GEOTECHNICAL POLICY

AND PROCEDURES

MANUAL

June 2003

DISCLAIMER STATEMENT

The purpose of this handbook is to provide contractors and NDOR personnel working in the field with guidance concerning appropriate procedures for performing geotechnical activities in support of Nebraska Department of Roads projects. This manual contains only some of the tasks and methods involved in geotechnical investigations. It discusses many but not all geotechnical aspects of design that must be specified for construction of roads and road structures. This manual should be used only for guidance. It is not intended to be a comprehensive or all-encompassing methods handbook.

Each project has unique considerations and requires application of engineering judgment based upon a thorough knowledge of the specific project site and its particular characteristics. This manual is not intended to serve as a procedural handbook that defines a scope of geotechnical services required for each project. The design engineer is responsible for defining the scope of geotechnical services for specific projects. This manual is not intended to bypass nor to supplant the engineering judgment or experience of the design engineer.

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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 steps that should be considered for any project. A review of available data 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.

1.2 Sources of Geotechnical Data.

1.2.1 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.

1.2.2 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 before construction of modern features. Historical aerial photographs may also reveal remnants of previously existing man-made structures, some of which could adversely affect proposed structures.

1.2.3 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. The Soil Survey Section of Materials and Research Division maintains geologic maps that pertain to the State of Nebraska.

1.2.4 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.

1.2.5 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. Bridge Foundations Section stores written boring logs for most bridge foundation locations on Nebraska roads, dating from approximately 1927 to the present. Bridge Foundations Section also stores pile-driving records associated with pier and bent construction dating from approximately 1932 to the present. Soil Survey Section stores soil boring data associated with grading operations for many 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 Soil Laboratory has recently begun storing boring logs in electronic format using the *GeoSystems* software. This information is available on the NDOR mainframe in a subdirectory called \\dorimage3\SoilBoring. The Soil Laboratory 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 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.

Chapter 2

Geotechnical Investigations

2.1 Subsurface Investigations.

2.1.1 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:

- 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.

- Observe and comply with federal, state and local laws, ordinances and regulations that affect the work being conducted.

- Obtain all applicable permits and licenses from the appropriate agencies. Notify landowners of any work done on private land.

- Determine if environmental or archeological clearances are required if there is sufficient evidence to suspect this may be a concern.

- 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.

- 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.

- 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.

- Ensure proper closure of all bore holes, according to applicable laws and regulations of the State of Nebraska and local agencies.

2.1.2 Soil and Subgrade 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. 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 dividing the project into sections based upon type of soil in the upper subgrade or identifying and locating any problems with moisture in the existing subgrade. 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.

2.1.2.1 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.

2.1.2.2 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.

2.1.2.3 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. Total number of samples submitted for testing is thus held to a more reasonable number. Large samples (80 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.

2.1.3 Borings for Embankment Areas (Soil Mechanics Borings).

2.1.3.1 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. Increased intervals can be used when drilling boreholes for smaller embankments less than 20 ft (6 m) in height.

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.

2.1.3.2 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 5 ft (1.5 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.

2.1.3.3 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 (1.5 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.

2.1.4 Borings for Structures.

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.

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.

The depth of boring required can 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.

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 to 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 AST 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.

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. It is not uncommon to have several boreholes from different days on the same project identified as DH-1. One solution to avoid duplication is to designate that all boreholes for bridge piers or abutments begin with the letter “B”, followed by the initials of the river being crossed and finally a sequential number from a series of numbers assigned to that specific project. For example, the first borehole on a bridge project across the Platte River might be designated DH-BPR-100. Drill holes for embankments could begin with the letter “E” while drill holes for cut sections could begin with the letter “C”.

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.

2.1.5 Borings for Mechanically Stabilized Earth (MSE) Walls.

Typically, two borings per MSE 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 retaining embankments 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 MSE wall.

While two-thirds of MSE 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 MSE wall height, the borehole should extend a minimum of 5 ft (1.5 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.

2.1.6 Borings for Culverts.

2.1.6.1 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.

2.1.6.2 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.

2.1.7 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) without consulting a geotechnical engineer.

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 unexpectedly encountered before the above criteria are met, a geotechnical engineer should specify the depth of borehole required.

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 techniques (i.e. electronic CPT testing) may allow for a significant reduction in the number of samples required. Sampling frequency (with no additional subsurface testing) should be no greater than 2.5 ft (0.75 m) 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.

2.1.8 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.

2.1.9 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 fairly constant based upon experience or extended practice for a particular area. Almost no light poles experience internal failure from insufficient depth of embedment.

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

2.1.10 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.

2.1.11 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.

2.1.12 Backfilling Boreholes.

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 R 22-97, *Standard Guide for Decommissioning Geotechnical Exploratory Boreholes* and AASHTO R 21-96 (2000) *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 are also prevent migration of water between adjacent vertical aquifers via the borehole.

2.2 Laboratory Tests.

2.2.1 Soil Classification Systems.

2.2.1.1 AASHTO 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. One widely recognized system of soil classification associated with highways was devised by the Public Roads Administration for classification of subgrade soils. This system, known as the AASHTO M145 standard, classifies soils into one of seven groups, designated A-1 through A-7, according to their general load carrying capacity. The AASHTO M145 classification standard is illustrated in Figure 1.

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 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.

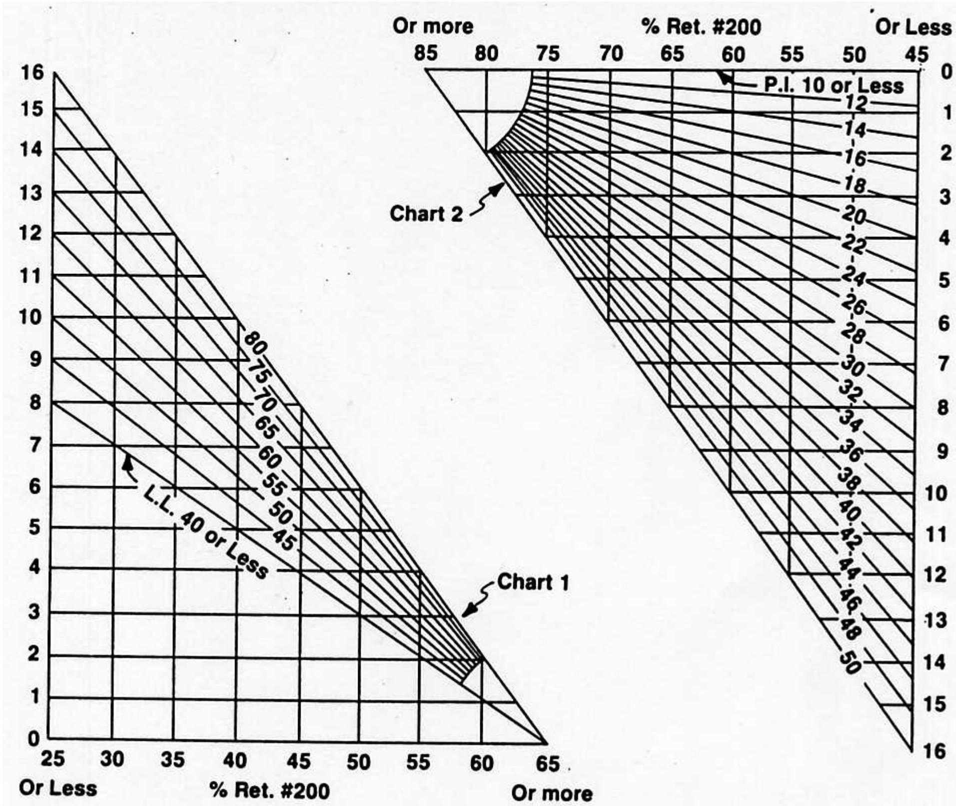
Computed group values of less than zero are reported as zero. Under conditions of good drainage and thorough compaction, the bearing capacity of the subgrade material may be assumed to be inversely proportional to its group index. Thus a group index of zero represents a subgrade material with a relatively high bearing capacity while a group index of 20 or more represents subgrade material with a low bearing capacity.

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (More than 35% passing No. 200)			
Group classification	A-1			A-2							A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	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 1 - 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 2. Group index values using the Nebraska Revised Group Index commonly range from –4 to 32.



Group Index { Positive Values - Sum of Readings on Vertical Scale of Charts 1 & 2.
Negative Values - Use Chart 3.

	Well Graded Gravel Base	Clean Coarse Sand	Clean Fine Sand	Loamy Coarse Sand	Loamy Fine Sand	Loamy Very Fine Sand
Group Index	- 4	- 3	- 2		- 1	0
% Ret. #10	40 Min.					
% Ret. #40	60 Min.	35 Min.	34 Max.	35 Min.	34 Max. 10 Min.	9 Max.
% Ret. #200	85 Min.	85 Min.	85 Min.	84 Max. 65 Min.	84 Max. 65 Min.	84 Max. 65 Min.
P.I.	4 Max.	4 Max.	4 Max.	10 Max.	10 Max.	10 Max.

The first group from the left into which the test data will fit is the correct classification.

Figure 2 – Nebraska Revised Group Index Charts.

2.2.1.2 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 D 2487.

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 3.

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^{o} 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 4, 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 4 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	Classification on basis of percentage of fines: Less than 5% Pass No. 200 sieve GW, GP, SW, SP; More than 12% Pass No. 200 sieve GM, GC, SM, SC; 5% to 12% Pass No. 200 sieve Borderline classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4; $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW
		Gravels with fines	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
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean sands	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
			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
Fine-grained soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		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		
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity		
Highly organic soils			Pt	Peat, muck and other highly organic soils		Fibrous organic matter; will char, burn, or glow

(a)

Figure 3 – Unified Soil Classification System (USCS).

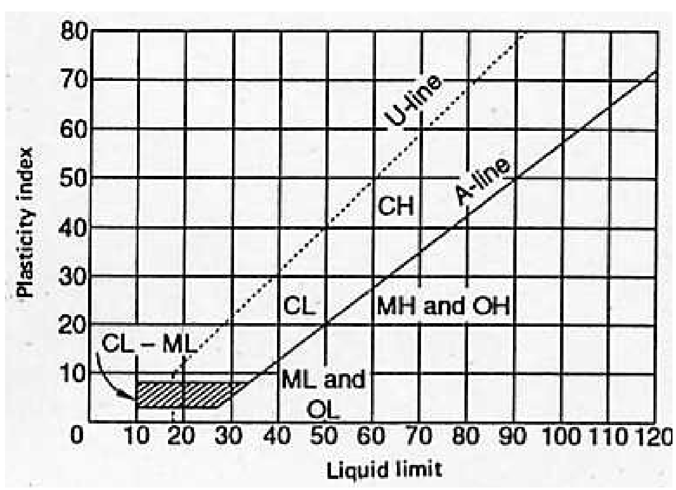


Figure 4 – Plasticity Chart for Classification of Fine-Grained Soils.

