existing slope can provide data useful in determining the rate of slope movement and in designing remediation systems to mitigate slope movement.

Most of the instrumentation described in this chapter has some type of appendages that protrude above ground level on the construction site. These

appendages are particularly susceptible to damage by construction equipment working at the site. Pieces, parts or cables protruding above ground should be clearly marked in a conspicuous manner so that each is visible to construction personnel. The project manager should ensure that all contractors and subcontractors are aware of this equipment and its importance to the project.

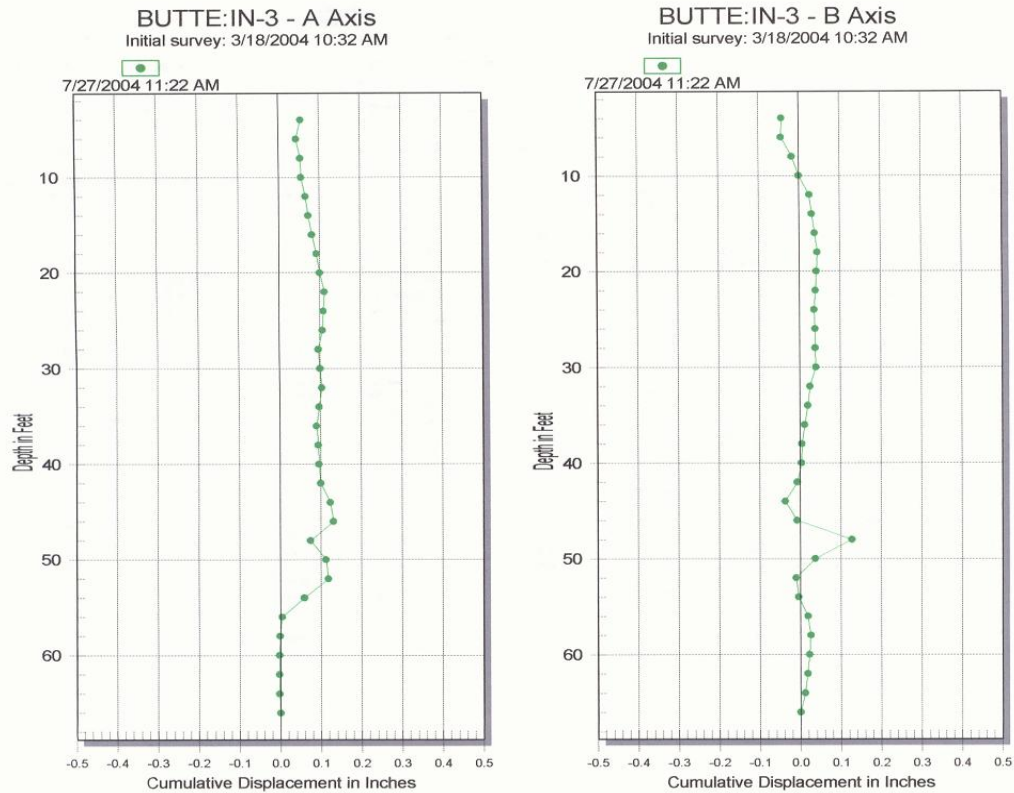
### **6.2a      Inclometers (Slope Indicators).**

Inclometers are used to monitor the stability of an embankment or slope. The inclinometer casing consists of a grooved metal or plastic tube that is inserted down a borehole. The casing should be inserted to sufficient depth to penetrate all layers in which stability problems are anticipated. The bottom of the casing is commonly anchored in rock, concrete or other dense material so that it remains at a fixed location. A probe is lowered down the casing and readings that measure the horizontal deflection of the casing are taken at fixed depths. Successive readings taken over a period of time provide a chronological record of horizontal deformation in the inclinometer casing as a function of depth (see Figure 21).

When inserting an inclinometer casing, space sometimes exists between the borehole wall and the casing. This space is normally filled with gravel, sand or firm grout. If compressible soils are being used for embankment construction, telescoping couplings are available which prevent damage to the inclinometer casing as the soil consolidates.

Casings must be installed so that the grooved channels are as close to vertical as possible. Spiraling of the casing will result in the grooves at depth being oriented differently from the grooves at the surface. Excessive spiraling of the casing will require a spiral-checking sensor and a computerized data reduction routine to provide meaningful data.

Inclinometer casings are normally placed at or near the toe of a slope to monitor stability as a high embankment is constructed. Readings should be taken frequently during embankment construction. Fill operations should be halted immediately if a sudden increase in rate of slope movement is detected.



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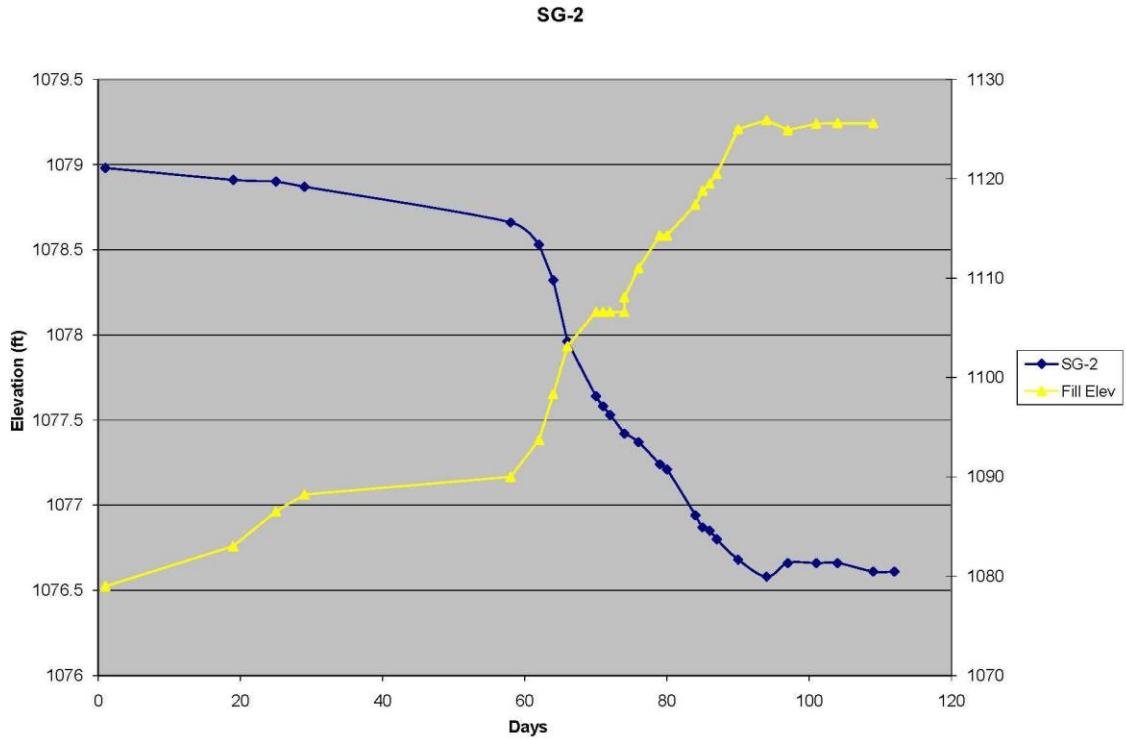
**Figure 21 – Inclinometer plot.**

**6.2b Settlement Plates.**

The simplest form of settlement indicator is a steel or wooden plate placed to the ground or attached to a horizontal structural surface. A reference rod with or without a protective cover is attached to or placed upon the plate. As construction progresses, additional rods and protective covers can be added as necessary. Settlement is measured with surveying instruments by precisely determining the elevation at the top of the settlement plate (or at the top of the reference rod). In addition, the elevation of the fill is also surveyed near the settlement instrument location to track the total fill height as settlement is occurring. Elevations are determined with respect to multiple benchmarks that are located outside the construction zone.

Settlement plates are normally placed at those points on a project where maximum settlement is anticipated. Multiple settlement plates are common on larger projects. An initial reading of plate elevation should be recorded before construction begins. All subsequent readings will be compared to the initial reading to determine magnitude of settlement. Readings should be taken at regular intervals during actual construction. After construction has been complete, readings can be taken at a reduced frequency unless problems

are indicated. Settlement data is normally plotted as a function of time (see Figure 22). Settlement data is analyzed to determine when the rate of settlement has diminished to the extent that construction of pavement or other structures can begin.



**Figure 22 – Settlement Plot**

**6.2c Piezometers.**

Piezometers measure the magnitude of water pressure within the pore spaces of a soil. The magnitude of pore water pressure that will begin to significantly degrade the engineering properties of a soil can be calculated before construction begins. Monitoring soil conditions with piezometers allows construction to be halted or slowed before soil failure due to buildup of excessive pore water pressure.

During project construction, piezometers are used to evaluate increases in pore water pressure resulting from construction activities. Piezometers are normally checked frequently during construction of embankments. If pore water pressure rises at unexpected rates, construction is normally halted until the excess pore water pressure has time to dissipate. Once construction has been completed, pore water pressure can be checked less frequently. Piezometer readings after construction has been completed are used to

evaluate the dissipation of pore water pressure over time, which is directly related to the rate of soil consolidation.

The simplest type of piezometer consists of an open standpipe extending through the fill. Since open standpipe piezometers may experience a significant time lag in registering changes in pore water pressure, this type has largely been replaced by pneumatic, vibrating wire or electrical piezometers. Pneumatic piezometers are used primarily to monitor static water levels, while vibrating wire and electrical piezometers are more commonly used to measure changes in water pressure.

Piezometers (other than the standpipe type) consist of a body containing a flexible diaphragm installed over a pressure sensitive device. The sensor is installed at the location where water pressure is to be measured. Tubes or wires commonly attach the sensor to a readout unit, and in some instances to a data logger, which provides a continuous record of pore water pressure.

Piezometers are normally installed prior to or during construction at any location where excess pore water pressure may develop into a problem. Piezometers may be placed at various depths within the same project depending upon the thickness of the layers involved, the loads anticipated and the construction activities scheduled.

#### **6.2d Monitoring Wells**

Monitoring wells are used to monitor groundwater levels typically in the design phase of a project or during construction. It consists of a perforated section of pipe attached to a riser pipe. The perforated section of pipe is located within the expected water-bearing portion of the soil or formation and is backfilled with clean free-draining sand. The clean sand is capped with a bentonite layer and then backfilled with cuttings to the surface.

Monitoring wells are often used in proposed wetland projects or for special applications where the water table can have an adverse impact on construction. Monitoring wells shall be installed and abandoned by a Licensed Well Drilling Contractor in the State of Nebraska according to Nebraska Department of Health and Human Services regulations.

## Chapter 7

### Analysis, Design and Report Format

#### 7.1 Introduction

Upon completion of the subsurface exploration and laboratory testing, the Geotechnical Engineer will organize, perform analysis, and provide design recommendations. The extent of the analysis is dependent on the scope of the project and the soil conditions.

This chapter discusses the factors that must be considered during the analysis and design phase and typical methods for solving possible project problems. Guidelines for suggested analysis are provided from FHWA and are shown in Table 11. Any computer software used for analysis shall be stated in the Geotechnical Engineering Report.

It is the responsibility of the Geotechnical Engineer to keep updated as new standards, methods, and technology in the engineering and construction field progresses. The suggested methods and references provide guidance for typical geotechnical situations with many other methods and options possible.

#### 7.1a Roadway Embankment

Soil Survey explorations are used to determine the suitability of the existing materials for use as roadway embankment according to Division 200 “Earthwork” of the Standard Specifications for Highway Construction.

The soil types encountered shall be classified according to both the Nebraska Modified Group Index (Chapter 4) and USCS. Any problematic soils shall be delineated both horizontally and vertically. Recommendations for soil types, shallow groundwater table, or other conditions that may adversely affect the pavement performance and constructability of the project shall be provided. Examples of the Soil Survey’s Soil & Situation Report and the Subgrade Survey & Situation Report are provided in Appendixes C and D, respectively. Typical recommendations for problematic soils include:

- o Excavation and replacement with controlled earth fill.
- o Stabilization with lime, cement, fly ash, ckd.
- o Stabilization with geosynthetics.
- o Moisture conditioning of soil (i.e. wetting or drying)
- o Installation of drainage system.

Other situations that may become visible during a Soil Survey exploration include expansive soils, rocky soils, springs, and frost-susceptible soils, etc.



The observation and affect of these properties on roadway performance should be stated and addressed.

## **7.1b Embankment Settlement/Stability**

### **o Settlement**

Settlement calculations shall be based on the results of consolidation tests performed on high-quality undisturbed samples. Design procedures should follow that of:

- o Soils and Foundations Workshop Manual – Cheney, Richard, S., Chassie, Ronald, G., FHWA-NHI-00-045, 2000
- o Foundation Analysis and Design – Fifth Edition, Bowles, Joseph, E., New York: McGraw-Hill Companies, 1996.

The results of consolidation curves should be plotted on a time-settlement curve included in the report. The total settlement estimate should be based on primary consolidation. The period of time for the settlement to occur (paving delay period) should be based on all but 0.5 inches of remaining of the total primary consolidation. The extent of the paving delay period shall be delineated in the Geotechnical report (i.e. station to station). If based on project criteria, the amount and/or paving delay period is excessive, a method of dealing with the problem must be addressed. There also may be a need to design and monitor a field instrumentation program. Possible solutions to excessive settlement and/or paving delays are:

- o Provide a waiting period to allow for a majority of settlement to occur.
- o Reduce fill height.
- o Use of fill surcharge.
- o Use of lightweight fill
- o Excavate compressible material and replace with granular or controlled earth fill.
- o Install wick drain system.
- o Design or recommend ground modification such as stone columns, deep soil mixing, etc.
- o Combinations of some of above.

## **7.1c Stability**

Stability analysis is performed based on results from in-situ strength tests and laboratory strength tests on high quality undisturbed samples. Both undrained and undrained triaxial tests shall be used to consider both short-term and long-term embankment stability. LRFD slope stability analysis

shall be based on a resistance factor of 0.75 for slopes supporting or affecting traffic. For slopes supporting structures a resistance factor of 0.65 shall be used in accordance with the current AASHTO LRFD Bridge Design Specifications. Design procedures should follow that of:

- o Soils and Foundations Workshop Manual – Cheney, Richard, S., Chassie, Ronald, G., FHWA-NHI-00-045, 2000
- o Advanced Technology for Slope Stability: Volume I, Slope Stability Manual – Abramson, L., Boyce, Lee, T., Sharma, S., -FHWA-SA-94-005, 1994.

The soil resistance shall be calculated for possible slope conditions (i.e. surcharge loading, equipment loading, varying water tables, etc.) for the service limit state. The geotechnical engineer shall design a method of dealing with potential stability issues and may also need to design and monitor a field instrumentation program. Possible solutions to slope stability remediation include:

- o Highway realignment.
- o Reduce fill height.
- o Flatten slope.
- o Staged construction allowing for weak soils to gain strength through consolidation.
- o Excavate and replace weak soils.
- o Use of berms at toe.
- o Construct shear key or toe key.
- o Use of lightweight fill.
- o Ground modification such as stone columns, deep soil mixing, etc.
- o Install underdrain system to minimize water infiltration into soils.
- o Proper drainage at the top of the slope, diverting surface water from the slope face.
- o Combinations of the above.

#### **7.1d Retaining Wall Design**

All retaining walls; including gravity walls, cantilever walls, mechanically stabilized earth (MSE) walls, soil anchor walls, and soil nail walls; shall be designed in accordance with the current AASHTO LRFD Bridge Design Specifications with sufficient soil resistance to bearing, sliding, overturning, and global stability. Internal stability for MSE walls shall be the responsibility of the Contractor's designer.

- o **Gravity Walls**

Design procedures should follow that of:

- o Foundations and Earth Structures – NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command, 1986.
- o Principles of Foundation Engineering – Sixth Edition, DAS, Braja, M., Thomson, 2007.

- o **MSE Walls**

Design procedures should follow that of:

- o AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, current edition.
- o Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, Elias, Victor, Christopher, Barry R., and Berg, Ryan R., FHWA-NHI-00-043, 2000.
- o NDOR Standard Specifications for Highway Construction, Sections 714 and 715, current edition.

NDOR maintains a list of approved proprietary mse wall systems on the Approved Product List. The Geotechnical Engineer is responsible for determining the total and differential settlement and the external stability (bearing, sliding, overturning, and global stability) for the proposed wall to ensure it is suitable for the existing foundation soils. Roadway plans provide the wall geometry and foundation soil information to allow the wall system vendor to design the proposed wall. Refer to Sections 714 & 715 of NDOR Standard Specifications for Highway Construction for procedures on design of walls.

- o **Cantilever Retaining Walls**

Design procedures should follow that of:

- o AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, current edition.
- o Foundations and Earth Structures – NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command, 1986.
- o Principles of Foundation Engineering – Sixth Edition, DAS, Braja, M., Thomson, 2007.

Although these wall types are typically more robust than mse walls, it is very important that settlement, especially differential, is minimized. It is also critical to have proper drainage and backfill requirements behind the wall.

- o **Soil Nail Walls**

Design procedures should follow that of:

- o Soil Nail Walls, Lazarte, C., Elias, V., Espinoza, D., Sabatini, P., FHWA GEC7, FHWA-IF-03-017, 2003.