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.

2.2.2 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 (or ASTM D 422) while the hydrometer analysis is conducted in accordance with AASHTO T 88 (ASTM D 2217).

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.

2.2.3 Specific Gravity.

Specific gravity of a soil is determined in accordance with AASHTO T 100 or ASTM D 854. 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.

2.2.4 Moisture Content (Atterberg Limits).

Soil moisture content is measured in accordance with AASHTO T 265 or ASTM D 2216/ASTM D 4643. Moisture content is the 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^o C using a conventional oven for 15-16 hours (ASTM D 2216), or by using a microwave oven requiring only a few minutes (ASTM D 4643). 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, which are 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 limits are commonly measured as a basis of 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.

2.2.4.1 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.

2.2.4.2 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.

2.2.4.3 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 preformed on soils that may undergo large volume changes as water content increases or decreases.

2.2.5 Unit weight.

The unit weight of a soil is represented by the symbol γ. Unit weight is commonly expressed in pounds per cubic foot or kilonewtons per cubic meter. Unit weights can be reported as wet unit weight, γ_{wet} or dry unit weight, γ_{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.

2.2.6 Moisture Density Relationship.

Most construction projects have specifications that indicate a minimum 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 are 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 and dividing by the volume determines the wet density of the soil. A small sample is cut from the center of the soil in the mold and used to obtain water content. The dry density 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 5 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.

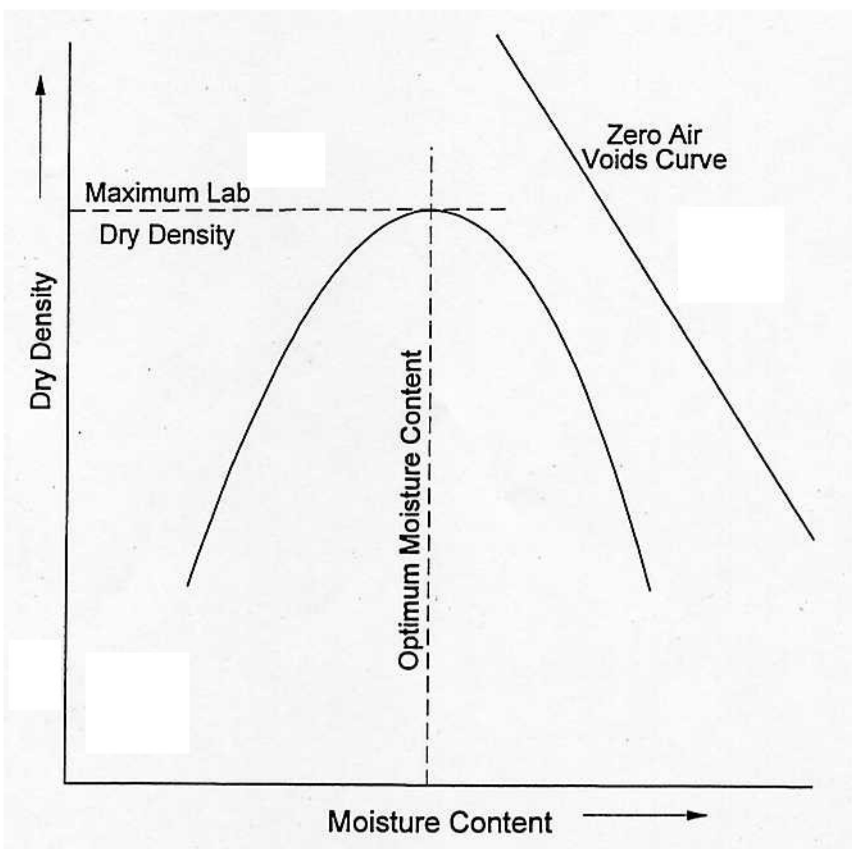


Figure 5 – Moisture Density Curve.

Knowledge of soil OMC is important to both the contractor and the inspector. Informed decisions must be made relative to the treatment of the soil prior to and during compaction. If the soil has actual moisture content vastly different from OMC, continued compaction will prove uneconomical to achieve the desire 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.

2.2.7 Consolidation/Swell/Collapse Tests.

2.2.7.1 One-Dimensional Consolidation Test.

The one dimensional consolidation test (AASHTO T 216 / ASTM D 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 transferring 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 can be obtained by use of a 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 deceasing 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 magnitude 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.

2.2.7.2 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 their overburden is wholly or partially removed. These types of soils may absorb water from the atmosphere or ground water and subsequently 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.

2.2.7.3 Collapse Potential Test.

The collapse potential for a specific soil can be determined from any test method that generates an e-log P curve (AASHTO T 216, ASTM D 2435 or ASTM D 4546). 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. Soils with these characteristics are 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 .

2.2.8 Shear Strength Tests.

2.2.8.1 Unconfined Compression Test.

The unconfined compression 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 the unconfined compression test can be found in AASHTO T 208 or ASTM D 2166.

2.2.8.2 UU Triaxial Test.

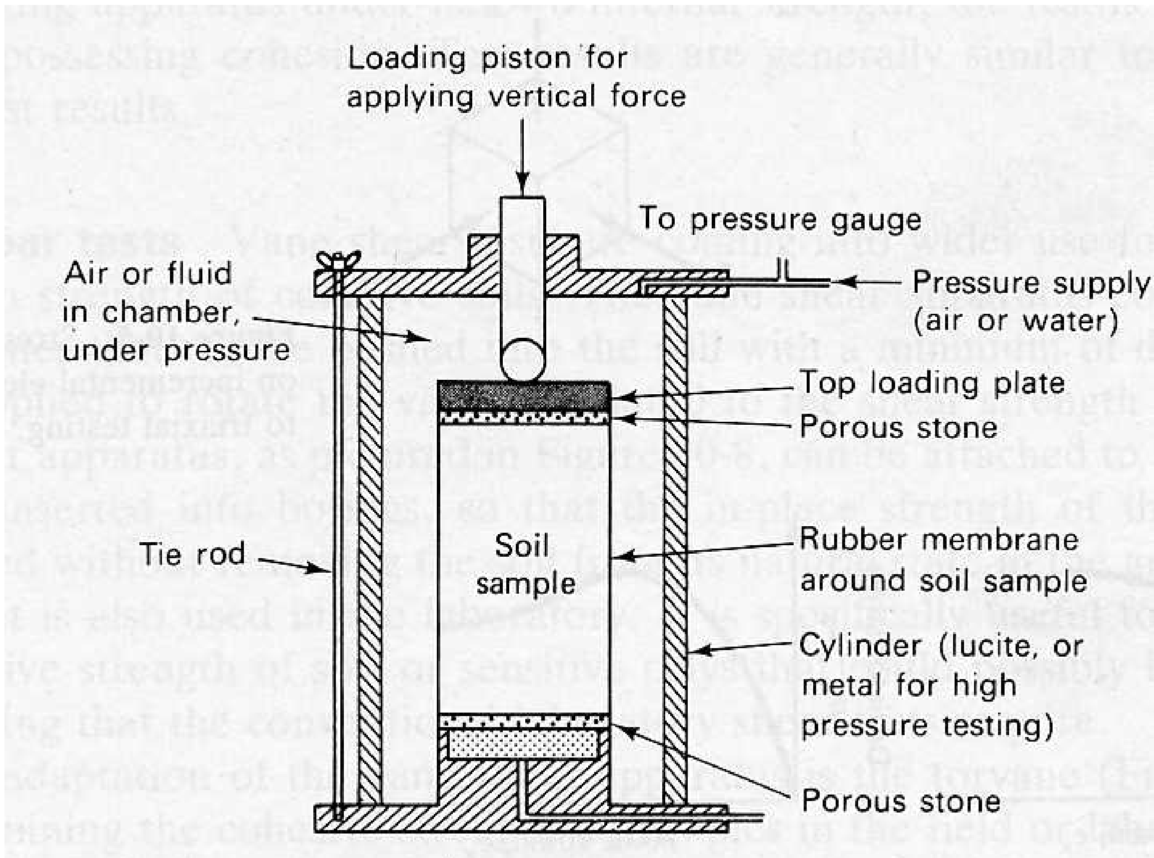
For common triaxial tests, a cylinder of soil, (typically obtained from boring) is wrapped in a membrane to protect it and placed in a closed chamber where a confining pressure can be applied, normally by a fluid, around the outside circumference of the soil sample. The sample sits on a fixed pedestal 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 6 shows a schematic diagram of a triaxial test apparatus.

The test procedure designated "UU" 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. Test procedures are contained in AASHTO T 296 or ASTM D 2850.

2.2.8.3 CU Triaxial Test.

For a CU (<u>C</u>onsolidated, <u>U</u>ndrained) triaxial test, 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 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. Test procedures are contained in AASHTO T 297 and ASTM D 4767.

A variation the CU test is the CD (<u>C</u>onsolidated, <u>D</u>rained) triaxial test. This test is conducted in exactly the same manner as the CU test, except that the drainage valve is opened as vertical axial stress begins to increase, allowing pore water pressure to dissipate. The CD test evaluates the strength of fine-grained soils under long-term, drained loading conditions.



(Source: McCarthy, 1993)

Figure 6 – Schematic Diagram of Triaxial Test Apparatus.

2.2.8.4 Direct Shear Test.

The direct shear apparatus used for performing the direct shear test 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 7. Specifications for the direct shear test can be found in ASTM D 3080.

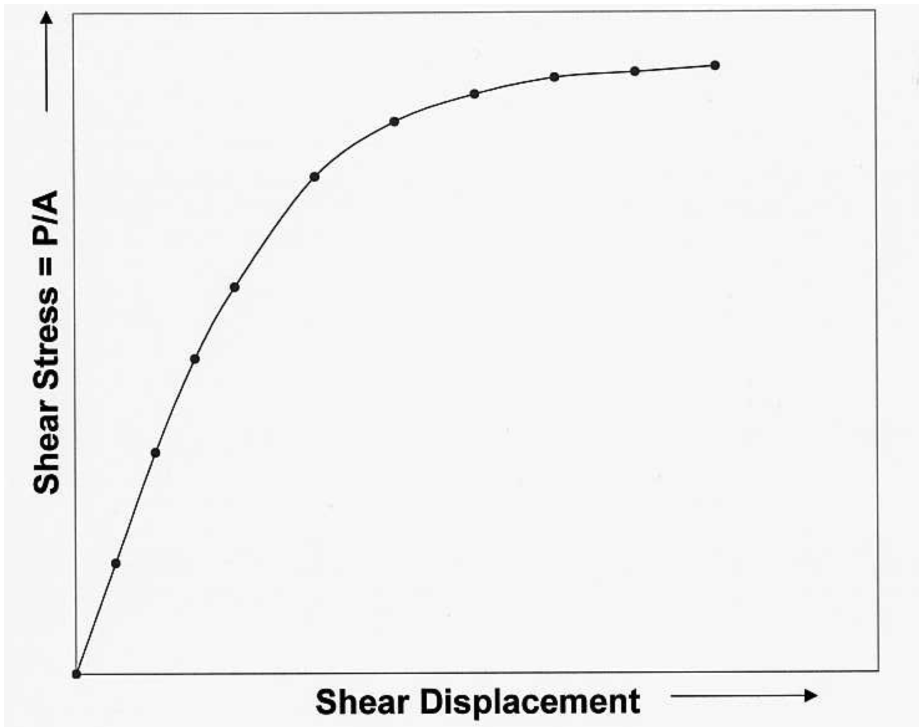


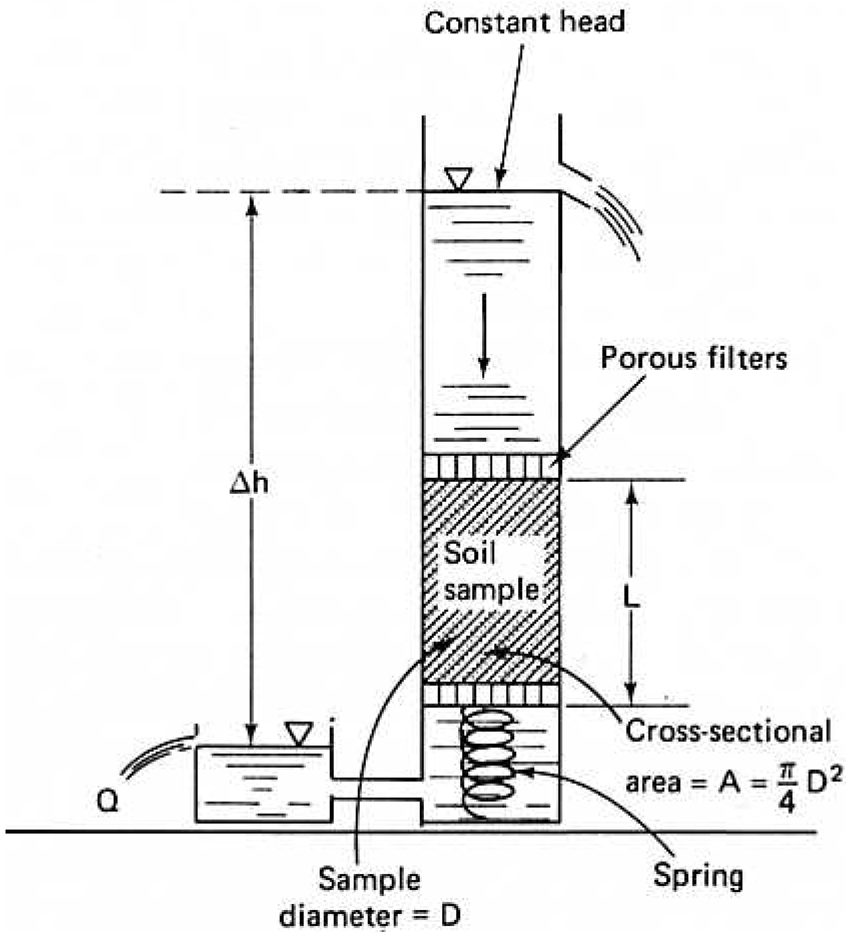
Figure 7 – Direct Shear Test Data.

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.

2.2.9 Hydraulic Conductivity Tests.

2.2.8.1 Constant Head Test.

The constant head test 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 specified 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 8 to determine hydraulic conductivity for the sample. This test is commonly 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 8 – Constant Head Hydraulic Conductivity Test.
(Source: McCarthy, 1993)

2.2.8.2 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 9.

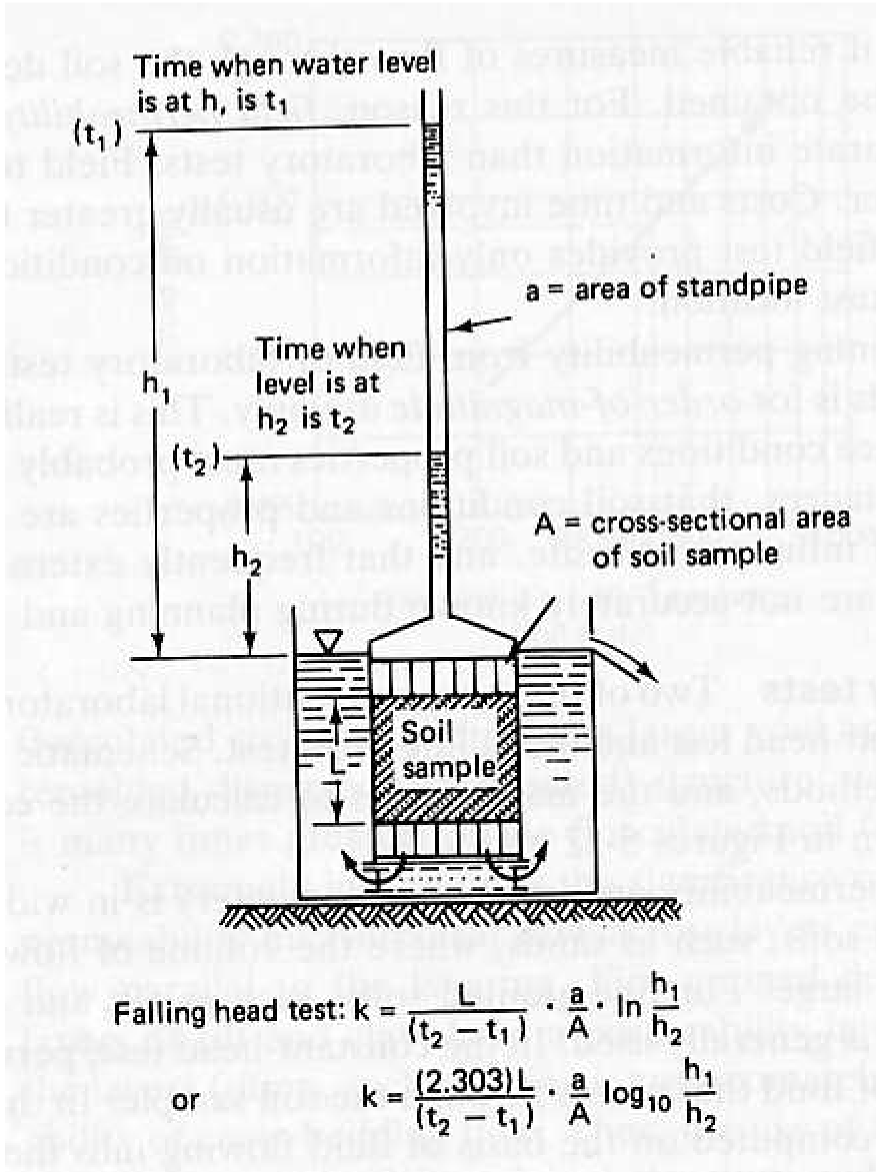


Figure 9 – Falling Head Hydraulic Conductivity Test.
(Source: McCarthy, 1993)

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 9 to obtain the hydraulic conductivity.

2.2.8.3 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 plastic, so the sides of the permeameter enclosing the sample are 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 were created to address this deficiency. 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 permeater can be used as the sample container for either a constant or falling head test.

2.3 Field Tests.

2.3.1 Shelby Tube Sampler.

Shelby tube borings are taken to obtain relatively undisturbed soil samples, usually in order to conduct more detailed soil tests. Weak cohesive soils are frequently the subject of such tests. Shelby tube borings should be taken when embankment slope stability or settlement are judged to be marginal or when the slope stability analysis results in a factor of safety of less than 1.5 for natural embankments or less than 1.75 for cut slopes. Tests performed on Shelby tube samples may include settlement analysis, consolidation tests, unconfined compression tests, moisture content, Atterberg limits, and particle size analysis. Shelby tube borings are required to accurately quantify settlement in any situation where the fill height is greater than 15 feet (4.5 meters) or where the moisture content of the fill material is greater than 25%.

2.3.2 Standard Penetration Test (SPT).

The SPT is conducted in accordance with AASHTO T 206 or ASTM D 1586. Standard penetration testing uses a sampling device known as a split-spoon or split-barrel sampler. 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. A common variation of the split spoon sampler is a split-barrel sampler, which consists of a solid shaft with a split insert liner.

The standard penetration test consists of driving the sampler into the soil while recording the blow count required to drive the sampler a specific distance. The number of blows, N, required to drive the sampler 12 inches (300 mm) with a hammer weighing 140 lbs (63.6 kg) while falling from a height of 2.5 feet (0.762 m) is known as the standard penetration resistance.

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. The sum of the latter two increments is the N value.

The SPT is performed to obtain a representative sample of subsurface soil 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 cohensionless 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.

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.

N Value	**Relative Density**	**Friction Angle**
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	$\leq$50

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

N Value	**Consistency**	**Strength, Q_u, kPa (tsf)**
< 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)

2.3.3 Cone Penetration Test (CPT).

The cone penetrometer is composed of a thin metal rod equipped with a cone-shaped tip. The penetrometer is advanced vertically though the soil at a specified rate and the resistance to penetration is measured. The penetrometer can be pushed into the earth by a hydraulic jack (static cone penetrometer) or driven into the earth by blows from a drop hammer (dynamic cone penetrometer). Electric penetrometers, where an electrical cell within the penetrometer advances the tip, are also available. Electric penetrometers that are capable of measuring pore water pressure during penetration are known as piezocone penetrometers.

For all types of penetrometers, a cone angle of 60^o and a tip area of 1.55 in^2 (10 cm^2) are standard. Penetration rates are normally between 0.4 and 0.8 in/sec (10 to 20 mm/sec). Tests are performed in accordance with ASTM D 3441 (for mechanical penetrometers) or ASTM D 5778 (for piezocone penetrometers).

A series of tests performed on soil at various depths in a single location is normally referred to as a sounding. Penetrometer data is plotted as a standard log that shows end bearing resistance, friction resistance along the penetrometer sides and the friction ratio (ratio of side friction resistance divided by the end bearing resistance). Pore water pressures are generally plotted against depth or time for pieziocone penetrometers.