The geotechnical engineer shall provide foundation soil properties for the design of the soil nail walls. Roadway plans shall provide wall location and geometry.

- o **Soldier Pile Walls**

Design procedures should follow that of:

- o AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, current edition.
- o Foundations and Earth Structures – NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command, 1986.
- o Foundation Analysis and Design – Fifth Edition, Bowles, Joseph, E., New York: McGraw-Hill Companies, 1996.

Soldier pile and lagging walls typically consist of steel H-piles and horizontal lagging and are primarily used for top-down construction. Soldier piles may be cantilevered or anchored. Soldier Pile walls can be considered at locations where sheet pile walls are needed, but difficulty in sheet pile installation is expected.

## **7.1e Foundation Types**

Most foundation types in Nebraska consist of spread footings, driven piles, and drilled shafts, with driven piles and drilled shafts being the predominant foundation type used for bridges. Design capacity for these foundations shall be based on SPT, and/or cone penetration tests, laboratory and/or in-situ strength tests, and consolidation tests. Consideration should be given to additional field tests when variable soil conditions are encountered. All foundations shall be designed based on the most current edition of the AASHTO LRFD Bridge Design Specifications.

- o **Spread Footings**

The use of spread footings is typically controlled by the proposed loads, the depth to adequate bearing material, and the potential for settlement. Design procedures should follow that of:

- o Soils and Foundations Workshop Manual – Cheney, Richard, S., Chassie, Ronald, G., FHWA-NHI-00-045, 2000
- o Foundations and Earth Structures – NAVFAC DM-7.2, Department of the Navy, Naval Facilities Engineering Command, 1986.
- o Spread Footings for Highway Bridges– Haley & Aldrich, Inc., FHWA-RD-86-185, 1987.

Varying depths of footings should be considered to achieve maximum efficiency of design. Scour depth shall be the controlling factor at water crossings and may exclude spread footings. The total and differential settlement, along with the rate of settlement shall be addressed. Recommendations regarding difficult conditions such as dewatering and foundation soil preparation should be provided.

- o **Driven Piles**

Driven piles shall be designed for axial and lateral loading conditions as applicable. The following types of driven piles are considered acceptable for supporting structural loads on permanent NDOR structures:

- o Steel H-piles
- o Concrete piles
- o Steel Pipe piles
- o Other pile types may be considered

Design procedures should follow that of:

- o AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, current edition.
- o Soils and Foundations Workshop Manual – Cheney, Richard, S., Chassie, Ronald, G., FHWA-NHI-00-045, 2000
- o Design and Construction of Driven Pile Foundations Volumes I & II, FHWA-NHI-05-042 and 043, 2006.

Different pile types and sizes should be analyzed to obtain the most efficient design. Depth of scour must be considered for both axial and lateral load analysis at water crossings. Pile group effects, punching shear in thin bearing layers, settlement and downdrag shall be addressed as applicable.

o **Drilled Shafts**

Design procedures should follow that of:

- o AASHTO LRFD Bridge Design Specifications, American Association of State Highway and Transportation Officials, current edition.
- o Drilled Shafts: Construction Procedures and LRFD Design Methods, Brown, D., Turner, J. and Castelli, R., – FHWA-NHI-10-016, FHWA GEC 10, 2010

Various drilled shaft sizes should be analyzed to obtain the most efficient design. At water crossings, the depth of scour shall be considered. The method of construction (dry, slurry, or casing) should be addressed, as these methods will affect the side friction and end bearing values assumed during design. Any additional anticipated construction problems (i.e. boulders, hard drilling, etc.) should be made known. Crosshole Sonic Log (CSL) testing shall be utilized to evaluate the integrity of drilled shaft foundations. To verify the design capacity and construction methods, a load test on a test shaft may be specified.

**Table 11, Geotechnical Engineering Analysis Required in Reference for Embankments, Cut Slopes, Structure Foundations and Retaining Walls**

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)						
Unified	AASHTO <sup>1</sup>	Soil Type	Slope Stability <sup>2</sup> Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis					
GW	A-1-a	GRAVEL Well-graded	Generally not required if cut or fill slope is 1.5H to 1V or flatter, and underdrains are used to draw down the water table in a cut slope.  Erosion of slopes may be a problem for SW or SM soils.	Generally not required except possibly for SC soils.	Required for spread footings, pile or drilled shaft foundations.  Spread footings generally adequate except possibly for SC soils	Generally not needed except for SC soils or for large, heavy structures.  Empirical correlations with SPT values usually used to estimate settlement	GW, SP, SW & SP soils generally suitable for backfill behind or in retaining or reinforced soil walls.  GM, GC, SM & SC soils generally suitable if have less than 15% fines. Lateral earth pressure analysis required using soil angle of internal friction.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base.  External slope stability considerations same as previously given for cut slopes & embankments.					
GP	A-1-a	GRAVEL Poorly-graded											
GM	A-1-b	GRAVEL Silty											
GC	A-2-6 A-2-7	GRAVEL Clayey											
SW	A-1-b	SAND Well-graded											
SP	A-3	SAND Poorly-graded											
SM	A-2-4 A-2-5	SAND Silty											
SC	A-2-6 A-2-7	SAND Clayey											
ML	A-4	SILT Inorganic silt Sandy							Required unless non-plastic. Erosion of slopes may be a problem.	Required unless non-plastic.	Required. Spread footing generally adequate.	Required. Can use SPT values if non-plastic.	These soils are not recommended for use directly behind or in retaining or reinforced soil walls.
CL	A-6	CLAY Inorganic Lean Clay							Required	Required			
OL	A-4	SILT Organic	Required	Required									

**Table 11 Continued**

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO <sup>1</sup>	Soil Type	Slope Stability <sup>2</sup> Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
MH	A-5	SILT Inorganic	Required. Erosion of slopes may be a problem.	Required.	Required.	Required.	These soils are not recommended for use directly behind or in retaining walls.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base.
CH	A-7	CLAY Inorganic Fat Clay	Required.	Required.	Deep foundation generally required unless soil has been preloaded.	Consolidation test data needed to estimate settlement amount and time.		
OH	A-7	CLAY Organic	Required.	Required.				
PT	----	PEAT Muck	Required.	Required. Long term settlement can be significant	Deep foundation required unless peat excavated and replaced.	Highly compressible and not suitable for foundation support	External slope stability considerations same as previously given for cut slopes & embankments	
Rock			Fills – not required for slopes 1.5H to 1V or flatter. Cuts – required but depends on spacing, orientation and strength of discontinuities and durability of rock		Required for spread footings or drilled shafts. Empirically related to RQD <sup>3</sup>	Required where rock is badly weathered or closely fractured (low RQD). May require in situ test such as pressuremeter.		Required. Use rock backfill angle of internal friction.
<p>REMARKS:</p> <p>Soils – temporary ground water control may be needed for foundation excavations in GW through SM soils.</p> <p>Backfill specifications for reinforced soil walls using metal reinforcements should meet the following requirements in insure use of non-corrosive backfill: pH range = 5 to 10; Resistivity &gt; 3000 ohm-cm; Chlorides &lt; 100 ppm; Sulfates &lt; 200 ppm; Organic content 1% maximum</p> <p>Rock – Durability of shales (siltstone, claystone, mudstone, etc.) to be used in fills should be checked. Non-durable shales should be embanked as soils, i.e., placed in maximum 0.3 m (1 ft) loose lifts and compacted with heavy sheepsfoot or grid rollers.</p>								

**7.2a Geotechnical Report Format**

The information from the subsurface investigation and analysis shall be organized in a report format that is clear and easy to follow. The Geotechnical Engineering Report shall serve as the permanent record of all the geotechnical data known during the design of the project and will be referenced throughout the design, construction, and service life of the project.

NDOR recognizes the Geotechnical Consultant’s need to conform to company and/or industry standards when preparing a Geotechnical Report. Conversely, the NDOR has two standard report formats that are used to efficiently convey Geotechnical information within the department. These formats will be referred to as an executive summary and soil and situation report. A Geotechnical Engineering report, executive summary, and soil and situation report shall be provided as a result of the subsurface exploration, laboratory testing and geotechnical analysis.

The field and laboratory data that shall be included in the geotechnical engineering report is as follows:

- o Discussion of geotechnical analysis.
- o Geotechnical recommendations.
- o Boring logs.
- o Tabular form of boring logs (*Soil Survey*) including location, depth, sample identification, similar to sample number, USCS symbol, soil

- description, depth to water, % moisture, % retained #200 sieve, plasticity index, group index (Nebraska).
- o Tabular form of lab test data (*Soil Survey*) including lab identification, sample identification, location, depth of material, sieve analysis, % silt, % clay, liquid limit, plasticity index, group index, in-situ moisture content, maximum density, optimum moisture.
- o Density and moisture content of undisturbed soil samples.
- o Unconfined compressive test, triaxial test and consolidation test reports.
- o Soil classification test data.

The Executive Summary Report shall conform to the format presented in Attachment B.

The Soil and Situation Report for the Soil Survey shall contain in order.

- o Description of project location and length.
- o Description of proposed construction.
- o Description of topography, drainage and water table.
- o General description of the soils, pedology and geology.
- o Tabulation summarizing the different soils types showing the range of % retained on the 200 sieve, plasticity index and group index (Nebraska) for each soil type.

Draft reports shall be provided to NDOR for review prior to issuance of final reports.

Monitoring wells are used to monitor groundwater levels typically in the design phase of a project or during construction. It consists of a perforated section of pipe attached to a riser pipe. The perforated section of pipe is located within the expected water-bearing portion of the soil or formation and is backfilled with clean free-draining sand. The clean sand is capped with a bentonite layer and then backfilled with cuttings to the surface.

Monitoring wells are often used in proposed wetland projects or for special applications where the water table can have an adverse impact on construction. Monitoring wells shall be installed and abandoned by a Licensed Well Drilling Contractor in the State of Nebraska according to Nebraska Department of Health and Human Services regulations.

## **7.2b Example of Geotechnical Executive Summary Format**

The following are examples of report templates that are frequently used for common conditions. The intent of this document is to provide a guide in preparing the general style of the Geotechnical Executive Summary. Modifications to the following verbiage will very like be necessary to fit conditions that are encountered.



The Geotechnical Executive Summary is intended to be a separate document from the Consultant's. However, the Geotechnical Executive Summary should make reference to the Geotechnical Engineering Report. The Geotechnical Executive Summary will be submitted to NDOR's Roadway Design Division for their use in designing the project. Therefore, the Geotechnical Executive Summary should present only the specific geotechnical information that the designer will need. A well written Geotechnical Executive Summary and Geotechnical Engineering Report will take into account NDOR's standard specifications, and will not recommend alternative requirement unless necessary.

**Date:**

**To:** Mark Lindemann; Geotechnical Engineer; Materials and Research

**From:** Name, Title; Company (signature) \_\_\_\_\_

**Thru:** Name, Title; Company (signature) \_\_\_\_\_

**Subject:** Final Foundation Report for [Project Name]  
[Project Number; Control Number]

*{briefly describe the project and the areas analyzed and discussed in the report.  
Typically this description is one or two paragraphs}*

*{Example 1: discussion for standard embankment}*

STATION [Approximate Station of Interest (*commonly a bore hole station*)]

The embankment height at [where] will be approximately [##] feet above existing grade. The overall settlement at this location, due to the [##]-foot embankment is [##] inches; however, only [##] inches are expected if the groundwater level remains relatively static. There will be 0.5 inches of settlement remaining [pavement delay time] days after the full height embankment is constructed, if groundwater remains static.

As much as [##] inches of additional settlement could occur after the pavement is placed if the groundwater level rises significantly.

*{If pavement delay time is excessive portions or all of the following may be applicable}*

A [pavement delay time]-day delay period is considered excessive; therefore, surcharge should be used to accelerate the settlement rate and shorten the delay time. Table 1 presents the calculated delay times needed to limit settlement to 0.5 inches.

**Table 1**

Surcharge Height (feet)	Delay Time (days)
0	##
5	##
8	##

Delay time refers to the time required for consolidation settlement of the clayey subsurface materials to occur. Therefore, paving should be postponed for a minimum period corresponding to the desired surcharge height. Delay time starts when the embankment is constructed and the full-height surcharge is placed.

The full-height surcharge should extend from Station [##] to [##], from [##] feet [Right/Left] to [##] feet[Right/Left]. Surcharge side slope grades should not exceed [##] [H] : 1 [V].

Paving should be delayed for the selected time (refer to Table 1) from Station [##] to Station [##].

Table 2 presents the slope-stability safety factor values that have been calculated for the embankment.

**Table 2**

Circumstance	Factor of Safety
Side Slope Stability without Surcharge	##
Side Slope Stability with 5-foot Surcharge	##
Side Slope Stability with 8-foot Surcharge	##

*{Evaluate 8-foot or higher surcharge only if requested}*

The above listed slope stability safety factors are considered acceptable for all of the circumstances presented in Table 2, except for [circumstance] which is less than FHWA’s minimum required value of [##].

*{Describe alternatives to increase slope stability safety factor such as; toe-berms, phased embankment construct, embankment key, over-excavations, etc..., if necessary}*

*{Example 2: discussion for “stable” embankment and MSE Wall}*

STATION [Approximate Station of Interest (commonly a bore hole station)]

The embankment height at [where] will be approximately [##] feet above existing grade. The overall settlement at this location, due to the [##]-foot embankment is [##] inches; however, only [##] inches are expected if the groundwater level remains relatively static.

There will be 0.5 inches of settlement remaining [pavement delay time] days after the full height embankment is constructed, if groundwater remains static.

As much as [##] inches of additional settlement could occur after the pavement is placed if the groundwater level rises significantly.

*{If pavement delay time is excessive add portions or all of the 2 paragraphs and table }*

A [pavement delay time]-day delay period is considered excessive; therefore, surcharge should be used to accelerate the settlement rate and shorten the delay time. Table 3 presents the calculated delay times needed to limit settlement to 0.5 inches.

**Table 3**

Surcharge Height (feet)	Delay Time (days)
0	##
5	##
8	##

*{do not add the following paragraph if previously stated}*

Delay time refers to the time required for consolidation settlement of the clayey subsurface materials to occur. Therefore, paving should be postponed for a minimum period corresponding to the desired surcharge height. Delay time starts when the embankment is constructed and the full-height surcharge is placed.

The full-height surcharge should extend from Station [##] to [##], from [##] feet [Right/Left] to [##] feet[Right/Left]. Surcharge side slope grades should not exceed [##] [H] : 1 [V].

Paving should be delayed for the selected time (refer to Table 3) from Station [##] to Station [##].

Table 4 presents the slope-stability safety factor values that have been calculated for [describe the embankment, MSE wall or approach].

**Table 4**

Circumstance	Factor of Safety
Global Stability of the proposed MSE Wall	##
Global Stability of the proposed MSE Wall with 5-foot Surcharge	##
Global Stability of the proposed MSE Wall with 8-foot Surcharge	##
Circumstance	Factor of Safety
Side Slope Stability without Surcharge	##

Side Slope Stability with 5-foot Surcharge	##
Side Slope Stability with 8-foot Surcharge	##

*{Evaluate 8-foot or higher surcharge only if requested}*

The above listed slope stability safety factors are considered acceptable for all of the circumstances presented in Table 4.

Table 5 presents the allowable bearing pressure ( $q_{\text{allowable}}$ ) cohesion (C) and angle of internal friction ( $\phi$ ) values for the foundation soils that will support the east and west MSE Walls at the proposed Highway 30 overpass structure.

*{The following is an example of a table used for a situation with a clayey soil over sand. The table may need to be modified to apply to the specific site conditions.}*

**Table 5**  
**Foundation Soil Characteristic Values for the MSE Walls**

Foundation Soil Description			Cohesion (psf)	Angle of Internal Friction, (degrees)	Allowable Bearing Capacity* (psf)
Location	Soil Type	Depth			
East MSE Wall	Clayey Soil	Above an Elevation of 1167.0 feet	810	12	4500
	Sandy Soil	Below an Elevation of 1167.0 feet	0	30	7800
West MSE Wall	Clayey Soil	Above an Elevation of 1169.5 feet	620	15	4500
	Sandy Soil	Below an Elevation of 1169.5 feet	0	30	7800

\*Assumes MSE Wall foundation elevations at least 4 feet below finished grade.

*{Example 3: discussion for embankment and MSE Wall with stability insufficiencies}*

STATION [Approximate Station of Interest (commonly a bore hole station)]

The embankment height at [where] will be approximately [##] feet above existing grade. The overall settlement at this location, due to the [##]-foot embankments is [##] inches; however, only [##] inches are expected if the groundwater level remains relatively static. There will be 0.5 inches of settlement remaining [##] days after the full height embankment is constructed, if groundwater remains static.

As much as [##] inches of additional settlement could occur after the pavement is placed if the groundwater level rises significantly.

*{If pavement delay time is excessive add portions or all of the 2 paragraphs and table }*

A [pavement delay time]-day delay period is considered excessive; therefore, surcharge should be used to accelerate the settlement rate and shorten the delay time. Table 6 presents the calculated delay times needed to limit settlement to 0.5 inches.

**Table 6**

Surcharge Height (feet)	Delay Time (days)
0	##
5	##
8	##

*{do not add the following paragraph if previously stated}*

Delay time refers to the time required for consolidation settlement of the clayey subsurface materials to occur. Therefore, paving should be postponed for a minimum period corresponding to the desired surcharge height. Delay time starts when the embankment is constructed and the full-height surcharge is placed.