The friction ratio from penetrometer plots can be analyzed to determine soil classification, shear strength and liquefaction potential. Correlations have been made that allow design of spread footings and pile foundations based upon CPT data. Generally, soil samples will not be obtained in conjunction with CPT soundings, so cone penetrometer testing is normally augmented by SPT borings or other borings where soil samples are collected.

2.3.4 Vane Shear 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 is conducted in accordance with AASHTO T 223 or ASTM D 4648.

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 does not work well in soils that contain pebbles or stones. Soils that drain or dilate during testing will yield inconsistent results.

2.3.5 Pressuremeter Test.

The pressuremeter test (PMT) employs a device designed to determine 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, while 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 10.

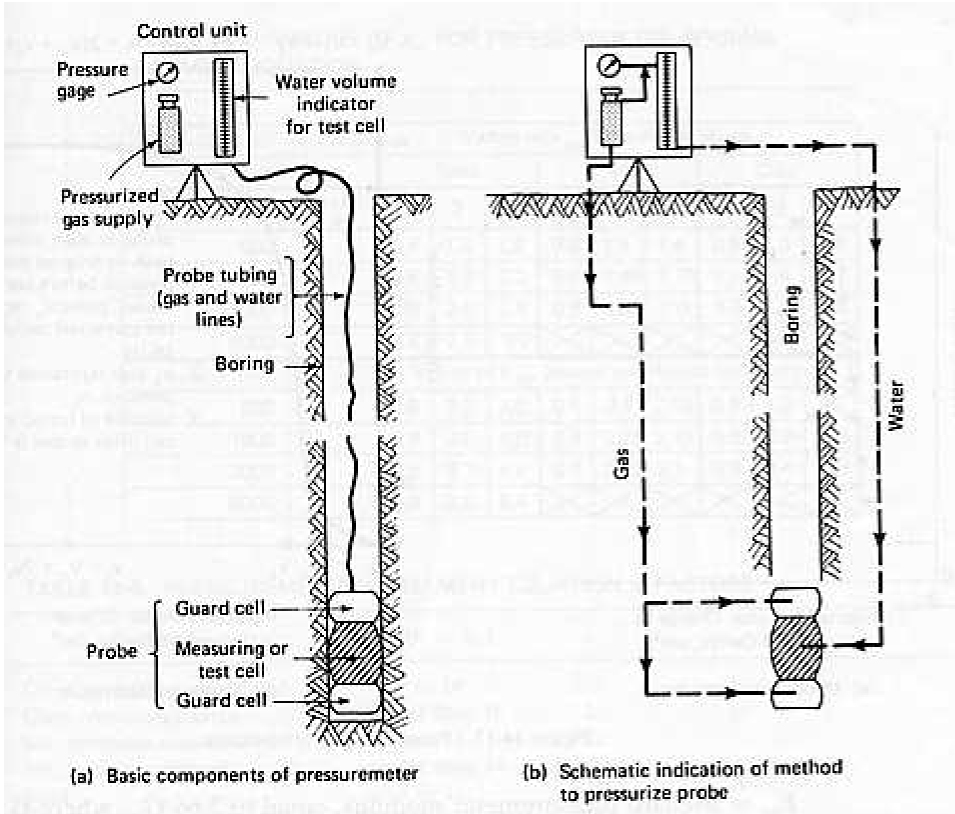


Figure 10 – Setup for a Pressuremeter Test.

2.3.6 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.

The Torevane shear test is conducted using a small, cylindrical device that has an axially radiating set of small vanes on one end and a dial on the other. A fresh clod or block of relatively undisturbed soil is selected from the spoil pile and a smooth surface is cut on it using a knife or shovel. The vaned end of the testing device is inserted into the soil and retracted, leaving an imprint showing the vane pattern. The vanes are then reinserted into the imprint. The device is held firmly and rotated clockwise until the soil fails in shear in a circular pattern around the vanes. The shear strength of the soil is then read off the dial indicator.

The Torevane shear test is similar to the pocket penetrometer test in that 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 shear value obtained. More precise tests should be conducted as the basis for any design work, especially stability or settlement analysis.

2.3.7 Field Moisture-Density Testing.

2.3.7.1 Nuclear-Moisture Density Testing.

The wet field density of a soil can be determined by the nuclear gauge method (AASHTO T 310) 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 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.

2.3.7.2 Rubber Balloon Method.

The rubber balloon method measures in-place wet 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 wet unit weight of the material excavated.

2.3.7.3 Sand Cone Method.

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.

2.3.8 Field Identification of Soils.

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 11 is a flow chart that aids in field classification of clayey, silty, sandy and organic soils.

Table 3 – Particle Description Based Upon Size.

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	-

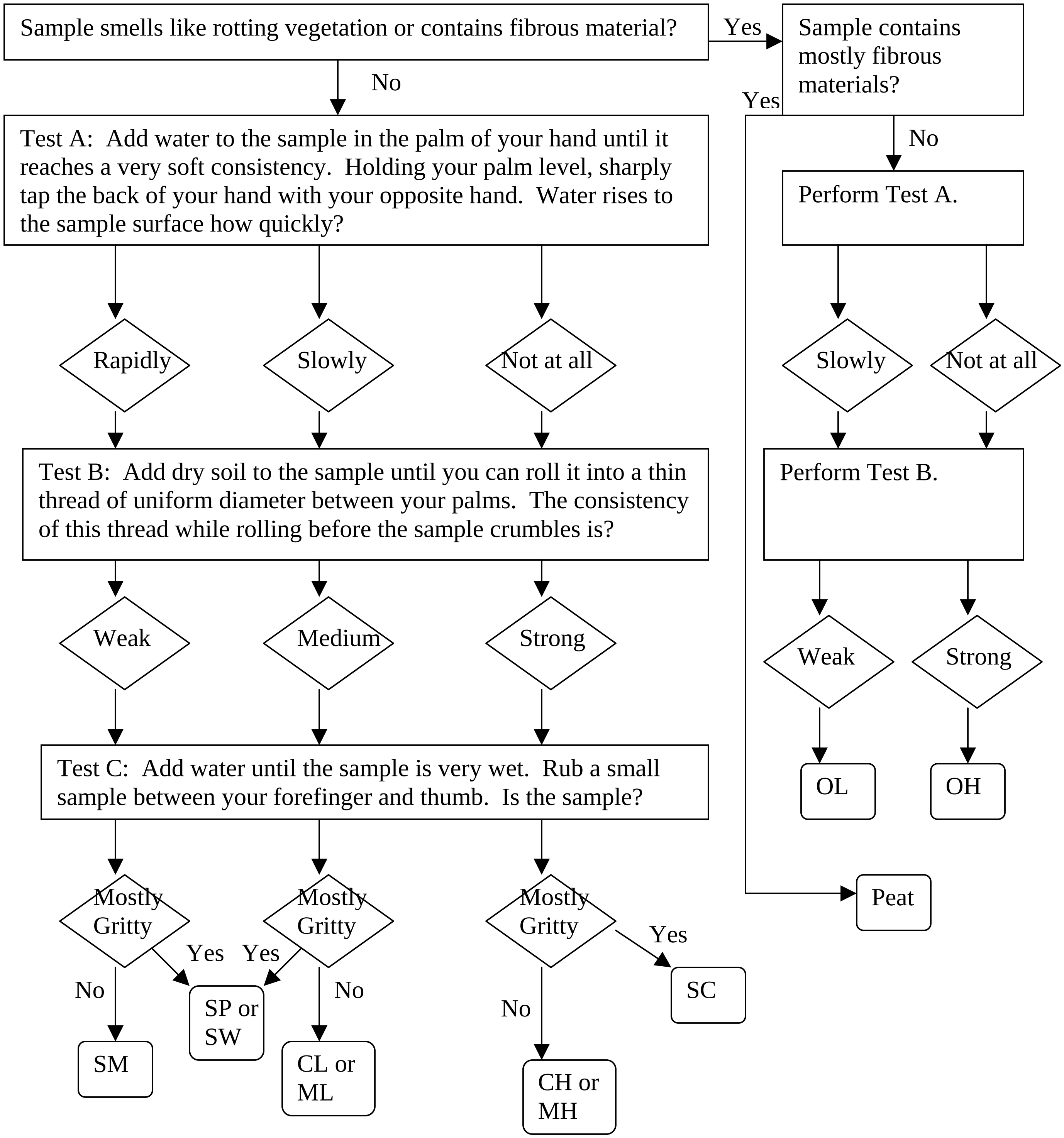


Figure 11 – Flow Chart for Field Identification of Nebraska Soils.

2.3.9 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 rock results 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:

- Rock type, if possible (shale, sandstone or mudstone)
- Color (which may change with weathering/moisture)
- Moisture condition (wet or dry)
- Grain size and shape (if visible)
- Texture (stratified, foliated, thin-bedded, massive, etc.)
- 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 are12 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 can be 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

Chapter 3

Geotechnical Analysis and Design

3.1 Introduction.

With the soil data, profiles and parameters known, geotechnical analysis can be used to design the foundation for a bridge, culvert, retaining wall or road. This chapter will assist readers in identifying soil and foundation concerns that need to be evaluated prior to or during design. It is presumed that the reader has at least minimal familiarity with some technical aspects of engineering related to the behavior of soils, structures and roadways.

3.2 Bridges.

The minimum scope of geotechnical analysis needed for design of highway structures is largely dependent upon site conditions and the type of structure proposed. The following paragraphs address geotechnical analysis with respect to the most common types of bridges used on highway systems.

Highway bridges in Nebraska vary from small stream crossing structures to large and complex bridges across the Platte and Missouri Rivers. The geotechnical aspects of bridge construction are primarily dependent upon the topographic and geologic surroundings. The most common aspects of geotechnical analysis associated with bridge substructures are discussed below.

3.2.1 Slope Stability.

The embankment or fill material supporting the roadway approach to the bridge on either end should be checked for slope stability. Slope geometry can be modeled based upon details from plans or section sheets. Slope stability analysis based upon the simplified Bishop method or sliding wedge method is commonly used. For fill slopes designed based upon Standard Penetration Test (SPT) soil data, a minimum factor of safety (FOS) of 1.5 should be obtained. For fill slopes designed from Shelby tube lab data, a minimum factor of safety of 1.3 or 1.25 is normally acceptable for end slopes or side slopes respectively. When designing cut slopes, a minimum FOS of 1.7 should be used for design based upon SPT soil data, while a minimum FOS of 1.5 should be used for design based upon Shelby tube data.