The full-height surcharge should extend from Station [##] to [##], from [##] feet [Right/Left] to [##] feet[Right/Left]. Surcharge side slope grades should not exceed [##] [H] : 1 [V].

Paving should be delayed for the selected time (refer to Table 6) from Station [##] to Station [##].

Table 7 presents the slope-stability safety factor values that have been calculated for the north approach embankment.

**Table 7**

Circumstance	Factor of Safety
Global Stability of the proposed MSE Wall	##
Global Stability of the proposed MSE Wall with 5-foot Surcharge	##
Global Stability of the proposed MSE Wall with 8-foot Surcharge	##
Side Slope Stability without Surcharge	##
Side Slope Stability with 5-foot Surcharge	##
Side Slope Stability with 8-foot Surcharge	##

*{Evaluate 8-foot or higher surcharge only if requested}*

The above listed slope stability safety factor for the global stability of the proposed MSE Wall is considered acceptable except for [circumstance] which is less than FHWA’s minimum required value of [##].

*{Example of alternatives for a low slope-stability SF for a MSE Wall with surcharge}*

Alternative that will improve the global stability of the MSE wall with 5-foot surcharge include (a) preloading and surcharging the MSE wall area or (b) constructing a temporary toe berm *{and additional options as applicable}*.

- Preloading would consist of constructing the full height embankment at the MSE Wall, and placing the surcharge materials atop them. Following the ## day delay period, the embankment would be excavated to allow for the construction of the MSE wall. The preload end slope shall be no steeper than [##] [H] : 1 [V].
- A temporary toe berm would consist of mounding a ## foot-tall berm of earth that extends full-height a minimum of 25 feet out from the face of the MSE wall. [Describe where toe berm is needed {i.e. The berm must be place in front of those portions of the MSE wall that are taller than 17 feet, but need not extend farther than 30 feet Right or 40 feet Left}]. The toe berm must remain in place until the surcharge materials are removed.

Surcharge should not be placed within [##] feet of the MSE Wall unless one of the above alternatives is implemented.

Table 8 presents the slope-stability safety factor values that have been calculated for the north approach embankment, for the above listed alternatives.

**Table 8**

Circumstance	Factor of Safety
Global Stability of the proposed MSE Wall with 5-foot Surcharge and Toe Berm	##
End Slope <sup>†</sup> Stability of Preload Embankment with 5-foot Surcharge	##
Side Slope Stability with 5-foot Surcharge	##

<sup>†</sup> Based on a 2 [H] : 1 [V] slope

The above listed slope stability safety factors are considered acceptable for all of the circumstances presented in Table 8.

Surcharge should extend from Station ## to ## if no surcharge is placed within 100 feet of the North MSE Wall, and from ## to ## if surcharge is placed behind the North MSE Wall. The surcharge should extend from ## feet Right to ## feet Left.

A ## day delay period will be needed for the portion of the embankment located within ## feet of the MSE wall (from station ## to ##) if no surcharge is place in this area.

Table 9 presents the allowable bearing pressure ( $q_{allowable}$ ) cohesion (C) and angle of internal friction ( $\phi$ ) values for the foundation soils that will support the east and west MSE Walls at the proposed Highway 30 overpass structure.

*{The following is an example of a table used for a situation with a clayey soil over sand. The table may need to be modified to apply to the specific site conditions.}*

**Table 9**  
**Foundation Soil Characteristic Values for the MSE Walls**

Foundation Soil Description			Cohesion (psf)	Angle of Internal Friction, (degrees)	Allowable Bearing Capacity* (psf)
Location	Soil Type	Depth			
East MSE Wall	Clayey Soil	Above an Elevation of 1167.0 feet	810	12	4500
	Sandy Soil	Below an Elevation of 1167.0 feet	0	30	7800
West MSE Wall	Clayey Soil	Above an Elevation of 1169.5 feet	620	15	4500
	Sandy Soil	Below an Elevation of 1169.5 feet	0	30	7800

\*Assumes MSE Wall foundation elevations at least 4 feet below finished grade.

*{Example 4: discussion for embankment and box-culvert extensions }*

STATION [Approximate Station of Interest (commonly a bore hole station)]

##-foot-high embankments will be added along the existing Right and Left fore slopes from Station ## to ##. The existing box culvert at Station ## will be lengthened to extend through the new embankment. The overall settlement of the subgrade beneath the new embankment is [##] inches; however, only [##] inches are expected if the groundwater level remains relatively static. The box-culvert extension points should be made at ## and ## feet Right and Left, respectively, to limit differential to 0.5-inches per 20 feet.

*{Example 5: discussion for 20+ foot embankment with a culver }*

STATION [Approximate Station of Interest (commonly a bore hole station)]

The embankment height at [where] will be approximately [##] feet above existing grade. The overall settlement at this location, due to the [##]-foot embankment is [##] inches; however, only [##] inches are expected if the groundwater level remains relatively static. There will be 0.5 inches of settlement remaining [pavement delay time] days after the full height embankment is constructed, if groundwater remains static.

As much as [##] inches of additional settlement could occur after the pavement is placed if the groundwater level rises significantly.

The calculated settlement profile along the culvert pipe at Station ## is presented in Table 10.

**Table 10**

Right		Left	
Offset	Settlement, (inches)	Offset	Settlement, (inches)
120	##	0	##
100	##	20	##
80	##	40	##
60	##	60	##
40	##	80	##
20	##	100	##
0	##	120	##

**Conclusion**

The calculated maximum settlement under the embankments at [locations] are ##, ## and ## inches, respectively. However, only ##, ## and ## inches of settlement are expected at [where], respectively, if the groundwater level remains relatively static. The remaining settlement is expected to occur if groundwater levels rise significantly.

The ## paving delay period at [where] are considered acceptable. Therefore, alternatives to reduce the delay times have not been provided at this time.

The ##, ## and ## day paving delay periods calculated for [where] embankment are considered excessive; therefore, surcharge is recommended at this location to reduce the delay period (refer to Table #, # and #). Surcharge heights of 5 feet [and 8 feet] will reduce the delay period to ##, ## and ## days respectively. Adequate stability of the [where] approach embankment (with and without) surcharge is expected.

The ##, ## and ## day paving delay period have been calculated for the [where] embankment for “no-surcharge” and “5-foot surcharge” options, respectively. Adequate side-slope stability of the [where] (with and without surcharge) is expected. The global stability of the MSE wall is adequate without surcharge; however, the global stability of the MSE wall is not adequate with surcharge placed directly behind it. Three alternatives have been presented to allow for adequate global stability of the MSE Wall. The alternatives are:



1. Do not place surcharge within 100 feet of the North MSE Wall, and delay paving in this portion for 150 days following the construction of the full height wall and embankment.
2. Preload and surcharge the North MSE Wall area for 90 days to allow for settlement. Then excavate the preloaded embankment area to construct the North MSE Wall.
3. Provide a temporary toe berm in front of the North MSE Wall during the time period when the 5-foot surcharge is in-place behind the wall.

Please contact the [Consultant] via the NDOR Geotechnical Section if you have questions or comments about this transmittal, or if additional information is needed.

## Chapter 8

### Geotechnical Reports & Forms

#### 8.1 Geotechnical Reports and Forms.

Chapter 6 consists of geotechnical reports and forms used to keep accurate and complete records of the progress of work and material tests performed. Blank copies of these forms are available online or from NDOR.

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NDOR Form 12, “Sample Identification Form .....	101
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# Sample Identification Form

Name of Material: Silty Clay Soil for Compaction Tests					
County: Lancaster			Project No.: IM-NH-80-9(845)		
Date Sampled: 2/1/90	Control No.: 12472	Contract No.:	Sample Taken From: Excavation/ Borrow Pit #1		
Material for use in (type of work or kind of structure): Embankment					
Location to be used (Station or other information): Station 153+00 to Station 200+00					
Sampled by: J. Doe		Title: EA-II		Address: Box 2306 Lincoln, NE	
Report to be sent to: D. Jones		Title: Project Manager		Address: " "	
Contractor (Prime): Contractors Inc.		Contractor No.: 123456		Address: Lincoln, NE	
<b>AGGREGATE-SOIL-FILLER — ROAD GRAVEL</b>			<b>CONCRETE CYLINDERS (6x12)</b>		
Field Identification No.	S-1	S-2	S-3	Field Identification No.	
Location (Hole No., etc.)	40'Rt	Station	159+00	Class of Concrete	
Depth of Sample	0 -1.5'	1.5' - 3'	3' - 9'	Water/Cement Ratio	
Depth of Overburden	0	1.5'	3'	Air Voids, Percent	
Thickness of Stratum	topsoil	Subsoil	Peoria	Slump, Inches	
Type or Class				Quantity, Cu.Yds.	
Pit Location	SW¼, Sec.20, T-8N, R-7E			Method of Curing Structure or Pavement	
Kind of Pit: <input checked="" type="checkbox"/> Dry <input type="checkbox"/> Wet	Quantity, Cu.Yd.			Method of Curing Cylinders	
Owner of Pit: Dennis Smit				Days Cured in Field	
Produced By: Soils R Us				Brand and Class of Fly Ash	
<b>ASPHALT AND ASPHALTIC OIL</b>			Location of Power Plant		
Field Identification No.				Brand and Type of Cement	
Type				Location of Mill	
Ticket No.				Air Entraining Agent	
Lot No.				Admixture Name	
Quantity, Gal.				Source of S.G. or Fine Aggregate	
Manufactured By				Pit Location	
Location of Refinery				Source of Coarse Aggregate	
Specific Gravity				Pit Location	
<b>BITUMINOUS AGGREGATE — ASPHALTIC CONCRETE</b>			<b>CEMENT — FLY ASH</b>		
Field Identification No.				Field Identification No.	
Sampled from Sta.				Brand	
Lift, Lane, Dist. from Edge				Type *	
Stations Rep: From				Mill Location	
To				Ready Mix Plant	
Asph. Oil, Gal./Sta. (Bit. Agg.)				Quantity, Tons	
Agg., Tons/Sta. (Bit. Agg.)				Place of Storage	
Type of Asphalt Concrete				Length of Storage	
Job Mix No. (Asph. Conc.)				* Type I, Type II, Type III, Type I-II	
Asph. % (Asph. Conc.)				<b>OTHER MATERIALS</b>	
Type Asph. or Asph. Oil				Field Identification No.	
Mfr. Asph. or Asph. Oil				Kind of Material	
Specific Gravity				Brand	
<b>OTHER INFORMATION</b>			Quantity Represented		
			Size and Weight Specified		
			To Comply with Spec. No.		
			Manufactured by		
			Jobber		
			Submitted by		
<b>TO BE FILLED IN AT LABORATORY</b>			Title		
Date Received at Laboratory			Address		
Laboratory Identification					

Nebraska Department of Roads  
**Field Gradation Tests of Aggregates**  
*(Eliminates the need for this information in the field notebooks)*

Project No.:	F-111(1)
Report No.:	2

Kind of Material: Select Soil				For Use In: Embankment			
Name of Road: Lincoln - Northeast				County: Lancaster			
Pit No.:	Location:	Section:	T 12N N	R 7E	(Town, City, Landmark) 3.5 Miles North of Davey		
514	NW 1/4	6					
Name of Producer: Contractors Inc.				Contractor: Contractors Inc.			
Delivery Point: Station 65+00 to Station 90+00				Sampled From: <i>(Pit, Car, Truck, Window, Stockpile)</i> Windrow			
SPECIFICATIONS		Section:	Subsection:	SPECIAL PROVISIONS		Section:	Subsection:

**MECHANICAL ANALYSIS OF MATERIAL**  
 Spacer sieves *(were)* *(were not)* used

DATE	SAMPLE NO.	WASH OR DRY TEST	DRY WEIGHT OF SAMPLE	TOTAL WEIGHT RETAINED ON SIEVE												QUANTITY OF MATERIAL REPRESENTED BY THIS TEST	STATION
				TOTAL PERCENT RETAINED ON SIEVE * Insert appropriate Sieve Size													
				*	*	3/4	*	3/8	4	10	20	*	30	50	100		
			Weight														
			Percent														
			Weight														
			Percent														
			Weight														
			Percent														
			Weight														
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			Percent														
			Weight														
			Percent														
			Weight														
			Percent														

**SPECIFICATION RANGE**

<b>DISTRIBUTION</b> 1-Materials and Research Division 2-District Engineer 3-Project Manager	Duplicate of Sample Number FS-35 Submitted to Lincoln Laboratory for Correlation.											
	Remarks: <i>(Indicate disposal of material failing Specification Requirement)</i>											
	Inspector:	Approved: <i>(Project Manager)</i>										

Distribution  
Materials and Research Division  
District Engineer  
Project Manager

Nebraska Department of Roads

# Weekly Report of Moisture and Density Tests

(Eliminates the need for this information in the field notebooks)

Project No.:	IM-80-2(121)
Control No.:	54321
Name of Road:	Interstate 80
Report No.:	2
Week Ending:	10-27-10

Type of Work: (embankment, subgrade, foundation course, soil-aggregate base, etc.)  
Subgrade Compaction

### Method of Compaction

Class Type

III

Column Number	1	2	3	4	5	6	7	8	
Date	10-22	10-22	10-22	10-22	10-22	10-22	10-22	10-22	
Sample No.	6	7	8	9	10	11	8A	11A	
Curve No.	E-4	E-4	E-2	E-3	E-5	E-3	E-2	E-3	
Optimum Moisture From Curve	10.3	10.3	9.0	14.9	14.9	14.9	9.0	14.9	
Maximum Density (Dry) from Curve	1.97	1.97	2.02	1.63	1.63	1.63	2.02	1.63	
Required Moisture Limits	Opt -2%	Opt -2%	Opt -2%	Opt -2%	Opt -2%	Opt -2%	Opt -2%	Opt -2%	
Required % of Maximum Dry Density	100	100	100	100	100	100	100	100	
Sample Taken At	Station	574+00	571+00	564+00	554+00	550+50	56000	564+00	560+00
	Feet Right or Left of Centerline	CL	15' Rt	20' Rt	15' Rt	12' Lt	15' Lt	20' Lt	15' Lt
	Depth Below Finish Grade	Grade	Grade	Grade	Grade	Grade	Grade	Grade	Grade
Actual Volume	Final Reading (cc or cu ft)	2400	3050	3325	3325	3600	3325	3450	3400
	Initial Reading (cc or cu ft)	1500	1500	1450	1425	1450	1450	1450	1475
	Difference (cc or cu ft)	900	1550	1875	1900	2150	1875	2000	1925
	Volume of Sample (cc)	255	439	531	538	609	531	566	545
Actual Moisture	Wet Weight (gms)	200	200	200	200	200	200	200	200
	Dry Weight (gms)	180	181	188	181	182	182	188	182
	Loss (gms)	20	19	12	19	18	18	12	18
	Moisture (%)	11.1	10.5	6.4	10.5	9.9	9.9	6.4	9.9
Actual Density	Wet Weight (gms)	579	968	1114	1023	1160	914	1214	1029
	Wet Density (gms/cc)	2.27	2.2	2.10	1.91	1.91	1.72	2.15	1.89
	Dry Density (gms/cc)	2.04	1.99	1.98	1.73	1.74	1.56	2.02	1.72
Maximum Density (% of)	103	101	98	106	107	96	100	105	
Thickness (GFC, SABC, etc.)									

Remarks:

Test #'s 8 & 11 need more Rolling. Test # 8A & 11A are retests of #8 and 11.

Inspector:

Approved: (Manager)

DR Form 86, April 2010

## Nuclear Density Tests for Soils

Field Performed Tests

NDOR M&R  
Omar Qudus, Geotechnical Engineer

Template ID: SLF002001  
Version: 20110215

Nuclear Density Gauge				
Make	Model	Serial Number	Calibration Date	
<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/> // <input type="text"/>	
Method of Compaction				
Class	Type	Test Depth		
<input type="text"/>	<input type="text"/>	<input type="text"/>		
Test Number	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Curve Number	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Optimum Moisture Percentage	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Maximum Dry Density (pcf)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture Limits	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Required Percentage of Maximum Density	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min
Station	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>
Offset (ft)	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>
Direction of Offset	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Depth Below Final Grade (ft)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Density Standard Count	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Percent of Proctor 1	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Percent of Proctor 2	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Average Percentage Density	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture Standard Count	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
% Moisture 1	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
% Moisture 2	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Average Moisture Percentage	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Density Correction Factor Used	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture Correction Factor Used	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
<p><small>Note: A moisture correction is required if lab moisture and nuclear moisture test results differ by more than 1.5%. Refer to the Moisture Calculation template for moisture correction. If a volumetric test is performed and the volumetric and nuclear density test results differ by more than 2.5 pcf, a density correction factor is required. Refer to the Volumetric Balloon Test template for density correction.</small></p>				
Test Status	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Comments:	<input type="text"/>			
	<input type="text"/>			
	<input type="text"/>			
	<input type="text"/>			
Test Method:	AASHTO T310			

### Volumetric Balloon Test

Field Performed Tests

NDOR M&R  
Omar Qudus, Geotechnical Engineer

Template ID: SLF004001  
Version: 20110217

Volumetric Balloon Apparatus				
Make	Model	Serial Number	Calibration Date	
<input type="text"/>	<input type="text"/>	<input type="text"/>	// <input type="text"/>	

Method of Compaction		
Class	Type	Test Depth
<input type="text"/>	<input type="text"/>	<input type="text"/>

Test Number	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Curve Number	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Optimum Moisture Percentage	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Maximum Dry Density (pcf)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture Limits	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min
Required Percentage of Maximum Density	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min	<input type="text"/> min
Station	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>	<input type="text"/> + <input type="text"/>
Offset (ft)	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>	<input type="text"/> ' <input type="text"/>
Direction of Offset	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Depth Below Grade (ft)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Volume					
Initial Reading (cc)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Final Reading (cc)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Volume of Sample (cc)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Mass					
Total Wet Weight (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture					
Sample Wet Weight (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Sample Dry Weight (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Loss (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
% Moisture	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Density					
Wet Density (pcf) (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Dry Density (pcf)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
% of Proctor	0.0	0.0	0.0	0.0	0.0

Note: Before performing a density correction a separate moisture correction of the nuclear gauge shall be performed first (Refer to the Moisture Calculation and Correction for Soils template). A density correction is required if the volumetric and nuclear density test results differ by more than 2.5 pcf and should be noted in the comments section below.