3.2.2 Settlement.

Settlement can become a serious problem when a new embankment or fill is placed on weak or compressible soils. Settlement of a bridge approach will create a bump in the road and can adversely affect the abutments if downward drag imposes additional loads on the abutment piling or drilled shafts. As a rule of thumb, bridge pier spread footings should always be placed on granular soil with a relative density equal to or greater than 35% or cohesive soil with an unconfined compressive strength of 2.0 tsf (200 kPa) or greater. However, to preclude settlement problems, the bearing capacity of footings on cohesive soil should be limited to a magnitude less than the preconsolidation pressure, as determined from a consolidation test. A settlement analysis should be performed to verify that the calculated settlement of the footing is within specifications. If the calculated settlement exceeds specifications, alternatives such as reducing bearing pressure, use of deeper or larger footings or some method of soil reinforcement should be considered.

3.2.3 Foundations.

Three types of foundations commonly used for highway structures include spread footings, piles and drilled shafts. Foundations should be of sufficient strength to adequately support all anticipated design loads and must be of sufficient size to transfer imposed loads to the soil. Based upon the geologic conditions at the site, imposed loads and esthetic considerations, foundations should be designed to be as economical as possible.

3.2.3.1 Spread Footings.

Spread footings can be used as foundations for retaining walls, culvert wing walls, abutments, grade separation structures and traffic control structures where adequate bearing capacity is available. In stream channels where adequate solid bearing is available, spread footings can be used to support bridge piers. Adequate solid bearing within a stream channel is restricted to conditions of little to no scour and high competency of underlying material. Rock and dense glacial till are two of the few materials that possesses high enough competency to allow use of spread footings beneath bridge piers in stream channels.

Design of spread footings is based upon bearing capacity of the underlying material. Spread footings can be used as bridge foundations only where the bearing capacity of the underlying material is greater than 2.0 tsf (200 kPa). Lower soil bearing capacities can adequately support retaining walls, abutments, grade separation and traffic control structures and culvert wing walls. Loads are commonly transferred to spread footings from the bridge substructure by piers or abutments. Approximately ten feet (3 m) is the maximum depth at which spread footings are considered more economical than pile foundations.

3.2.3.2 Pile Foundations.

Piles transfer supported loads to the soil through two mechanisms, end bearing capacity and side friction. Bearing piles transfer all or most of an imposed load through unstable soils to denser, more stable materials below. Since loads are carried vertically through a bearing pile, the load capacity depends upon the end cross-sectional area of the pile and the bearing strength of the material upon which the end rests. A friction pile supports imposed loads through the frictional resistance developed between the soil through which the pile passes and the exterior surface of the pile. The load capacity of a friction pile thus depends upon the surface area of the pile in contact with the soil and the shear strength of the soil.

Bearing piles are normally driven with a steam, air or diesel hammer. Air or water jets may be used to assist in driving bearing piles. Guidelines have been published which quantify hammer types and restrictions on driving depth (see Section 703 of NDOR's *Standard Specifications for Highway Construction*). Piles are commonly driven to the depth shown in the plans or to a depth of practical refusal. Practical refusal is defined in Section 703.03.7 of *Standard Specifications for Highway Construction*. Piles may be composed of many different types of materials, the most common types of which are discussed in the paragraphs that follow.

3.2.3.2.1 Timber Piles.

Timber piles generally have greater skin friction than other types of piles and are best suited for use as friction piles in unconsolidated soils. Timber piles should not be used in situations where hard driving is required, as the pile tip tends to broom out or crush in those situations. Timber piles have an indefinite life expectancy when placed underwater or when driven below the groundwater level. To increase the useful life of the unsubmerged portion of timber pile, the portion of a pile that will not be emplaced permanently below the ground water table should be pressure treated with a preservative.

Wooden piles are subject to attack by insects, fire, fungi and marine borers unless treated. They have lower resistance to driving forces than other types of piles and may splinter when driven. Wooden piles support the smallest load per unit area of any type of pile, which generally requires using more piles and construction of a larger pile cap.

3.2.3.2.2 Steel Piles.

Steel H-piles consist of rolled wide-flange steel beams available in a variety of sizes, ranging from 8 inches (200 mm) to 14 inches (360 mm) in depth. Due to their relatively small cross-sectional area, steel piles can be driven in dense soils where driving a pile with a larger cross-sectional area would be difficult.

Steel piles can be customized for any project to any desired length by cutting and/or welding. A steel pile is normally driven until it develops a specified resistance. The top is then cut off at a predetermined elevation. A cluster of steel piles is covered with a reinforced concrete pile cap, which distributes the supported load uniformly over all piles in the cluster.

Steel pipe piles varying from 8-72 inches (200-1800 mm) are used for specific applications. Pipe piles are usually driven from the top by a mechanical hammer. Pipe piles may be driven open-ended or closed-ended, and are often filled with concrete. If a pile was driven open-ended, the material inside must be removed by water jet before the pipe can be filled with concrete.

3.2.3.2.3 Cast in Place (CIP) Concrete Piles.

Cast-in-place piles are divided into two general types, the shell type and the shell-less type. Shell piles are constructed by driving a steel casing into the ground, filling it with concrete and leaving the casing in place. The casing normally consists of seamless, welded or spirally wound pipe. Piles may be driven in either an open-end or closed-end configuration. Less commonly used types include longitudinally fluted tapered shells or thin shells that require a mandrel to retain their shape during driving. All shell piles are filled with concrete after driving.

Shell-less piles are constructed by driving a steel pipe, fitted with a tapered shoe, into the soil to the full depth of the pile. The pipe is then pulled up, leaving the shoe at the bottom, and the hole is subsequently filled with concrete. Concrete may be forced downward into the hole under pressure as the pipe is being lifted, which lessens the possibility that earth will become mixed with the concrete. Common variations of shell-less piles include holes that have been bored rather than punched and piles belled at the bottom for extra bearing resistance.

3.2.3.2.4 Precast Concrete Piles.

Precast reinforced concrete piles are manufactured under controlled conditions in various lengths as round, square, hexagonal and octagonal shapes. The process of casting hollow reinforced sections and then joining the sections together by stressed steel cables can be used to create very long piles.

Conventionally reinforced concrete piles consist of reinforcing steel bars surrounded by a spiral steel cage. Prestressed concrete piles are similar in design except that tensioned wire strands replace the reinforcing bars. Conventionally reinforced piles are more susceptible to damage from mishandling or over-driving, but prestressed concrete piles are more difficult to splice together.

3.2.3.2.5 Drilled Shafts.

Drilled shafts have been used for bridge structures around the world, but only recently have they gained widespread acceptance in the United States. Drilled shafts for bridge structures have historically been used only where the depth to rock was relatively shallow. Shallow depth to rock allowed the bottom of a drilled shaft to be anchored in a bedrock socket. Shallow drilled shafts in soil are commonly used by NDOR as foundation elements for traffic control structures (see Section 3.5). Drilled shafts can be advantageous when used in situations where deep excavations and/or large number of piles can delay the construction progress (i.e. railroad viaducts, large structures). Drilled shafts should also be considered when the noise or vibration commonly associated with pile driving operations must be minimized.

3.3 Culverts.

Culverts consist of two basic types, pipe culverts used for hydraulic openings with smaller discharges, and box culverts composed of one or more cells capable of handling much larger discharges. At both ends of a box culvert, wingwalls are constructed to retain earth adjacent to the culvert, preventing soil from sliding into the channel. Horizontal cantilevered walls can be constructed so that they are structurally continuous with the headwall and thus do not require a foundation. T-type and L-type vertical cantilevered wings are usually structurally independent of the headwall and require separate foundations.

Geotechnical analysis and evaluation for culverts should at minimum address the following concerns:

3.3.1 Slope Stability.

The overall slope height and width from the roadway embankment to the flow line or toe of the side slope must be evaluated for slope stability. If the material comprising the slope has insufficient stability to stand at the proposed slope angle, other materials can be substituted, a shallower slope can be created, or the slope can be reinforced using geotextiles or other methods.

3.3.2 Settlement.

Settlement of a culvert can be problematic when a deep embankment is proposed over the culvert or when the culvert must be placed on weak or compressible soils. Settlement directly below and adjacent to the culvert should be calculated based upon laboratory consolidation tests. Differential settlement between the culvert and its wingwalls can be mitigated through the use of articulated joints.

3.3.3 Foundations.

Foundation needs of both the culvert and its wingwalls should be addressed during the design process. Modifying the type of wingwalls or use of articulated joints can mitigate differential settlement between the culvert and its wingwalls.

3.4 Retaining Walls.

A variety of earth retaining structures are currently used in various aspects of highway construction. These include cast-in-place (CIP) concrete and precast concrete gravity walls, mechanically stabilized earth walls (MSE) walls, cantilever sheet pile walls, tieback sheet pile walls, cantilever soldier pile and lagging walls, tangent pile walls, tieback soldier pile and lagging walls, soil nail walls, and many other types.

The selection of a specific type of wall should be based upon economy and ease of construction. A feasibility study should be conducted which addresses the approximate scope of construction for the most feasible types of walls. Cost comparisons between the alternative designs can then be evaluated. Common design loads on earth retaining structures include vertical soil and traffic loads, lateral earth pressure, hydrostatic pressure and wind loads from structures located along the top of the wall.

Design recommendations for retaining walls should include wall type, lateral earth pressures and drainage requirements. Design recommendations for CIP concrete retaining walls should include type of foundation, allowable loads, predicted settlement, lateral earth pressure and drainage requirements. All retaining walls must also be analyzed for external stability and settlement.

Soldier piles, lagging walls and sheet pile walls can be either cantilevered or anchored. Anchored walls can be stabilized using buried “dead men” or by drilled tiebacks. Design recommendations should include elevations of the top and bottom of the walls, size of members, lateral earth pressures, drainage requirements and installation requirements. If anchors are needed, the size, type location, and inclination of the anchors should be specified, as well as the acceptance testing procedure.

3.5 Traffic Control Structures and Light Poles.

Many traffic control structures require only a single foundation element. In soft soils or loose sands below the phreatic surface, pile footings may be required. When the ground surface consists of rock, spread footings may be used. The common foundation element of choice in Nebraska is a reinforced concrete drilled shaft 24 inches (610 mm) to 48 inches (1220 mm) in diameter. Drilled shaft foundations for traffic control structures must be designed to extend for a sufficient depth to resist horizontal loading, overturning moments, and torque from wind loads.

Designers of drilled shafts must consider lateral as well as vertical loads. Numerous references for drilled shafts are available, with FHWA Publication FHWA-H1-88-042 (August 1988) particularly useful for design.

3.6 Roadways.

3.6.1 Settlement Analysis.

Consolidation settlement takes place when the weight of an embankment exceeds the load previously imposed on the underlying strata. The pore water contained in the voids between soil particles initially carries the extra load, creating excess pore water pressure. As excess pore water pressure dissipates and the void space is compressed, load is transferred to an irregular lattice formed by physical contact between the soil particles. The total magnitude of surface settlement is directly proportional to the reduction in void space between the soil particles.

Settlement consists of primary and secondary consolidation. Primary consolidation is represented by the portion of the consolidation curve in which the reduction in void ratio is directly associated with dissipation of the excess pore water pressure. Speed of dissipation of pore water pressure is a function of hydraulic conductivity. Hydraulic conductivity is a function of particle size and soil structure. Granular materials are generally capable of dissipating pore water pressure as fast as a load can be applied. Some thick deposits of clay may not be able to achieve pore water pressure equilibrium for decades or even centuries.

Secondary consolidation begins after full dissipation of excess pore water pressure. Secondary consolidation is normally a problem only when dealing with organic soils such as peat. Peat can exhibit secondary consolidation equal to or greater in magnitude than primary consolidation. Hence peat deposits are often totally or partially removed as the primary method of settlement mitigation. With nonorganic soils, secondary consolidation is normally less than ten percent of primary consolidation and thus does not constitute a problem.

3.6.2 Stability Analysis.

Subgrade stability must consider the short-term and long-term behavior of the subgrade. The subgrade should be designed to support the weight of equipment used during construction, as well as to support the roadway throughout its design life. Throughout the life of the pavement, the stress level on the subgrade should be maintained below an upper limit defined by the resilient modulus of the subgrade (determined by AASHTO T 274). Exceeding the limit of the resilient modulus for the subgrade will result in loss of pavement support and subsequent pavement failure.

3.6.3 Drains and Filters.

Internal drainage of the pavement system and the subgrade beneath can have a significant effect on pavement performance. As the water table rises within a fine-grained soil, the soil becomes more saturated, resulting in a reduction in the soil's subgrade stabilty. Subgrade drainage conditions may be classified into one of four categories, good, fair, poor or very poor.

Drainage classification cannot be made based solely upon soil tests, as all classifications of drainage may exist within most types of soil under varying conditions. The extent to which adverse conditions may develop also depends upon local topography and the road profile.

Good internal drainage is characterized by conditions where the permanent water table is low enough that the underlying soil will never become saturated by capillary action. The local topography allows surface water to be removed without saturating the underlying subgrade from above. There are no other drainage conditions that will produce saturation or instability of the subgrade under good drainage conditions.

A permanently low water table also characterizes fair internal drainage conditions, but the possibility of a temporary higher water table does exist. There is the possibility that surface water may not drain off rapidly due to minor irregularities in the topography. There are no internal drainage characteristics or other conditions that will produce instability in the subgrade under fair drainage conditions.

A temporary high water table characterizes poor drainage conditions. Surface water may not drain off rapidly due to topography or there exists internal drainage characteristics or other causes which may result in saturation or instability of the subgrade due to the presence of water.

A water table that is permanently high characterizes very poor drainage conditions, as does surface water that will not drain off due to topography or the existence of internal drainage characteristics that produce saturation or instability in the subgrade. The four drainage classes as determined by various soil types, moisture conditions, road profile and cross section and grade is illustrated in Table 6.

For a specific section of proposed roadway, pavement system internal drainage can be improved primarily though use of subgrade materials with higher hydraulic conductivity. For an existing roadway, pavement system internal drainage can be improved in the most economical manner by cutting deeper ditches.

Table 6 – Pavement Systems Internal Drainage Classification.

Earthwork Involved in Construction				Soil Type									
Less than 3' (0.9 m) of Fill to Less than 6' (1.8 m) of Cut		More than 6' (1.8 m) of Cut*		ML, MH, CH					CL				
Road Cross Section													
Ditch > 3' (0.9 m)	Ditch < 3' (0.9 m)	Ditch > 3' (0.9 m)	Ditch < 3' (0.9 m)	High Water Table	Very Wet	Wet	Moist	Dry	High Water Table	Very Wet	Wet	Moist	Dry
Grade				Drainage Classification									
			<0.5%	VP	VP	P	P	P	VP	P	P	P	F
	<0.5%		≥0.5%	VP	P	P	P	F	P	P	P	F	F
	≥0.5%	<0.5%		P	P	P	F	F	P	P	F	F	G
<0.5%		≥0.5%		P	P	F	F	F	P	F	F	G	G
≥0.5%				P	F	F	F	G	F	F	G	G	G

* Fill greater than 6' (1.8 m) should be engineered to have good drainage.

Earthwork Involved in Construction				Soil Type									
Less than 3' (0.9 m) of Fill to Less than 6' (1.8 m) of Cut		More than 6' (1.8 m) of Cut*		SW or SP					GW or GP				
Road Cross Section													
Ditch > 3' (0.9 m)	Ditch < 3' (0.9 m)	Ditch > 3' (0.9 m)	Ditch < 3' (0.9 m)	High Water Table	Very Wet	Wet	Moist	Dry	High Water Table	Very Wet	Wet	Moist	Dry
Grade				Drainage Classification									
			<0.5%	P	P	P	F	F	P	P	F	F	F
	<0.5%		≥0.5%	P	P	F	F	F	P	F	F	F	G
	≥0.5%	<0.5%		P	F	F	F	G	F	F	F	G	G
<0.5%		≥0.5%		F	F	F	G	G	F	F	G	G	G
≥0.5%				F	F	G	G	G	F	G	G	G	G

VP = very poor P = poor F = fair G = good

3.6.3.1 Drains for Pavements.

Pavement systems frequently become saturated during periods of heavy precipitation, primarily by movement of water downward through the pavement. At the same time, pavement may be subjected to repetitive loading by vehicular traffic. Periodic loading and unloading induces momentary increases in pore water pressure within the underlying soil, the base course(s) and the subgrade. Rigid pavements built directly on top of clay subgrades and subjected to heavy traffic loading during periods of precipitation experience a phenomenon called "pumping". Pumping refers to the ejection of water and suspended solids from beneath the pavement during loading. Early efforts to mitigate this phenomenon by building pavements on dense granular subbases appeared to be successful.

However, long-term studies showed that repetitive loading on a rigid slab causes high water pressure even in dense granular materials. The high pore water pressure results in water velocities as high as 6 m/sec (20 ft/sec), operating over very short distances. A velocity of 6 m/sec will eject the fines from a dense granular subbase through the pavement joints and cracks, resulting in a significant reduction in pavement support and ultimately pavement failure.

A second study revealed that the same repetitive loads on rigid pavement placed on a coarse, open graded aggregate reduced water velocity to about 1.8 m/sec (6 ft/sec). Results indicated that the open graded aggregate significantly reduced water pressure so that water velocities generated during traffic loading were not sufficient to cause loss of fines.

Moisture within the pavement system readily crosses the interfaces between the pavement, base, subbase and subgrade. Performance of open graded aggregate over a clay subgrade can be degraded by fines from the subgrade migrating into the voids of the open graded aggregate base. A layer of geotextile material is sometimes installed as barrier to prevent migration of fines between layers.

Longitudinal pavement edge drains have been mandated by some states, but even these do not provide an easy solution to pavement drainage problems. The purpose of an edge drain is to remove water from beneath the pavement. However, edge drains do not generally reduce the moisture content of the subgrade soil. The moisture content of the subgrade soil may remain at or near saturation all year round. This has been documented for pavements, with and without edge drains and for all types of subgrade materials. If granular materials are used to provide drainage under a pavement, the system must be designed to allow flow of water without loss of particles from the granular layer or from the subgrade below.

3.6.3.2 Drains to Promote Consolidation.

Embankment construction above thick, high moisture content, fine-grained soils may result in consolidation problems during the project life. If not alleviated during construction, differential settlement may continue for years or even decades. The most popular method of stabilizing thick deposits of fine-grained soil has historically been lowering the moisture content through use of vertical sand drains. Thin, prefabricated wick drains are now used, some of which can be stitched into the ground by machinery. Modern wick drains consist of a deformed plastic extrusion wrapped in a geotextile fabric. Most require minimum construction time, equipment and personnel for installation.

3.6.3.3 Drains for Cut Sections.

When the depth of cut section for a roadway is deep enough to intersect the groundwater table, seepage begins immediately along the face of the slope. Left unchecked, a small amount of seepage can result in sloughing of the slope, leading to eventual instability across the whole slope face.

When the elevation of the ground water table can be established by a soil survey, the elevation and extent of drains should be detailed in the design plans. Two methods are commonly used for draining slope faces. The first is a pipe collector placed in a longitudinal trench and backfilled with a suitable granular filter material. The second method involves construction of a reverse filter on the face of the slope. To construct a reverse filter, the finest material is first used to blanket the soil slope. The finer material is then covered by coarser material with the coarsest material placed on the surface. A reverse filter drain works best if the slopes are steeper than 3H to 1V.

Use of French drains is another possibility. A French drain consists of a trench, lined with suitable filter fabric and backfilled with an open graded coarse stone (but no collector pipe). The hydraulic conductivity of the coarse stone, the cross section of the trench and the trench gradient must be adequate to effectively move the required quantity of water from the slope for the French drain to be effective.

3.6.3.4 Filter Specifications.

Natural filters consist of two or more layers of material. One layer, called the drainage filter layer, is designed to collect water and remove it from the system. The other layer, called the protected layer, contains excess water that must be removed. To permit easy flow and to “draw” water into the drain, it is necessary to have the hydraulic conductivity of the filter material considerably greater than the hydraulic conductivity of the protected material. The requirements for each layer are expressed in terms of particle dimensions obtained from a particle-size distribution curve. Thus D_{15} refers to the particle diameter at which 15 percent of the soil material (by weight) is smaller. Filter requirements are commonly specified as a series of ratio of various diameters.

To avoid head loss in the filter, D_{15} of the filter layer divided by D_{15} of the protected layer should be greater than four. This ratio ensures hydraulic conductivity of the filter material is adequate for the drainage system.

To avoid movement of particles from the protected layer into the filter layer, D_{15} of the filter layer divided by D_{85} of the protected layer should be less than five, D_{50} of the filter layer divided by D_{50} of the protected layer should be less than twenty-five, and D_{15} of the filter layer divided by D_{15} of the protected layer should be less than twenty. For a uniform protected layer (coefficient of uniformity less than 1.5), D_{15} of the filter layer divided by D_{85} of the protected layer may be increased to six. For a well-graded material (coefficient of uniformity greater than 4), D_{15} of the filter layer divided by D_{15} of the protected layer may be increased to forty.