Comments:

Test Method: NDR T205

### Moisture Calculation and Correction for Soils

Field Performed Tests

NDOR M&R  
Omer Qudus, Geotechnical Engineer

Template ID: SLF003001  
Version: 20110215

#### Moisture as Determined by AASHTO T265

Curve Number	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Wet Weight (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Dry Weight (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Loss (g)	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
% Moisture	<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>

#### Moisture as Determined by AASHTO T310

##### Gauge

Make	Model	Serial Number	Calibration Date
<input type="text"/>	<input type="text"/>	<input type="text"/>	<input type="text"/>
Nuclear Gauge	<input type="text"/>	<input type="text"/>	<input type="text"/>
% Moisture	<input type="text"/>	<input type="text"/>	<input type="text"/>
Moisture Correction %	<input type="text"/>	<input type="text"/>	<input type="text"/>

Note: A moisture correction is required if lab moisture and nuclear moisture test results differ by more than 1.5%. Refer to the Moisture Calculation template for moisture correction. If a volumetric test is performed and the volumetric and nuclear density test results differ by more than 2.5 pct, a density correction factor is required. Refer to the Volumetric Balloon Test template for density correction.

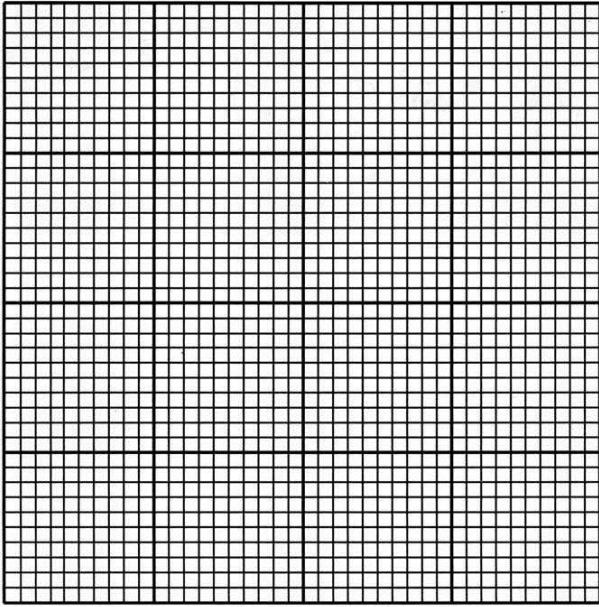
Comments:

Test Method: AASHTO T265, T310



## Moisture-Density Relations of Soils Test Method AASHTO T 99 and T 180

Lab Worksheet

DENSITY DETERMINATION								
Determination No.	1	2	3	4	5	6	7	8
PERCENT MOISTURE ADDED	4	6	8	10	12			
WT. WET SOIL & MOLD, gm	3742	3838	3937	3942	3908			
WT. MOLD NO. 2 gm	2036	2036	2036	2036	2036			
WT. WET SOIL, gm	1706	1802	1901	1906	1872			
VOLUME OF MOLD, cc	940	940	940	940	940			
WET DENSITY, gm/cc	1.815	1.917	2.022	2.028	1.991			
DRY DENSITY, gm/cc	1.610	1.668	1.726	1.701	1.642			
MOISTURE DETERMINATION (TEST METHOD: AASHTO T 265)								
CONTAINER NO.	1	2	3	4	5			
WT. WET SOIL & CONT., gm	348.1	394.3	404.6	394.7	430.9			
WT. DRY SOIL & CONT., gm	318.1	353.8	357.4	344.2	369.9			
WT. OF WATER, gm	30.0	40.5	47.2	50.5	61.0			
WT. OF CONTAINER, gm	82.1	82.1	82.9	81.4	81.7			
WT. OF DRY SOIL, gm	236.0	271.7	274.5	262.8	288.2			
PERCENT MOISTURE	12.7	14.9	17.2	19.2	21.3			
Project: 6-6(126)		<div style="display: flex; align-items: center; justify-content: center;"> <div style="writing-mode: vertical-rl; transform: rotate(180deg); font-size: small; margin-right: 5px;">DRY DENSITY, gm/cc</div>  </div>						
Lab Ident: S95-381	M/D Curve No.: G-119							
Max. Density, gm/cc: 1.73	Max. Density, lbs./ft.: 108.0							
Opt. Moisture, Percent: 17.2								
Operator: Allen P								
Date Tested: 7/27/95	Date Reported: 7/28/95							
Remarks:								
		PERCENT MOISTURE						



Nebraska Department of Roads  
**Field Gradation Tests of  
 Gravel for Surfacing**

(Eliminates the need for this information in the field notebook)

Project No.:	F-2(111)
Report No.:	545329

Name of Road: Hwy 59 M.P. 24.8 1/2 mile west of Jct 121/59				County: Knox			
Pit No.:	Location:	Section:	T	R	Miles	NE of	(Town, City, Landmark)
317	NE 1/4	18	26 N	1W	6		Pierce
Name of Producer: Backus Sand & Gravel				Contractor: Backus Sand & Gravel			
Delivery Point: Stockpile				Sampled From: Stockpile			
SPECIFICATIONS		Section:	Subsection:		SPECIAL PROVISIONS		Section:

**MECHANICAL ANALYSIS OF MATERIAL**  
*Spacer sieves were used.*

DATE	SAMPLE NO.	WASH OR DRY TEST	DRY WEIGHT OF SAMPLE	TOTAL WEIGHT RETAINED ON SIEVE							QUANTITY OF MATERIAL REPRESENTED BY THIS TEST & CAR NUMBER	STATION	
				1"	3/4"	3/8"	#4	#10	#50	#200			
7/2/11	1	Wash	Weight	50			1		37	49	50	50 cu. yd	Stockpile
			Percent			2		74	98	100			
7/2/11	2	Wash	Weight	75			1		57	74	75	50 cu. yd	Stockpile
			Percent			1		76	99	100			
7/2/11	3	Wash	Weight	37			1		28	37	37	50 cu. yd	Stockpile
			Percent			1		76	100	100			
7/2/11	4	Wash	Weight	46			1		33	46	46	50 cu. yd	Stockpile
			Percent			1		72	100	100			

Computation for Deduction =	Sum of % Retained on No. 10 Sieve divided by Number of Tests Performed =	298	% Deduction (From Tables in Specifications)	=
		74.5 = Avg %	x Contract Unit Price \$	=
			\$	per C.Y. for Lot No.

		Weight											
		Percent											
		Weight											
		Percent											
		Weight											
		Percent											

Computation for Deduction =	Sum of % Retained on No. 10 Sieve divided by Number of Tests Performed =		% Deduction (From Tables in Specifications)	=
		= Avg %	x Contract Unit Price \$	=
			\$	per C.Y. for Lot No.

**SPECIFICATION RANGE**

<b>DISTRIBUTION</b> 1 - Materials and Research Division 2 - District Engineer 3 - Project Manager	Duplicate of Sample Number MA-96-2 Submitted to Lincoln Laboratory for Correlation.	
	Remarks: (Indicate Disposal of material failing Specification Requirement)	
	Inspector:	Approved: (Project Manager)

Form RT-811 (Note: This form can be obtained from M & T soils unit.)

### Sample Identification—Local Pit Materials

STATE OF NEBRASKA  
DEPARTMENT OF ROADS

Project No. F-100-4(1001)

AFE No. --

Name of Material Fine Sand Pit No. 514 Sample No. FS-1 thru FS-5

Name of road Lincoln N.W. County Lancaster Date sampled 4-17-89

Location of pit NW Quarter, Section 6 Township 12-N Range 7-E

Ave. dimensions of pit 270 ft. x 225 ft. x 9.0 ft. Quantity represented 20,250 cu. yds.  
Length Width Depth

Wet or dry pit dry Owner of pit J. Roe Ave. thickness of overburden 1.0'

Distance to public road Adj. Length of haul to project Adj. Mi. to Sta 39+50

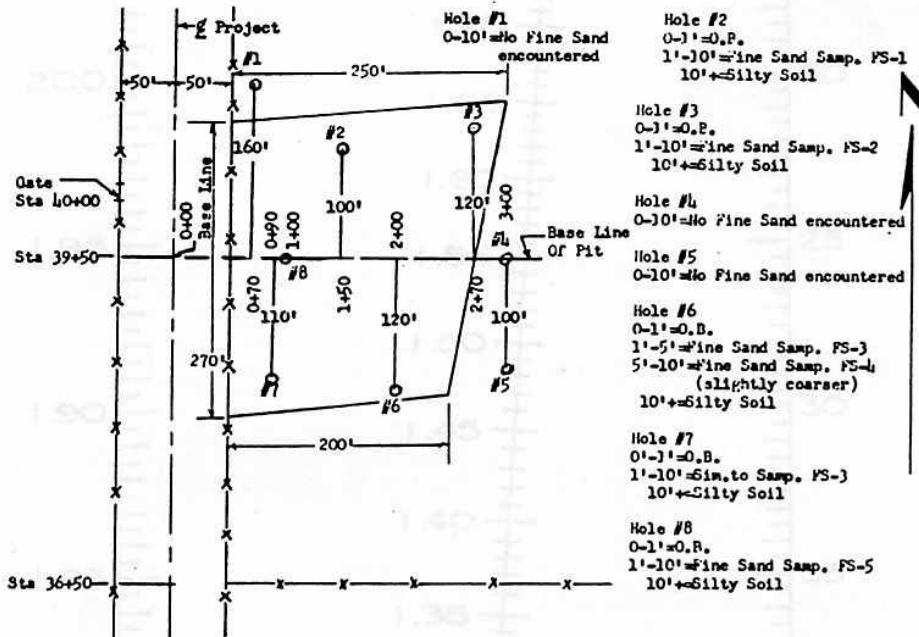
Type of road between pit and project Adj. to Project

Sampled by J. Doe Title E.A. II

Report to be sent to D. Jones Title Sr. Engr. Address Lincoln Ne.

Remarks: Contractor performing Work--Contracts Inc. , Lincoln Ne.

(Include detailed location sketch, showing exact location of pit with respect to section lines, and making adequate reference to roads, fences, trees, buildings, etc. Show location of each boring, thickness of material and overburden in each boring, location and depth of each sample submitted. Show haul road and other pertinent data. In reporting dimensions of pit and quantity represented, show actual estimated values. Do not report "Ample".)



Soil and Situation Report  
 NH-2-2(112)  
 Whitman East  
 C.N. 61008  
 3/10/2004

This Project, a part of highway 2, begins just west of Whitman and extends in an easterly direction, on a new alignment, for a distance of 15.9 miles. The new alignment is approximately 70'-100' south of the existing alignment.

Present plans provide for grading, structures, and surfacing.

The terrain traversed by the project is typical of the sandhills of Nebraska; rough, choppy, hilly sand dunes dissected by low rolling dunes to nearly flat meadows in the valley sections.

Drainage is good due to the sandy soils with the exception of the valley sections. The water table in the valley sections is high and apparent by the ponds and marshes in the low-lying areas.

The majority of the soils on this project are clean dune sands (SP, SP-SM). Silty sands (SM) and clayey sands (SC) are located in the valley sections to a depth of 0.5' to 2.0' and underlain by clean sand. Shown in the following tabulation are some of the more important engineering characteristics of the soil encountered.

<b>SUCS Symbol</b>	<b>Description</b>	<b>% Ret. #200</b>	<b>P.I.</b>	<b>G.I.</b>
SC	Clayey Sand	76	8	0
SM	Silty Sand	82 – 87	NP	-2 – 0
SP-SM	Poorly Graded Sand With Silt	89 – 95	NP	-2
SP	Poorly Graded Sand	96 – 97	NP	-2

Stabilization in the upper subgrade of these fine sands may be necessary to support construction equipment.

Attached to this report are:

1. Compaction Requirements
2. Soil Moisture Density Information

March 11, 2004  
C.N. 61008

### SOIL COMPACTION CURVES

Project No. NH-2-2(112)  
Project Name Whitman East

In order to assist in the density control of the subgrade of embankment on this project; the following soils data is shown for information on compaction samples taken previously on this project or adjacent ones

Originally constructed as Project No. F-80(13)

Year constructed 1956

Curve No.	USCS symbol & Soil description	Location Depth	% Retained			P.I .	G.I.	Max. Density	Opt. Moisture
			#10	#50	#200				
C-100	SP-SM Poorly graded sand w/ silt	0-4.0'	0	2	90	NP	-2	1.71	13.2
C-101	SP-SM Poorly graded sand w/ silt	0-12.0'	0	7	94	NP	-2	1.71	13.2
C-102	SP-SM Poorly graded sand w/ silt	0-12.0'	0	4	92	NP	-2	1.70	14.2

**Nebraska Department of Roads  
Material and Research Division  
Geotechnical Section-Soil Survey Unit  
Tabulation of Soil Survey Borings**

Project Number: RD-14-3(1012)  
Project Name: In Fullerton and South

Control No: 42476  
Date: 5/11/2010

Location	Depth (Ft)	Sample No	Similar To	USCS Symbol	Soil Description	Water Level	% Moist.	% 200	P.I.	G.I.
790+52.50 107 Elev. 1642.509	0.00-2.50		F-104	_SM	Silty sand with clay; 70-85% fine to medium sand; 30-50% silty fines brownish gray, slightly moist	4.73		77	NP	0
	2.50-4.50		F-103	_SM	Silty sand with clay; 85-95% fine to medium sand; 5-15% silty clayey fines ; slightly moist			87	NP	-2
791+99.21 114 Elev. 1644.974	0.00-3.50		F-104	_SM	Silty sand with clay; 70-85% fine to medium sand; 30-50% silty fines brownish gray; slightly moist			77	NP	0
	3.50-4.50		F-102	_ML	Sandy Silt; 30-50% fine sand; slightly plastic; dark brown; medium stiff			41	1	5
	4.50-6.00		F-101	_SM	Silty sand; 50-70% fine sand; 30-50% silty fines brownish Tan			51	NP	3
	6.00-9.00		F-103	_SM	Silty sand with clay; 85-95% fine to medium sand; 5-15% silty clayey fines ; slightly moist			87	NP	-2
793+42.25 108 Elev. 1643.879	0.00-0.50	TS4			Topsail	9.06		41	1	5
	0.50-2.50		F-102	_ML	Sandy Silt; 30-50% fine sand; slightly plastic; dark brown; medium stiff			77	NP	0
	2.50-5.00		F-104	_SM	Silty sand with clay; 70-85% fine to medium sand; 30-50% silty fines brownish gray; slightly moist			87	NP	-2
	5.00-10.00		F-103	_SM	Silty sand with clay; 85-95% fine to medium sand; 5-15% silty clayey fines ; slightly moist			89	NP	-2
	10.00-15.00		F-100	_SP-SM	Poorly graded sand with silt; 85-95% fine to medium sand; 5-15% moist					
795+14.69 112 Elev. 1651.758	0.00-0.50	TS3			Topsail	18.27		77	NP	0
	0.50-10.00		F-104	_SM	Silty sand with clay; 70-85% fine to medium sand; 30-50% silty fines brownish gray, slightly moist			61	NP	1
	10.00-15.00		F-105	_SM	Silty sand with clay; 50-70% fine to medium sand; 30-50% silty fines			77	NP	0
	15.00-19.00		F-104	_SM	Silty sand with silt; 70-85% fine to medium sand; 30-50% silty fines brownish gray; slightly moist					
796+70.72 102 Elev. 1642.384	0.00-0.50	TS2			Topsail			51	NP	3
	0.50-2.00		F-101	_SM	Silty sand; 50-70% fine sand; 30-50% silty fines brownish Tan			41	1	5
	2.00-5.00		F-102	_ML	Sandy Silt; 30-50% fine sand; slightly plastic; dark brown; medium stiff			87	NP	-2
	5.00-11.00		F-103	_SM	Silty sand with clay; 85-95% fine to medium sand; 5-15% silty clayey fines ; slightly moist					
	11.00-14.00		NA							
797+61.29 87 Elev. 1634.640	0.00-1.50	TS1			Topsail	1.75		89	NP	-2
	1.50-4.00		F-100	_SP-SM	Poorly graded sand with silt; 85-95% fine to medium sand; 5-15% moist					

Nebraska Department of Roads  
Material and Research Division  
Geotechnical Section-Soil Survey Unit  
Tabulation of Lab Test Data

Project Number: RD-14-3(1012)

Project Name: In Fullerton and South

Control No: 42476

Date: 5/11/2010

Lab Id	Sample No	Location	Depth (Ft)	Sieve Analysis - Percent Retained											Silt	Clay	L.L.	P.I.	G.I.	M.C.	Max. Density	Opt. Moist.	Moist. Density Curve No.	Organic Content	
				1 1/2	1	3/4	3/8	4	10	20	40	50	100	200											
S10-32	F-100	797+61.29 87.98' LT	1.50-4.00				1	2	4			23	36	71	89	9	2	NP	NP	-2		1.87	11.1	X-25	
S10-33	F-101	796+70.72 102.44 LT	0.50-2.00									3	8	34	51	42	7	NP	NP	3		1.83	12.8	X-26	
S10-34	F-102	796+70.72 102.44 LT	2.00-5.00									1	4	26	41	47	12	21	1	5		1.83	14.2	X-27	
S10-35	F-103	796+70.72 102.44 LT	5.00-11.00									7	19	72	87	9	4	NP	NP	-2		1.81	12.1	X-28	
S10-36	F-104	795+14.69 112.16 LT	0.50-10.00									2	9	50	77	19	4	NP	NP			1.81	12	X-29	
S10-37	F-105	795+14.69 112.16 LT	10.00-15.00									1	5	39	61	32	7	NP	NP	1		1.86	12.2	X-30	



**COMPACTION REQUIREMENTS**

Project No. **XXXXXX** C.N. **XXXX**  
 Project Name: **Compaction Requirements for All**

The following compaction requirements are recommended for the plans.  
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**COMPACTION REQUIREMENTS**  
 Class III (See Specifications)

	<u>SOIL TYPE</u>	<u>DEPTH BELOW FINISH SUBGRADE</u>	<u>PERCENT DENSITY</u>	<u>MOISTURE REQUIREMENT MINIMUM</u>
Embankment / Roadway Grading, including driveways, to receive concrete pavement	Silt-Clay	Upper 3 feet	98 Min.	Opt. -3%
	Silt-Clay	At depths greater than 3 feet	95 Min.	Opt. -3%
	Granular	All depths	100 Min.	**
Embankment / Roadway Grading, including detours, temporary roads, and driveways, to receive flexible pavement	Silt-Clay	Upper 3 feet	100 Min.	Opt. -2%
	Silt-Clay	At depths greater than 3 feet	95 Min.	Opt. -3%
	Granular	All depths	100 Min.	**
Embankment / Roadway Grading to receive gravel surfacing	All	All depths	95 Min.	**
Embankment / Roadway Grading not to be surfaced, and Noise Wall Berm	All	All depths	95 Min.	Opt. -3%
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Surcharge Height =		5'		
Upper		4'	All --	Class II (See Specifications)
Lower		1'	Silt-Clay --	Opt. -2%
			Granular --	**
-----				
Subgrade Preparation, Shoulder Subgrade Preparation (Concrete Pavement)	Silt-Clay	The upper 6 inches of subgrade soil	98 Min.	Opt. -3%
	Granular	The upper 6 inches of subgrade soil	100 Min.	**
Subgrade Preparation, Shoulder Subgrade Preparation (Flexible Pavement)	Silt-Clay	The upper 6 inches of subgrade soil	100 Min.	Opt. -2%
	Granular	The upper 6 inches of subgrade soil	100 Min.	**

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## Appendix A

### ASTM STANDARDS

<u>Subject</u>	<u>Number</u>
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C 127
Guide to Site Characterization for Engineering Design, and Construction Purposes	D 420
Standard Test Method for Particle-Size Analysis of Soils	D 422
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft <sup>3</sup> (600 kN-m/m <sup>3</sup> ))	D 698
Standard Test Method for Specific Gravity Soils	D 854
Standard Practices for Soil Investigation and Sampling by Auger Borings	D 1452
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft <sup>3</sup> (2,700 kN-m/m <sup>3</sup> ))	D 1557
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	D 1587
Standard Practice for Diamond Core Drilling for Site Investigation	D 2113
Standard Test Method for Unconfirmed Compressive Strength of Cohesive Soil	D 2166
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D 2216
Standard Test Method for Density of Soil and Soil-Aggregate In Place by Nuclear Methods (Shallow Depth)	D2292

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Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D 2435
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487
Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488
Standard Test Method for Field Vane Shear Test in Cohesive Soil	D 2573
Standard Test Method for Unconsolidated, Undrained, Compressive Strength of Cohesive Soils in Triaxial Compression	D 2850
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat And Other Organic Soils	D 2974
Standard Test Method for Water Content of Soil and Rock In Place by Nuclear Methods (Shallow Depth)	D 3017
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D 3080
Standard Classification of Soils and Soil-Aggregate Mixtures For Highway Construction Projects	D 3282
Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D 3385
Standard Test Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil	D 3441
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D 4186
Standard Practices for Preserving and Transporting Soil Samples	D 4220
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D 4254
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D 4318

Standard Test Methods for One-Dimensional Swell of Settlement Potential of Cohesive Soils	D 4546
Standard Test Method for Rock Mass Monitoring Using Inclinometers	D 4622
Standard Test Method for Determining the Water (Moisture) Content Of a Soil by the Microwave Oven Method	D 4643
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648
Standard Test Method for Pressuremeter Testing in Soils	D 4719
Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084
Standard Guide for Field Logging of Subsurface Explorations of Soil And Rock	D 5434
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151
Standard Test Method for Determining Dispersive Characteristics Of Clayey Soils by the Crumb Test	D6572

## AASHTO STANDARDS

<u>Subject</u>	<u>Number</u>
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	M 145
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	T 85
Standard Test Method for Particle-Size Analysis of Soils	T 88
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity	T 89
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft <sup>3</sup> (600 kN-m/m <sup>3</sup> ))	T 99
Standard Test Method for Specific Gravity of Soils	T 100
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft <sup>3</sup> (2,700 kN-m/m <sup>3</sup> ))	T 180
Standard Practice for Soil Investigation and Sampling by Auger Borings	T 203
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	T 206
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	T 207
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	T 208
Standard Test Method of Permeability of Granular Soils (Constant Head)	T 215
Standard Test Method for One-Dimensional Consolidation Properties of Soils	T 216
Standard Test Method for Field Vane Shear Test in Cohesive Soil	T 223
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	T 236

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Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	T 251
Pore Pressure	T 252
Standard Test Method for Rock Mass Monitoring Using Inclinedometers	T 254
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	T 258
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	T 265
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	T 267
Resilient Modulus – Soil	T 294
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	T 296
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	T 297

## Appendix B

### SOIL AND SITUATION REPORT

The Soil and Situation Report (SSR) has two primary objectives. The first is to report soil, situation and water table data, while the second is to transmit recommendations from the Materials and Tests Division to the Design and Construction Divisions. The SSR usually consists of four parts, the test or body of the report, a tabulation of soil properties and subgrade borings, tabulations of test results and a diagram or diagrams illustrating the soil profile.

The text of the SSR begins with a statement regarding project location and length, followed by a description of the proposed construction. The next paragraph commonly discusses topography, drainage characteristics and location of the water table. Information concerning soil formations and geology of the project area follows. The soil horizons and soil formations encountered in the project area are discussed in detail. A general description of the soils encountered and a tabulation of some of the more important engineering properties of the soil in each horizon are included. A table concerning recommendations for compaction requirements is normally included either in this section or attached as an enclosure.

If selective handling of excavated materials is recommended, requirements are discussed in detail. Selective handling of excavated material is currently limited to five general cases:

- 1) To produce embankment sections of uniform material (i.e. all silt-clay or sandy materials in an embankment).
- 2) To place materials suitable for use in a Bituminous Sand Base Course in the upper subgrade.
- 3) To place unsuitable materials at depth or in the outer slopes of an embankment.
- 4) To place select materials over heavy clays to reduce moisture problems.
- 5) To use select granular materials in lieu of a foundation course on projects when PCC pavement is the planned surface course.

Selective handling notes are based upon and should always specify the planned project surface.

Larger projects are often divided into cut and fill sections, according to soil types or volumes of material available for cut and fill operations. A discussion is usually included indicating the approximate volume of and properties of soil to be excavated or placed within each section. A single set of recommendations sufficient for the entire project is used whenever possible.

On some projects, soil may be mostly granular on one part of the project while in another part of the same project it is mostly cohesive. The soil report will require granular material in upper embankments through the granular areas, and may or may not require undercutting of cohesive soils to a specified depth and backfilling with granular materials

when cohesive soils are encountered. To produce a uniform embankment, granular materials encountered as layers in predominantly cohesive soils are usually buried or placed near the outer edge of the embankment.

## Appendix C

### SUBGRADE SURVEY/SUBGRADE AND SITUATION REPORT

A subgrade survey is conducted on previously graded roads for which rigid or flexible pavement is being designed. Its principle objectives are:

- 1) To sectionalize the project as to the type of soil in the upper subgrade.
- 2) To locate and explore any portions of the project where the subgrade may be of questionable stability due to seepage, springs, wet zones, etc.
- 3) To evaluate gravel windrow or crust which may have been placed or developed through the use of the project during temporary gravel or clay surfacing.
- 4) To obtain a check on the conditions resulting from the selective placement required by the grading plans.

In making a subgrade survey, holes are drilled to depths of 5 feet (1.5 m) or more into the subgrade. As in a soil survey, samples are not obtained from every borehole. The party chief is responsible for deciding when soil properties have changed sufficiently to require taking another sample. As the survey progresses along a project, locations of soil changes in the subgrade soil are noted and recorded.

Depth to water table should be determined and recorded if the water table is within nine feet (3 m) of the surface. "Frost boils" frequently develop on Nebraska highways in situations where the upper subgrade soil is underlain by a less permeable material at depths of five feet (1.5 m) or less. Springs, visible seepage water and high water tables often occur in these locations. All potential "frost boil" areas should be explored fully and carefully by borings during a subgrade survey. Recommendations for special underdrain systems, selective handling of subgrade soils, and/or extra strength pavement are commonly required at these locations.

In some locations, road gravel has been placed on the surface as the primary wearing course. Gravel surfaced roads may form a gravel crust from the rolling action of traffic combined with periodic mixing by maintenance equipment. The gravel or rock surface course should be mixed with and embedded into the upper layer of subgrade prior to placing rigid or flexible pavement as a surface course.

A "Subgrade and Situation Report" (report of a subgrade survey) is prepared when there is a significant period of time between grading and preparation of the paving plans. When grading and paving are let by the same contract, the design of the base and surface course is based upon information obtained during the soil survey.

A Subgrade and Situation Report is usually brief and contains the following entries as applicable:

- 1) Location.
- 2) Proposed construction.
- 3) Existing surface.
  - a) Year the project was last graded.
  - b) Width to which the project was graded.
  - c) Amount of grade that was left low.
- 4) Foundation course requirements (on PCC projects).
- 5) Topography
- 6) Pedology
- 7) Surface and subsurface drainage.
- 8) Water table (depths of water and dates borings made).
- 9) Compactions recommendations.
- 10) Statement of Attachments (Tabulations of tests, borings, etc.)
- 11) Notes for clay surfacing removal, if necessary.
- 12) Subgrade distress areas, including visible and potential areas and those reported by maintenance.
- 13) Embankment and slope stability areas.
- 14) Summary of subgrade soils by section.

## Appendix D

### NEBRASKA SOIL FORMATIONS

This section is intended to aid soil surveyors and grade inspectors in identifying various soil formations exposed in various parts of Nebraska.

Undisturbed soil has a zonal arrangement of near horizontal layers lying one over the other. These layers are collectively called the *soil profile*. In its simplest form, a soil profile is made up of three distinct layers, topsoil, subsoil, and parent material.

**Topsoil** is usually dark in color and extends from the surface of the ground to a depth of two feet or greater. Its exact nature will vary with the parent material from which it is developed, but it will usually be characterized by its relatively low clay content when compared to the underlying subsoil. The low clay content of topsoil is due to the action of percolating water removing or leaching the fine clay and soluble materials from the top layer.

**Subsoil** can vary in thickness from a few inches to as much as three feet or more. It is characterized by the presence of the additional clay and soluble material, which has been removed by leaching from the topsoil. The subsoil may vary in nature from light clay content when compared to topsoil to a tough and impervious claypan layer. In most soil profiles found in Nebraska, the subsoil contains more clay than the layer above it or below it.

The topsoil and subsoil taken together are known as the *solum*, or zone of weathered material. Below the solum, lies the **parent material** or the geologic formation from which the solum is developing. The parent material is, in turn, underlain by other geologic formations. The various geologic formations are usually different enough from each other that the change is readily recognized when boring or examining open excavations.

Natural processes form soil formations. The nature of the soil profile will vary depending upon the type of parent material, climate, topography, and vegetation in an area, and the length of time during which soil forming processes have been at work. A soil profile sketch (Figure D-1) shows the variation in soil development due to topography. Development of a soil profile depends on the action of percolating water to leach the topsoil and redeposit the leached material in the subsoil. On relatively flat areas where surface runoff is slight, more of the rainfall becomes ground water. On slopes, a large percentage of the water runs off. Consequently, topsoil and subsoil will usually be thicker on flat areas than on the slopes. If slopes are sufficiently steep to carry away the surface water fast enough to cause erosion, the topsoil and subsoil may be removed as fast as they are formed, resulting in parent material exposed at the soil surface.

In some instances, soil will be eroded from the slopes and redeposited along terraces and on stream bottoms in layers many feet thick. As these deposits are of recent origin and material is constantly being added, a soil of considerable thickness may be deposited in which no zonal arrangement or profile can be discerned. Such materials are commonly encountered in stream channels and are referred to as alluvial or colluvial deposits.

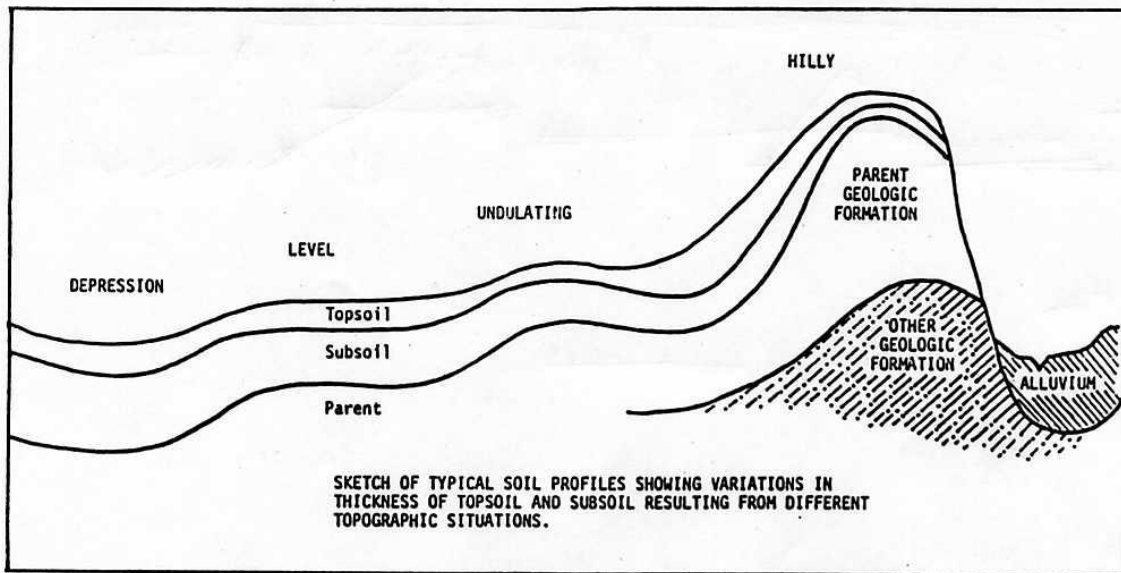


Figure D-1 – Variations in Soil Development Due to Topography.

The various soil layers can be distinguished in the field by visual inspection of the material and by feeling its texture and structure. The ability to judge a soil by feeling it and breaking it down between the fingers is very helpful for the soil surveyor and grading inspector and is quite easily developed. The texture (*relative quantities of fine granular materials versus fine material in the soil*) is easily determined by touch. Grains of sand and gravel feel rough between the fingers and can be seen as individual grains with the naked eye.

The relative quantities of the fine materials (*clay and silt*) may be determined by breaking up the material between the fingers. Material rich in clay, when wet, is tough, highly plastic and sticky. When pinched between the thumb and finger, it will form a thin, flexible ribbon. When kneaded in the hand, it does not crumble readily but tends to work into a compact mass. When dry, such material forms hard clods and small aggregations, which are hard to break up. Material low in clay, on the other hand, when wet is soft and difficult to mold between the fingers as it is continually breaking apart. When dry, material low in clay easily breaks down into a fine powder with a floury feel. Color is sometimes important to recognizing soil formations but should only be used in conjunction with other “feel and see” tests.

A brief description of various types of parent material and soil formations that are commonly encountered on Nebraska road construction projects follow. These descriptions are based upon characteristics commonly associated with the type soil. Mechanical and chemical weathering may have significantly altered many of the distinguishing characteristics of these materials and formations.