To avoid movement of the filter material into perforated drain pipes, D_{85} of the filter layer divided by the slot width of the pipe should be greater than 1.2-1.4. For drainage pipes with holes, D_{85} of the filter layer divided by the hole diameter should be greater than 1.2-1.4. To avoid particle segregation, the filter material should contain no particles larger than three inches in diameter. To avoid internal movement of fines, the filter material should have no greater than five percent passing the No. 200 (0.075 mm) sieve.

Materials available for use may render single filters insufficient to meet the above requirements, necessitating the use of multiple filter layers. Suitable geosynthetic fabrics can also be used in place of most natural filter materials. Fabric is commonly wrapped around the drainage pipe to satisfy opening requirements or is used to line a trench to protect against movement of fines into the collector.

3.6.4 Frost Susceptible Subgrades.

Fine-grained subgrades combined with free water often create the problem of frost heave or frost boil. Prolonged cold weather results in frost penetration deep into the subgrade. Water moving through the subgrade freezes from the surface to frost depth, developing a series of ice lenses. As the process freeze-thaw process continues, the size of lenses significantly increases. Since freezing is a purification process, frost lenses consist of almost pure water.

Frost susceptible materials in the subgrade must be located and remedial measures completed during construction or frost damage will occur. The following criteria are commonly used to determine frost susceptibility:

- The level of capillary rise must be higher than the depth of frost penetration. Level of capillary rise is dependent upon the soil type and the groundwater elevation.
- The soil usually contains sixty-five percent or more silt and fine sand.
- The plasticity index is usually less than twelve.

Frost penetration under a pavement system normally ranges from approximately thirty-five inches near northern Nebraska State line to twenty-five inches around the southern Nebraska State line. Maximum depths of frost penetration may be slightly greater but this will occur only during the most severe winters. The degree of frost susceptibility for various USCS soil classifications is shown in Table 7.

Ice lenses represent the most detrimental type of frost heave conditions. Frost heave problems may also be caused by water infiltration through the pavement surface with subsequent freezing immediately below the pavement. Tenting at pavement joints can result from water penetrating at the joints and freezing when it comes into contact with the frozen subgrade.

During spring of each year, the pavement and subgrade commonly thaw from the top down. Water from thawing frost lenses (or infiltrating through the pavement surface) may be unable drain through the frozen soil below. This condition creates a problem known as frost boil. Under frost boil conditions, pooling water may cause a complete loss of subgrade stability, characterized by a rapid and dramatic failure of the pavement surface.

Detrimental frost heave may also occur where there is an abrupt change in soil particle size from one layer to the next. A change in texture usually represents a change in hydraulic conductivity, which causes the water to pool within the layer having the higher conductivity. The most common method of correction for this problem is excavation and replacement if only small areas are involved. If larger areas are involved, the soils above and below the textural boundary are often mixed to provide a transition layer. Such textural changes are common when transitioning from a cut to a fill section or when there are textural differences between the A, B and/or C-horizons of a natural soil.

Table 7 – Frost Susceptibility of Soils.

Type of Soil	% Finer than 0.02 mm by Weight	USCS Soil Class	Degree of Frost Susceptibility
Gravelly Soils	3 to 10	GW, GP GW-GM GP-GM	Negigible to Low
Gravelly Soils	10 to 20	GW, GP GW-GM GP-GM	Low to Medium
Sands	3 to 15	SW, SP, SM SW-SM SP-SM	Low to Medium
Gravelly Soils	Greater Than 20	GM,GC	High
Sands	Greater Than 15	SM, SC	High
Clays PI > 12		CL, CH	High
All Silts		ML, MH	Very High
Fine Silty Sands	Greater Than 15	SM	Very High
Clays PI < 12		CL CL-ML	Very High
Varved Clays		CL, ML SM, CH	Very High

Chapter 4

Soil Modification

4.1 Introduction.

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.

4.2 Surface and/or Subgrade Treatment.

4.2.1 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 some type of design or soil modification.

AASHTO M 147 specifies various particle size gradations for material 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 8 – AASHTO M147 Grade Requirements for Soils Used as Subbase Materials, Base Courses and Surface Courses.

		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	8-Mar	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

4.2.2 Unsuitable Soils.

As a general rule, soil used in highway construction applications should have a minimum dry unit weight of 90 pounds per cubic foot, an organic content less than 2%, and a liquid limit of less than 50%. Soil outside of these limits will normally require some type of modification to alleviate adverse characteristics.

If a soil with a liquid limit (LL) higher than 50% is present as the subgrade or if soil with a LL greater than 50% 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 the high LL soil is unstable. If the soil is stable, no treatment is necessary. Treatment 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.

4.2.3 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.

4.2.3.1 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 covered in Chapter 3.

4.2.3.2 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 be sufficient. 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.

4.2.3.3 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. The addition of lime to medium to fine-grained soils will produce 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.

4.2.3.4 Lime Stabilization.

During periods of 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 water penetration and freeze-thaw cycles.

4.2.3.5 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, $Ca(OH)_2$, 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.

4.2.3.6 Calcium Chloride Stabilization.

Calcium chloride ($CaCl_2$) 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. $CaCl_2$ is highly soluble in, raises the surface tension of and lowers the freezing point of water.

Calcium chloride replaces the Na^+ ions within the diffuse double layers of sodium montmorillonites with Ca^{++} ions, reducing the thickness of that layer, thereby decreasing the plasticity and increasing the strength of the soil. $CaCl_2$ 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.

4.2.3.7 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 silicia (SiO_2), aluminia (Al_2O_3), ferric oxide (Fe_2O_3) and calcium oxide (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.

4.2.3.8 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 form 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.

4.2.3.9 Over-Excavation and Replacement of Soil.

Removal of a weak subgrade soil and replacement with more suitable material is a commonly used method of soil 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 traffic loads over a 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 granular 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 granular material used as backfill may also be adversely affected by submersion. 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 backfill, material from the soft subgrade may migrate into the granular backfill, significantly reducing the effectiveness of that layer over time.

High ground water in combination with a silty or clayey subgrade that loses strength when disturbed can also be problematic. These types of soils are commonly referred to as "sensitive soils". Over-excavation to remove the upper layer(s) of a sensitive soil can dramatically reduce strength in lower layers. Over-excavation and replacement of surface material under these conditions will often result in a thin platform of compacted material floating on top of unstable layers with significantly less strength.

Problems associated with sensitive soil and a high ground water level may be mitigated by several methods. The first step involves confirming that the soil is sensitive. Sensitivity is defined as the ratio of the unconfined compressive strength of an undisturbed sample divided by the unconfined compressive strength of a remolded sample. Most soils have sensitivities ranging between 2 and 4, while sensitive soils have values between 4 and 8.

If topography permits and the area of sensitive soil or high ground water is not extensive, drainage channels or French drains can be constructed to lower the ground water table in the problem area. If the ground water level cannot be lowered, total depth of the proposed excavation below ground water level should be evaluated with respect to economics. The expense and probability of encountering a problem while excavating below the groundwater level increase exponentially with depth while the associated benefits commonly increase only linearly. Admixtures or other solutions may prove more cost effective.

Proper equipment must be selected to complete any excavation in sensitive soils. Equipment with rubber tires has concentrated wheel loads that place considerable stress on the underlying soil. Tracked equipment surface loads are less concentrated and are thus preferential to wheeled equipment. Equipment that operates from outside the immediate project site (cranes or backhoes with extended booms) is best when seeking to minimize disturbance of underlying sensitive soil.

4.2.3.10 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, meshes 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, GP or SW, SP). Current backfill requirements are designed to produce a freely draining structure with a soil reinforcement friction factor (tan δ) 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. These elements 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 predicable properties once emplaced, and are able to withstand degradation under subsurface conditions. Synthetic fabrics are commonly referred to as geotextiles, a broad classification encompassing 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 9 lists some of the more common uses of geosynthetic products within the transportation industry. Some of the American Society for Testing and Materials (ASTM) procedures for determining the mechanical, hydraulic and durability properties of geotextiles are detailed in Table 10.

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

Table 9 – Common Uses of Geotextiles.

Application	**Use(s) of Geotextile**
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

various functions of geotextiles and relating these functions to improvement in soil properties.

Table 10 – ASTM Procedures for Geotextile Testing.

Topic	ASTM
Basic properties, sampling	D 4354
Test method for deterioration and durability	D 4355
Test method for permittivity	D 4491
Test method for tearing strength – trapezoidal	D 4533
Test method for tensile properties – wide strip	D 4595
Test method for breaking load and elongation	D 4632
Test method for transmissivity	D 4716
Test method for size distribution of openings	D 4751
Test method for puncture resistance	D 4833
Test method for seam strength	D 4884
Test method for abrasion resistance	D 4886

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.

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 3.6.3.4 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 12 and 13.

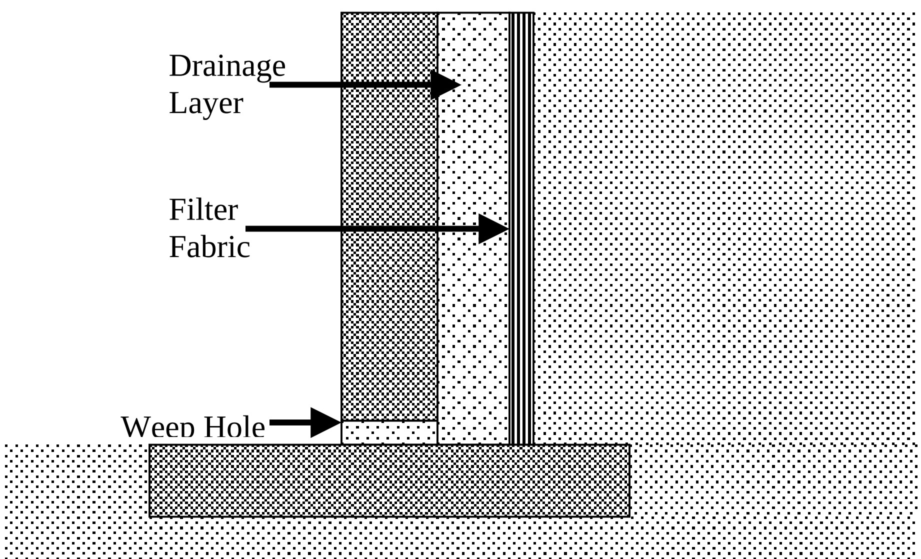


Figure 12 – Geotextile Used as a Filter Fabric Behind a Retaining Wall.