## **Soil Formations of Recent Age**

**ALLUVIUM:** Water deposited material in stream floodplains. Zones of development may be missing. Local variations in texture are denoted by Oa for Sand, Ob for Silt, and Oc for Clay.

**TOPSOIL:** Surface soil that supports vegetation. Topsoil is usually composed of sand, silt, and clay and is dark colored.

**BURIED TOPSOIL:** Remains of one-time surface soil buried beneath later deposits.

**REDEPOSITED TOPSOIL:** Topsoil accumulated on terraces or bottomlands as colluvium, then washed down by sheet erosion from adjacent uplands.

**SUBSOIL:** Soil formation layer resulting from the infiltration and accumulation of fines leached from the overlying topsoil.

**CLAYPAN:** Subsurface condition characterized by the development of a dense impervious clay layer.

**BURIED SUBSOIL:** Clay subsoil formed during a previous geologic age and now buried under later deposits.

**REDEPOSITED SUBSOIL:** Subsoil that has been eroded from its original position and redeposited at a lower elevation.

## **Formations of Pleistocene Age**

**PEORIAN LOESS:** Prevalent type of parent soil material in Nebraska. Wind deposited Pleistocene silt-clay materials that blanket much of eastern, central, and southwestern Nebraska. Exposed slopes in loess have a tendency to stand in near vertical faces. The color is light brown varying to tan or light buff.

**SANDY PEORIAN:** Loess mixed with sand found in areas transitional between the Nebraska Sandhills and typical Peorian loess mantle in east, south and southwestern Nebraska.

**SAND LENSES IN PEORIAN:** Very fine sand in thin beds that occasionally occurs in Peorian loess deposits.

**REDEPOSITED PEORIAN:** Loess that has eroded out of position; often found in talus at the toe of exposed loess slopes. The characteristic of loess to stand in vertical slopes is lost when loess has been redeposited.

**LOVELAND LOESS:** Loess deposit older than Peorian having a distinguishing reddish tint; it is usually heavier textured than Peorian loess. A buried weathering surface occasionally occurs at



the contact between Loveland and Peorian loesses. This is often seen in roadway cuts where the two are exposed in contact with one another.

**SANDY LOVELAND LOESS:** Textural phase of the Loveland Loess.

**REDEPOSITED LOVELAND:** Loveland loess that has slumped out of its original position.

**UPLAND FORMATION:** Greenish-gray silts, clays and sandy marl, sometimes intermixed with volcanic ash, usually found immediately above the Grand Island formation.

**FULLERTON FORMATION:** Gray silt-clay material usually found between the Grand Island and Holdrege formation.

**GLACIAL TILL:** Largely heavy clay soil with intermixed sand, rocks, and silt. It varies widely in color and may contain some pebbles. No distinction for NDOR purposes is made between the Kansas and Nebraskan till.

**GLACIAL GRAVEL:** Mixed sand, gravel, and boulders transported to their current location by glaciers.

**GLACIAL SAND:** Local sand deposits associated with glacial till.

**FINE SAND AND NATURAL SAND:** Wind-blown dune sands covering the Sandhill area of Nebraska and water deposited fine sands, wherever they may occur.

**GRAND ISLAND AND HOLDREGE SAND OR GRAVEL:** Sand-gravel materials from which the bulk of the road gravels are pumped in the valleys of the Platte, Blue, and Republican Rivers. The two are usually separated by the Fullerton formation. They also underlie, at considerable depth, the upland plain extending south from the Platte Valley to the Republican Valley. This formation is the source of most of the water for irrigation wells south of the Platte River and west of the town of Seward.

### **Formations of Tertiary Age**

**KIMBALL FORMATION:** Pinkish to gray partly cemented silt, clay, and fine sand capped by gray algal limestone beds.

**SIDNEY GRAVEL:** Sheet-like complex of channel or basin gravel deposits not widely persistent, occurring between the Kimball and Ash Hollow formations.

**ASH HOLLOW, VALENTINE, BOX BUTTE, SAND CANYON, SPOTTED TAIL, AND MARSLAND FORMATIONS:** Soft sandstone with interbedded sandy clay and irregularly cemented mortar beds. Concretions are generally missing.

**HARRISON, MONROE CREEK AND GERING FORMATIONS:** Distinguished from Ash Hollow by the Prevalence of “pipe” concretions, much clean fine sand and channel deposits.

**REDEPOSITED CONCRETIONS:** Transported concretions and coarse material making up coarse gravel deposits. Source of Class “D” limerock.

**BRULE CLAY:** A massive compact pinkish silty clay, occasionally imbedded thin with layers of volcanic ash.

**REDEPOSITED BRULE:** Slumped and weathered Brule Formation. It is loose and friable, very similar to loess in appearance and characteristics.

**CHADRON:** Greenish to buff colored clay, silt, and sandy clay. This material often weathers into a plastic, “gumbo-like” soil. Usually encountered only in the Hat Creek Basin, which is north of the Pine Ridge escarpment and in the extreme western part of the North Platte Valley.

### **Formations of Cretaceous Age**

**PIERRE SHALE:** Dark gray massive clay, containing some chalk, bentonite, thin sandstones and some concretions. It is a very plastic clay soil and is a very poor subgrade material since it absorbs water readily and changes volume dramatically when wet.

**NIOBRARA CHALK:** Lead gray to yellowish buff, massive to thin beds of chalk with some imbedded shales. It is a very poor subgrade material since it absorbs water readily and is very unstable when wet.

**CARLILE SHALE:** Gray shales containing a layer of fine-grained sandstone. It is not widespread at depths where it would be commonly encountered in Nebraska Highway construction.

**GREENHOUSE LIMESTONE:** Thin, medium soft gray limestones interbedded with gray shales. The presence of many oyster shell-like fossils marks the upper portions and makes it easy to identify.

**GRANEROS SHALE:** Dark gray plastic shale with some thin calcareous layers, sandy and sandy shale, and coal-like materials.

**DAKOTA SANDSTONE AND DAKOTA SHALES:** Mainly of importance as a source of fine sand, this sand varies from loose clean fine or slightly coarse sand to highly cemented sandstone and “ironstone” requiring blasting or ripping for removal. The Dakota Shales are usually interbedded with the sands and are fine-grained silty clay shales which generally have high swell characteristics and are detrimental subgrade materials. They usually have a glossy or soapy appearance and are multicolored.

### **Formations of Permian-Pennsylvanian Age**

PERMIAN-PENNSYLVANIAN: Limes and Shales. No distinction is usually made between the Permian and Pennsylvanian. The limestone usually exists as ledges with clay layers beneath. Shale beds are usually thicker than limestone beds. Exposures are limited to the southeastern portion of the state.

## Appendix E

### COMPACTION

Soil is used in greater quantities for construction of roads than any other material. All pavement and roadway structures depend upon soil for support. Without suitable design specifications, even the most carefully planned and constructed embankments, bridges and pavements are prone to failure.

This appendix has been prepared for use as a reference and instructional guide for contractors and inspectors working with compaction and grading projects across Nebraska. The following sources contain additional information and specifications applicable to compaction and grading operations. These sources are available online at <http://doroads.nol.org/ref-man/>:

#### *Standard Specifications for Highway Construction*

- Division 200 – Earthwork
- Division 300 – Subgrade Preparation, Foundation Courses, Bases Courses, Shoulder Construction and Grade Resurfacing
- Division 700 - Bridges, Culverts and Related Construction
- Division 800 – Roadside Development and Erosion Control
- Division 900 – Incidental Construction

#### *Supplemental Specifications for Highway Construction*

- Division 200 – Earthwork
- Division 300 – Subgrade Preparation, Foundation Courses, Bases Courses, Shoulder Construction and Grade Resurfacing
- Division 700 - Bridges, Culverts and Related Construction
- Division 800 – Roadside Development and Erosion Control
- Division 900 – Incidental Construction
- Division 1000 – Materials Details

#### *2002 Construction Manual*

- Division 200 – Earthwork
- Division 700 - Bridges, Culverts and Related Construction
- Division 900 – Incidental Construction
- Division 1000 – Materials Details

In the appendix entitled NDOR Forms, the following examples of forms are available in pdf format:

- DR 8 – Water Applied Haul Sheet
- DR 23 – Moisture Density Relationships of Soils
- DR 64 – Site Release
- DR 86 – Weekly Report of Moisture and Density Tests  
(or computer printout)
- DR 99 – Earthwork Computations

DR 101 – Truck Capacity Computations  
DR 210 – Moisture Density Test  
DR 232 – Final Status Material and Site Releases  
DR 264 – Field Gradation Tests of Gravel  
DR 309A, B, C – Contractor’s Estimate  
(Fuel Adjustment Computations)  
DR 234 – Source of Aggregate to be Used  
DR 348 – Material Pit Contract Release  
DR 478 – Nuclear Density Record

## Compaction

One characteristic of soil that is important to highway construction is its ability to support loads without excessive deformation or displacement. The load carrying capability of most soils is reduced as moisture content increases. The ability of a soil to support imposed loads also varies with soil density. To support maximum loads, most soils should be compacted as dry and dense as possible. However, excessive amounts of work are required to attain high densities in very dry soils. Detrimental amounts of swell are also more likely to occur in a soil that has been compacted under very dry conditions. An understanding of the relationship between moisture content, soil density, load carrying capacity and compaction effort is necessary if soil is to be properly emplaced in embankments.

Research indicates that increased weight of rollers is more effective in obtaining higher soil densities than requiring additional passes by a smaller roller. The relationship between soil density and compaction effort at constant moisture content is shown in Figure E-1. This figure shows the relationship between dry density and number of blows with a 5.5-pound hammer dropping twelve inches (standard Proctor) and a 10-pound hammer dropping eighteen inches (modified Proctor). Note the increase in soil density obtained from the larger hammer remains approximately constant across the entire range of number of hammer blows.

Typical moisture density curves for constant compaction effort are shown in Figure E-2. To obtain data from which to plot these curves, several identical samples were compacting into a mold at different moisture contents varying from 12-21%. The same amount of compaction effort was used on each sample (25 blows per layer for each of three layers by a 5.5 lb hammer dropped twelve inches). The weight of the wet compacted soil was divided by the soil volume to obtain the moist unit weight for each measured moisture content. The dry density curve was obtained by dividing the wet density by one plus the decimal moisture content. The equations are summarized below:

Wet density = wet unit weight/volume

Dry density = wet unit weight/1 + decimal moisture content

Where moisture content (%) = ((weight wet-weight dry) /weight dry)\*100%

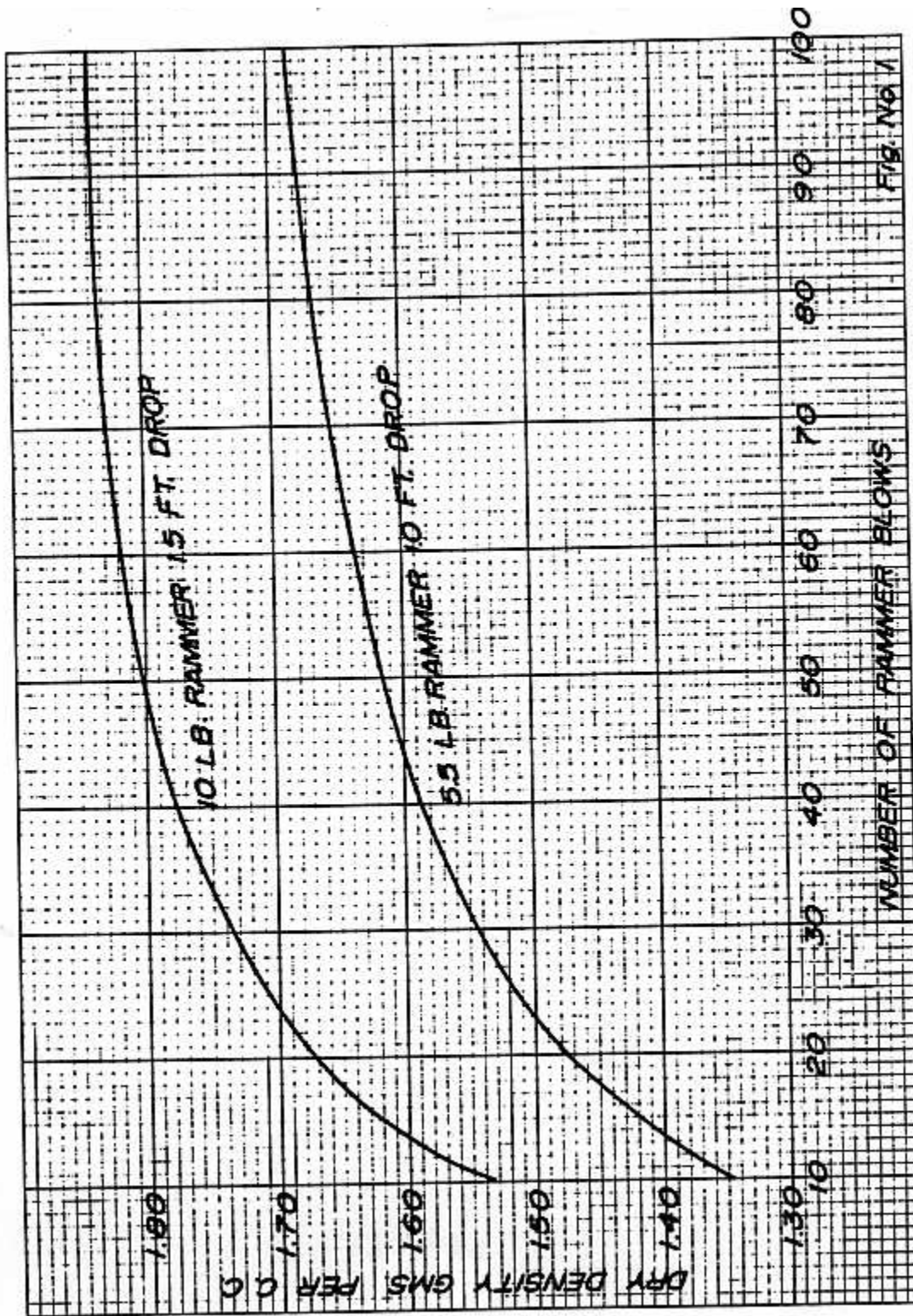


Figure E-1 – Relationship between Compaction Effort and Soil Density at Constant Moisture Content.

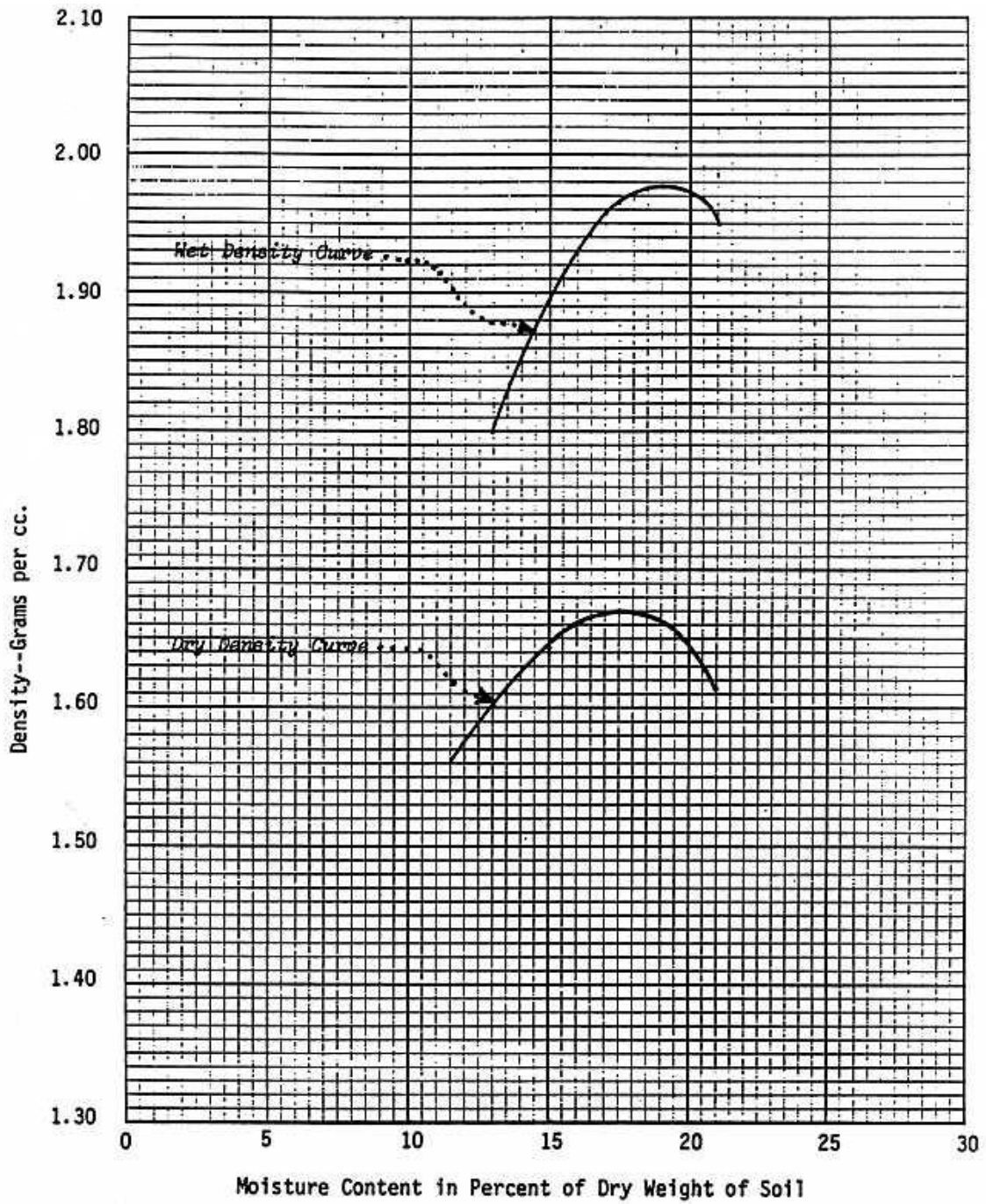


Figure E-2 – Relationship Between Soil Moisture and Density For Constant Compaction Effort.

The highest point of the dry density curve is called “maximum dry density: and the corresponding moisture content is called the “optimum moisture content” (OMC). For the soil shown in Figure E-2, the maximum dry density is about 1.67 gms/cc while the optimum moisture content is approximately 17.5%.

For each soil, there is a moisture content at which the maximum dry density can be achieved regardless of the quantity of compaction effort employed. At any moisture content lower than the optimum, insufficient water exists in the soil mass to adequately lubricate the surfaces of the soil particles. As more water is added, particle surfaces become better lubricated by water film and adjustment in position between soil particles is more easily accomplished. At the optimum moisture content (and maximum density) voids are nearly filled with water. Any increase in water content beyond OMC forces the soil particles apart resulting in lower than optimum density.

Any soil mass can be considered to be made up of three phases, solids, liquid (in the form of water) and air spaces or voids. When there is no water in the soil the voids are completely filled with air, while in saturated soil the voids are almost completely filled with water. A one cubic centimeter soil-water-air cube can be visualized as shown in Figure E-3.

If specific gravity of soil particles = 2.64, dry density = 1.6 gms/cc, and moisture content = 20%,

volume of solids = dry density/ specific gravity	= 1.6/2.64 = 0.606 cc
volume of water = 20% x 1.6 gms/cc	= <u>0.320</u> cc
total volume of solids and water	= 0.926 cc
volume of air = 1.000-0.926 cc	= 0.074 cc

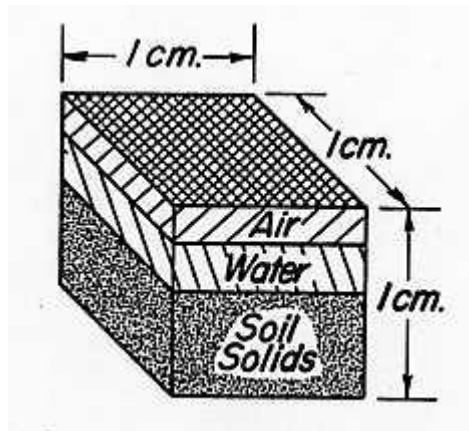


Figure E-3 – One Cubic Centimeter of Soil Divided into Solid, Liquid and Air Components.



If the dry density of compacted soil and specific gravity of soil particles are known, the moisture content where the voids will be completely filled with water can be calculated.

If the specific gravity of the soil particles = 2.64 and dry density = 1.6 gms/cc,

volume of soil solids = dry density/specific gravity =  $1.6/2.64 = 0.606$  cc

volume of water to completely fill the voids =  $1.000 - 0.606 = 0.394$  cc

0.394 cc of water weights 0.394 gms at 4° C

% moisture with all voids filled =  $0.394/1.6 \times 100\% = 24.63\%$

If this calculation is made for different values of dry density, a curve called the “zero air voids curve” (ZAV curve) can be plotted. The curve shown in Figure E-4 is for soil solids with a specific gravity of 2.64. Since the position of the zero air voids curve with respect to the moisture density curve is significant, the moisture density curve is shown in Figure E-4 as well.

Zero air voids curves can be plotted for soils with a variety of different specific gravities. ZAV curves commonly shift slightly from right to left corresponding to various values of specific gravities. This shift is slight for the range of specific gravities normally encountered when working with Nebraska soils. Since the zero air voids curve represents the condition where all void space is completely filled by water, no combination of dry density and moisture content can fall to the right of that curve. No known method of field compaction is capable of removing all of the air voids from a soil. Thus, the ZAV curve can serve as a check of test results for moisture density; if in plotting the results a sample falls on or to the right of the ZAV curve, an error has obviously been made.

Resistance to penetration can be considered as one method of measuring the ability of a soil to support loads. The dry density from Figure E-2 is plotted again on Figure E-5. A curve showing resistance to penetration, a ZAV curve and a curve for 3% air voids are also superimposed on Figure E-5. Values for plotting resistance to penetration were obtained by recording the pressure (in psi) required to force a needle of known end area into the compacted soil at a rate of 0.5 in/sec. Figure E-5 shows that resistance to penetration becomes weaker as moisture content increases, suggested that soil at higher moisture contents will carry less load.

Examination of Figure E-5 indicates that when this soil is compacted at a moisture content of 11.5%, resistance to penetration is greater than 2,000 psi. If water enters the soil at this density and it becomes nearly saturated (3% air voids), resistance to penetration falls to zero (by interpolation at bottom of resistance to penetration curve). If the soil shown in Figure E-5 is compacted to its maximum dry density (1.67 gms/cc) and saturated in the same manner (3% air voids), the soil now has approximately 300 psi resistance to penetration.

Research suggests that densities of typical Nebraska soils supporting flexible pavements remain near as-constructed density or show a slight increase in density due to the

kneading action of traffic. It is important that initial high densities are obtained in soils used for embankments and subgrades in order to limit the loss of strength that occurs if the moisture content increases.

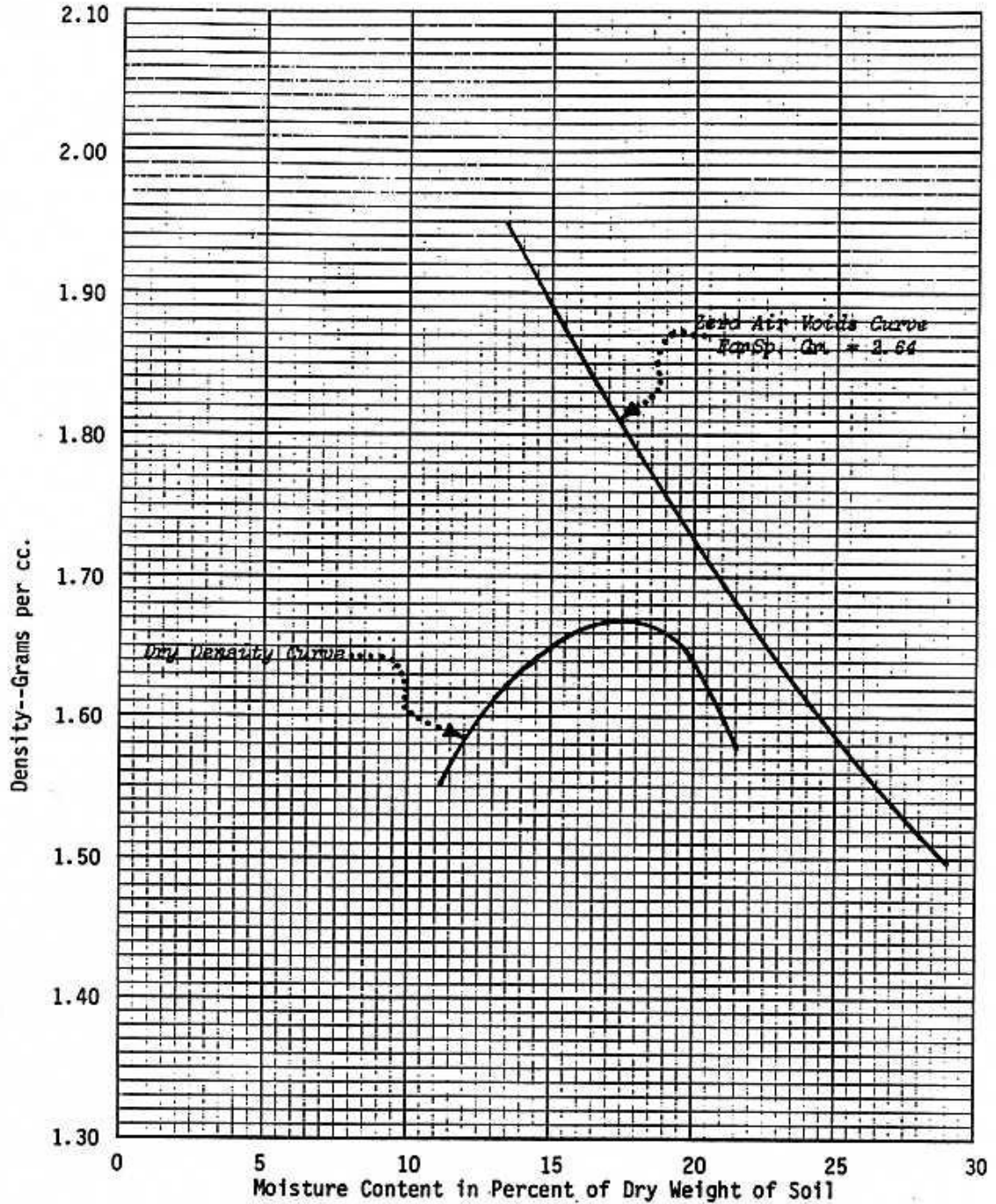


Figure E-4 - Relationship Between Standard Dry Density Curve and Zero Air Voids Curve.

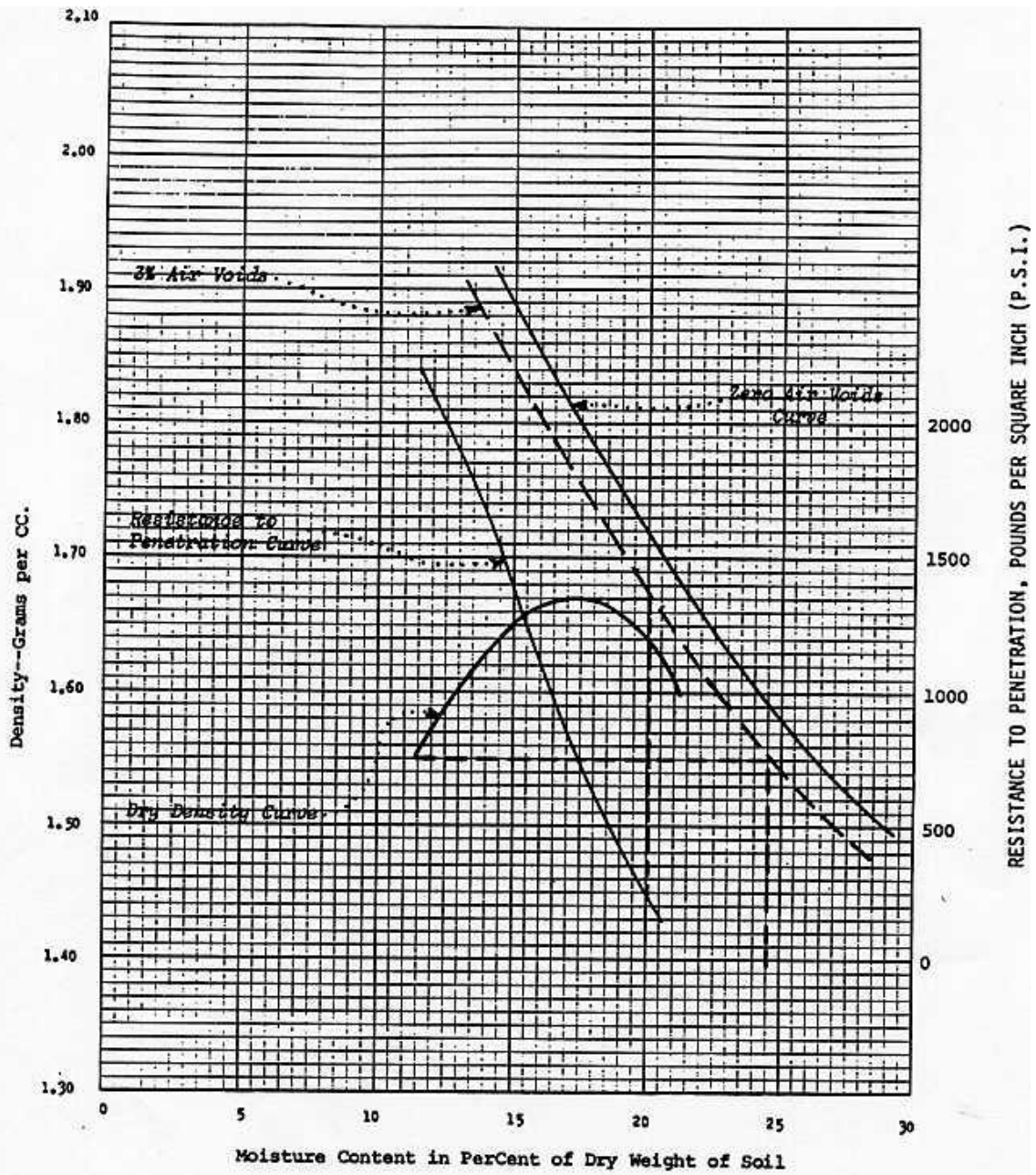


Figure E-5 – Resistance to Penetration at Different Moisture Contents and Dry Densities.

For any given method of compaction, the maximum density and optimum moisture content varies with the type of soil. In general, well-graded soils containing a mixture of gravel, sand, silt and clay have high maximum densities and low optimum moisture contents. Poorly graded and silt-clay soils have lower maximum densities and higher optimum moisture contents. Figure E-6 show moisture density curves representing typical Nebraska soils. The ZAV curve is also plotted to show its position with respect to this family of curves.

These typical curves can be used in field emergencies field when a different soil type is encountered for which no moisture density curve has been plotted. One compaction test is conducted using the unknown soil and standard ASTM procedure, having first adjusted the moisture content to near the anticipated optimum level. The moisture content and dry density of the compacted sample are than determined and plotted on the graph of typical curves. A new curve is drawn through the OMC, parallel with the curve(s) nearest the point. The new curve can be used as the moisture density curve for the unknown soil. This procedure should only be used on a temporary basis. If a significant quantity of unknown soil is encountered, standard moisture density testing should be conducted over a range of water contents so that a standard moisture density curve can be plotted.

When plotting a moisture density on a graph of typical curves, it is necessary to classify the soil as either granular or silt-clay. This can generally be determined with sufficient accuracy by visual inspection and by feeling the texture of the soil. Granular soils are materials that have 65-100% retained on the No. 200 sieve, while silt-clay soils have less than 65% retained on the No. 200 sieve.

An example of the example can be illustrated by an unknown soil with 72% retained on the No. 200 sieve. Dry density is 1.86 gm/cc at 8% moisture content. This point is plotted on granular series of curves and a curve is drawn parallel with the nearest granular curves. A close approximation of maximum density and optimum moisture content may be read from the peak of the new curve as approximately 1.88 gm/cc at 10.5% moisture content.

The same procedure can be used for a silt-clay soil with less than 65% retained on the No. 200 sieve. Dry density obtained from a compaction test is 1.60 gm/cc at 19.0% moisture content. Plotting parallel to the nearest silt-clay curve will yield a new curve with a maximum density of about 1.62 gm/cc at 20.8% OMC.

Selective placement notes are commonly associated with the construction of flexible pavement and are normally shown on the plan profile sheets. Selective handling and placement of soils may be required to create subgrade, base and/or surface courses of adequate capacity and thickness to support the layer(s) of flexible pavement. Experience has shown that cohesive soil placed over granular soil and then topped by pavement is detrimental to flexible pavement longevity. Selective placement is used to ensure that the upper part of the embankment is constructed of material similar to that in the lower part of the embankment or to place granular materials in the upper part of the embankment

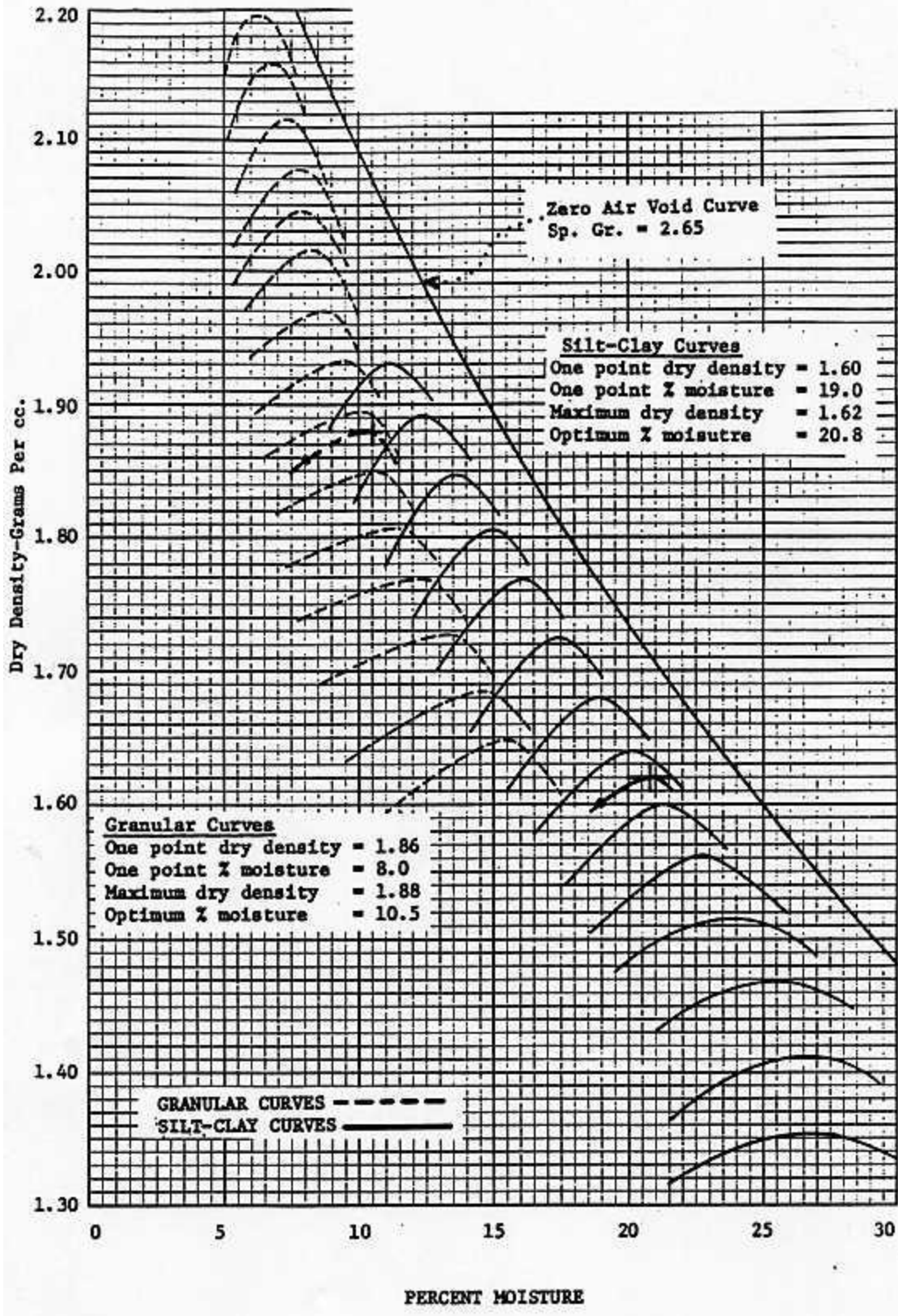


Figure E-6 – Moisture Density Curves for Typical Nebraska Soils.

immediately beneath the flexible pavement to enhance drainage. Selective placement may require meeting gradation requirements for fill materials. Materials considered for possible use must be sampled to ensure that gradation requirements are in accordance with fill specifications.

The compaction block in the grading plans will indicate whether the embankment(s) on the project are classified at Class I, Class II or Class III embankments. If a Class III embankment is to be constructed, the density and moisture requirements are show on the plans. As an example of a Class III embankments, assume the following specifications are shown in the compaction block on the plans:

\*\*\*\*\*

**COMPACTION REQUIREMENTS**  
CLASS III (See Specifications)

	SOIL TYPE	DEPTH BELOW FINISH SUBGRADE	PERCENT DENSITY	MOISTURE REQUIREMENTS MINIMUM	REQUIREMENTS MAXIMUM
Embankment/Roadway Grading, including driveways, to receive concrete pavement	Silt-Clay	Upper 3 feet	98 Min.	Opt. -3%	Opt. +2%
	Silt-Clay	At depths greater than 3 feet	95 Min.	Opt. -3%	Opt. +2%
	Granular	All depths	100 Min.	**	**
Embankment/Roadway Grading, including driveways, to receive flexible pavement	Silt-Clay	Upper 3 feet	100 Min.	Opt. -2%	Opt. +1%
	Silt-Clay	At depths greater than 3 feet	95 Min.	Opt. -3%	Opt. +2%
	Granular	All depths	100 Min.	**	**
Embankment/Roadway Grading not to be surfaced	All	All depths	95 Min.	Opt. -3%	Opt. +2%
-----					
Subgrade Preparation, Shoulder Subgrade Preparation (Concrete Pavement)	Silt-Clay	The upper 6 inches of subgrade soil	98 Min.	Opt. -3%	Opt. +2%
	Granular	The upper 6 inches of subgrade soil	100 Min.	**	**
Subgrade Preparation, Shoulder Subgrade Preparation (Flexible Pavement)	Silt-Clay	The upper 6 inches of subgrade soil	100 Min.	Opt. -2%	Opt. +1%
	Granular	The upper 6 inches of subgrade soil	100 Min.	**	**
-----					

A contractor is compacting a silt-clay subsoil to create the upper three feet of an embankment. Maximum density for this soil is 1.54 gm/cc with optimum moisture content of 23.5%. From the compaction requirements shown, the required density range is 93% to 99% of maximum density, which is 1.43 to 1.52 gm/cc. The allowable moisture range is  $23.5 \pm 3\%$ , which is 20.5 to 26.5% for this soil.

Before a contractor opens a borrow pit or a cut section, the moisture content of each soil layer should be analyzed to determine if the soil in that layer will have to be wetted or dried to bring its moisture content within the specified limits. If the natural moisture content is within specifications, the soil can sometimes be used with no modification. Sometimes a small amount of water must be added to replace moisture lost by evaporation during construction operations. Preliminary moisture determinations are commonly made several days to a week ahead of actual excavation so that proper equipment can be available for wetting or drying of the soil prior to final grading operations.

If the natural soil moisture content is greater than that allowed by specifications, the moisture content must be reduced. Drying may be accomplished by disking the soil and allowing water vapor to evaporate naturally. When insufficient time for natural drying is available, the use of chemical additives may be appropriate. Frequency of sampling should satisfy always meet or exceed project specifications.

Field moisture density sampling is now frequently done using a nuclear moisture density gauge. ASTM D 2292 details standards for determination of soil density using nuclear equipment while D 3017 covers determination of moisture content using nuclear gauges. Nuclear moisture density equipment is accurate and provides both moisture and density data with a minimum of time and effort.

## Appendix F

### General Guidelines for Using Geosynthetics in Foundation Reinforcement

In order to improve the ability of a soil to serve as a foundation for any structure, NDOR personnel typically examine four options. These include 1) bypass the unsuitable soil through relocation of the structure to another site, 2) redesign the structure to meet soil limitations, 3) alter the properties of the natural soil to meet foundation requirements, or 4) replace the poor soil with a better material or combination of materials that will offer adequate support. This appendix provides suggestions and guidelines for the fourth option.

Culverts are often situated on soils with poor bearing capacity and poor stability because of their location in stream bottoms. Replacement of poor soil beneath culverts has traditionally been used to solve foundation problems at locations where the bearing capacity of the soil is less than  $\sim 0.5$  tons/ft<sup>2</sup> (a man walking across the soil surface sinks about 1 inch). The most commonly used approach is to excavate the in-situ soil to a prescribed depth and then replace the excavated soil with a coarse, granular material having a high angle of internal friction. This process creates a stable platform of granular material that controls differential settlement and limits lateral deformation. In recent years, various geosynthetic materials have been incorporated into this platform to further increase a soil's stability and to decrease the thickness of granular layer required.

A large volume of literature is focused on the uses of geosynthetic materials to increase the bearing capacity and stability of various soils. For a geogrid reinforced foundation, model tests have been conducted to study the effects of various spacing and length factors on reinforced soil bearing capacity, including the distance between the uppermost reinforcement layer and the bottom of the footing ( $u$ ), the spacing between reinforcement layers ( $s$ ), the distance from the lowest geogrid to the bottom of the reinforced fill ( $a$ ), the width of the reinforced layers ( $b$ ), the number of reinforcement layers ( $N$ ), and the thickness of the reinforced soil zone ( $z$ ). These dimensions and lengths are illustrated in Figure F-1.

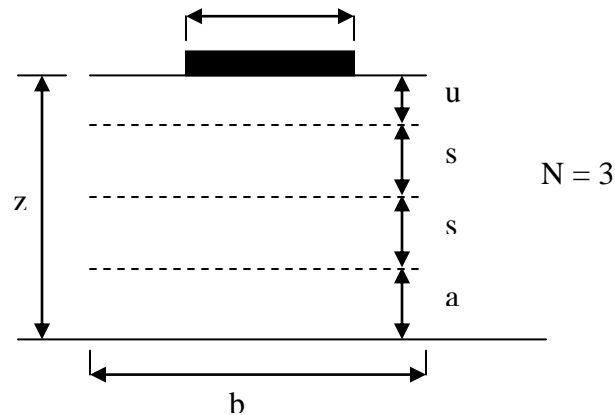


Figure F-1 – Geogrid Spacing and Length Factors.



Studies of geogrid-reinforced foundations have involved loads applied through strip and/or spread footings. Strip footing loading patterns approximate foundation loading conditions commonly found beneath box culverts. Based on experimental data, literature review and economic considerations, recommended design parameters for geogrid reinforced soil foundations beneath pipe culverts are shown in Table 1F.

The design values shown in Table 1F are based upon geogrid with a 1% junction tensile modulus of 2.48 kN/m in the machine direction and 2.92 kN/m in the cross machine direction. These values correspond to BX 6100 geogrid manufactured by Tensar Earth Technologies, Inc. However, geogrid produced by other manufacturers with similar tensile moduli should perform in a very similar manner.

A geosynthetic-reinforced foundation distributes applied loads across a wider footprint than an unreinforced foundation of similar dimensions. The minimum thickness of a geosynthetic-reinforced foundation can thus be less than the minimum thickness required for an unreinforced foundation and still significantly improve stability and bearing capacity of a soil.

Pipe culverts are normally bedded in a layer of granular material. The material used as a culvert-bedding layer serves as the layer of material between the uppermost reinforcement layer and the bottom of the footing (u). Care must be taken to place the invert of the culvert at a high enough elevation so that the minimum thickness of this layer is not compromised.

Figure F-2 shows the recommended minimum thickness of replacement material beneath pipe culverts when the replacement material has been reinforced with two layers of geogrid. The minimum number of geogrid layers recommended for any granular reinforced foundation is two. B, footing width of a strip footing, is assumed to be equivalent to D, outside diameter of the culvert. All layers of geogrid extend outward laterally for a distance of 0.5 D from the extreme lateral edges of the pipe culvert, as shown in Figure F-3.

Table 1F – Recommended Values for Geogrid Used in Reinforced Foundations.

Parameter	Recommended Value
a	0.1-0.2B
b	2.0-3.0B
N	2 to 4
s	0.15 to 0.3B
u	0.15-0.3B
z	0.5-1.0B

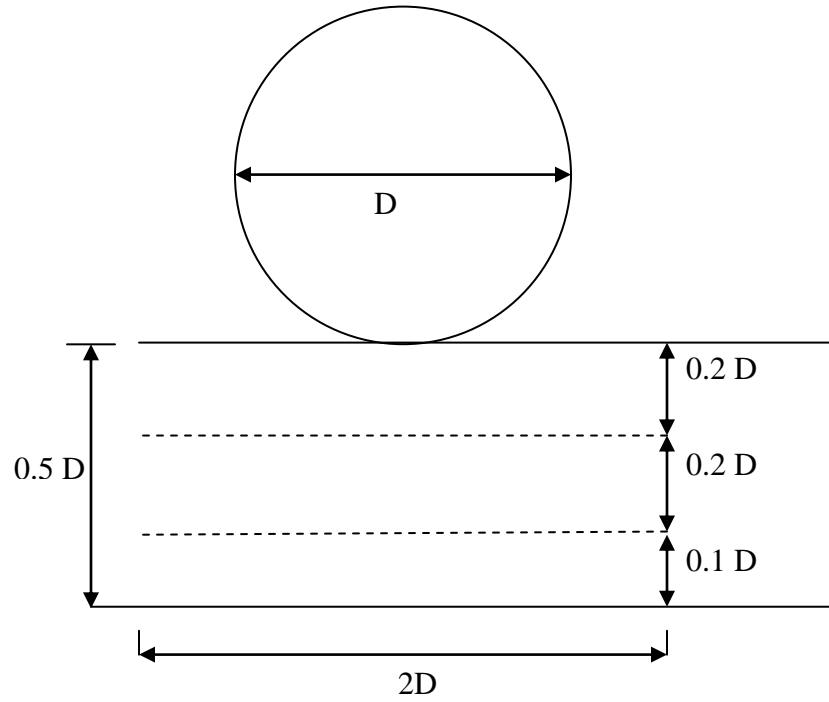


Figure F-2 – Minimum Thickness of Replacement Material with Geogrid Reinforcement.

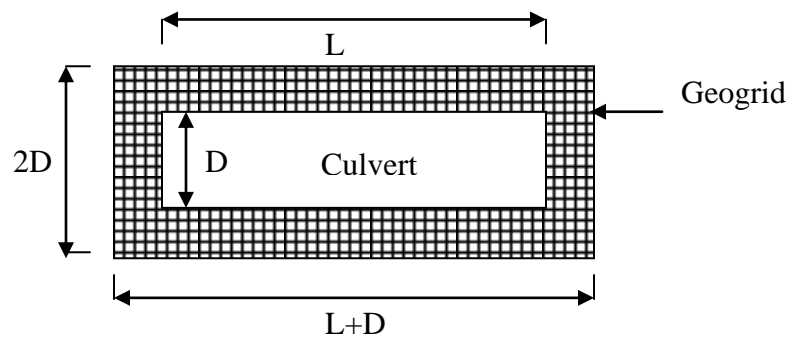


Figure F-3 – Plan View Showing Lateral Extent of Culvert and Geogrid .

Box culvert foundations that will be constructed in locations with poor soil can be stabilized in a similar manner. Once the box culvert location has been cut to approximately finished grade (bottom of the box), the area should be inspected for soft or very soft soils. Very soft soils are defined as materials with a compressive strength between 0 – 0.125 tons/ft<sup>2</sup>. A person will experience difficulty while walking across a very soft soil, as with each step they will sink to a depth of three inches or greater. Soft soils are defined as materials with a compressive strength between 0.125 – 0.25 tons/ft<sup>2</sup>. A person walking across soft soil will also experience difficulty walking but will sink only 1-3 inches. Neither soft nor very soft soils have the capacity to support construction equipment.

Soft and very soft soils are commonly removed from beneath a box culvert foundation to a depth where the soil will support foot traffic with one inch of deformation or less or to a depth of five feet (1.5 m), whichever is less. When excavating soft material from beneath a proposed box culvert location, the NDOR geotechnical engineer should be consulted if any of the following conditions are encountered a) the side slopes of the excavation appear unstable, b) excavation must proceed more than five feet (1.5 m) below the bottom of finished grade, c) excavation must proceed more than 3 feet (1 m) below the groundwater table, or d) other unusual conditions are encountered at the site. The resulting excavation is commonly backfilled immediately after completion of excavation operations. Near simultaneous excavation and placement operations may be required when groundwater is rapidly filling the excavation. An excavation is normally filled with granular material to finished grade and the box culvert is subsequently constructed directly on top at grade level.

Geogrid reinforced box culvert foundations will normally be designed for each specific location, as both the soil properties and culvert loads vary considerably from site to site. General guidelines suggest that if a geogrid reinforced foundation is to be constructed for a box culvert, soft or very soft soil should be removed to a minimum depth of approximately two feet (0.6 m). The area of excavation should extend one-quarter of the culvert width to either side of the culvert and for a distance of one-tenth of the culvert width at each end. Geogrid should be placed across the top of the natural soil and pulled tight before being covered with approximately two feet (0.6 m) of aggregate meeting the requirements of coarse aggregate used in NDOR 47B concrete.

The ground surface may consist of soft to very soft soil for a considerable distance in all directions around some culvert sites, requiring construction of a working platform for equipment. Design of the working platform is unique to each situation, as soil bearing capacity and shear strength will vary with soil type, moisture content and drainage. Loads imposed on the working platform by construction equipment will also vary. Lightest loads are normally associated with equipment that pushes soil, heavier loads with equipment that transports soil, and highest loads with cranes and other lifting equipment. Loads from wheeled equipment are more concentrated and generally heavier than loads associated with tracked equipment of the same capacity.

General guidelines suggest that if a geogrid reinforced foundation is to be constructed for an equipment platform, very soft soil should be removed to a minimum depth of approximately two feet (0.6 m). Geogrid should be placed across the top of the natural soil and pulled tight before being covered with approximately two feet (0.6 m) of aggregate meeting the requirements of coarse aggregate used in NDOR 47B concrete. With soft soil, the same guidelines should be followed except depth of excavation can be limited to one foot (0.3m). Geogrid should be placed across the top of the natural soil and pulled tight before being covered with approximately one foot (0.3 m) of aggregate meeting the requirements of coarse aggregate.

Geogrid reinforced foundations can improve the bearing capacity and stability of most soils under all loading patterns while limiting total and differential settlement and significantly reducing the quantity of fill material that must be purchased and transported to the site. In situations where fill is moderately expensive or where fill must be transported long distances to the project site, geogrid reinforcement may offer an economically attractive alternative.