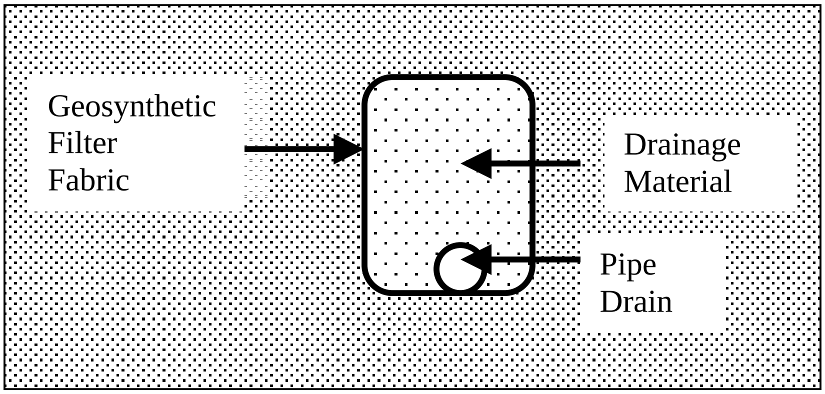


Figure 13 – Geotextile Filter Fabric Used in Trench Drain.

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 14.

Geocomposite strip drains have all but replaced sand and waxed cardboard “wick” drains in surcharge applications on cohesive soils, as illustrated in Figure 15. Drains used in this application perform a temporary function of accelerating the consolidation of a clay layer under a surcharge load.

Depending upon the properties of the geosynethic 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.

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

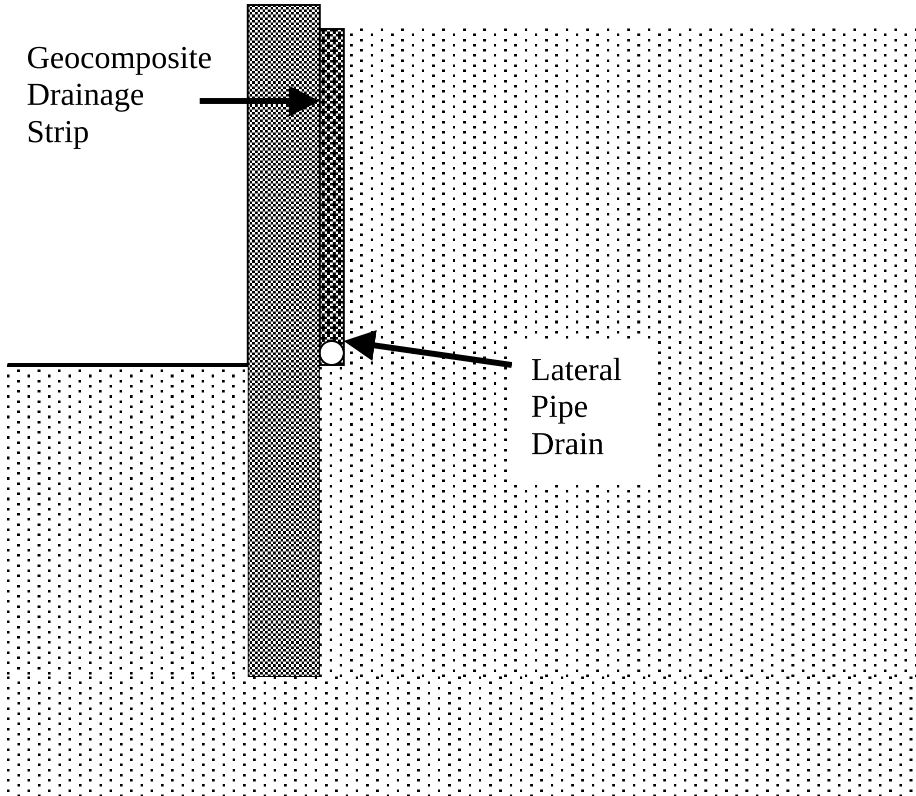


Figure 14 – Geocomposite Used as a Drain Behind a Retaining Wall.

upward into the granular layer, providing an upward and outward drainage path for water from the saturated soil below.

Geosysnthetic fabrics also allow reduction in the quantity of fill required beneath a foundation. A reduction in thickness of the granular material layer by one-quarter to one-half of the originally required thickness can commonly be achieved beneath spread or strip footings through the use of geosynthetics. A reduction in granular layer thickness results in reduction in the depth of cutting required, as well as reduction of 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 by the reduced quantity of aggregate required. Subgrade strength, magnitude of traffic loading and properties of the geosynthetic material all influence the minimum 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.

Several geosynthetic material manufacturers have developed design software applications that are available at no cost. This software is relatively easy to learn and can be used to determine quantities of natural versus geosynthetic materials required for specific projects. Each software application is valid only for the products from that particular manufacturer.

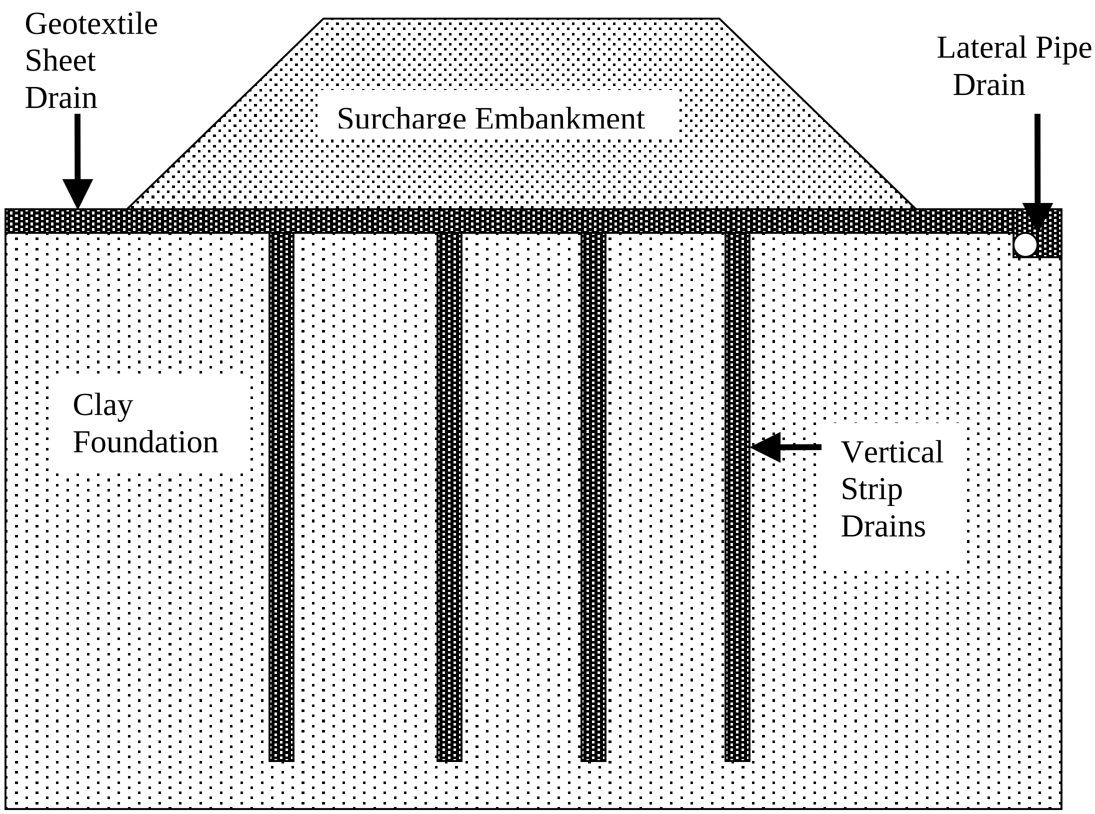


Figure 15 – Consolidation Surcharge Loading Using Strip and Sheet Geocomposite Drains.

Chapter 5

Construction Procedures and Instrumentation

5.1 Introduction.

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 a 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 is properly placed and compacted, the resulting soil mass has strength and support properties that are more uniform than the natural soil strata.

When soil is used as a foundation material, it is desirable for the soil to have certain characteristics. 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 excessive 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 shear strength with minimal changes in volume after compaction and can be emplaced under most moisture conditions. Compacted silt is stable, and develops fairly high shear strength, but has a tendency to exhibit unacceptable volume changes with variations in moisture content. Silty soils can prove very difficult to compact when the soil is wet or under rainy conditions. Compacted clay soils can develop high shear 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.

5.2 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 side slope or bearing failure. A factor of safety of 1.3 will be required against end slope failure.

5.2.1 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 with various 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 surcharge load above the compressible layer to accelerate settlement. The process is known as surcharging.
- Use of sand or wick drains in conjunction with surcharging to accelerate settlement.
- Use of instrumentation and time delays when the settlement problem is located adjacent to structural foundations, under paving and at other locations sensitive to excessive settlement. Bridge approaches are often constructed after other sections of pavement to allow additional time for consolidation at these critical locations. Instrumentation can be installed and used to monitor settlement so that construction of approach pavement is delayed only until primary settlement is completed.
- Vibrocompaction if granular materials are used to construct the embankment.
- Dynamic compaction of material used to construct the embankment by dropping a weight from a specified height in a regular pattern over the embankment surface.

The rate of settlement depends upon the hydraulic conductivity and thickness of the consolidating layer(s), the shape and length of the drainage pattern, and the magnitude of excess pore water pressure. 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 permeable 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.

5.2.2 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 at embankment midslope.
- Installation of some type of a drainage system.
- Installation of some type of structural support system, such as a soil key, retaining wall, soil nails, drilled shafts or micropiles.
- Soil reinforcement consisting of a geogrid or geotextiles.

5.2.3 Reinforcement.

Construction of a reinforced soil slope should be considered when there is insufficient right-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 thickness of soil layers.

5.3 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 (FOS) 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 increase the stability of unstable slopes. 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 may also remove fine materials from the slope face resulting in surface instability. The most effective way of dealing with groundwater seepage is to extract the water at a level higher 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 cut 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.

5.4 Surface Compaction Methods and Procedures.

Construction of a fill section consists of two distinct operations, placing and spreading of material to create layers and the subsequent layer compaction process. Compaction is normally the more critical of the two steps and its rate often controls the rate of progress for the entire project. 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 many different types 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. On 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 (700-4,200 kN/m^2), 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 is continually moving upward. Rising of the roller projections through a 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 with three to five passes, while smaller, lighter units can compact lifts up to six inches in thickness with the same number of passes. Sheep's foot rollers are well suited for compacting clay and silt-clay soils. They are not recommended for granular 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 creating a "wobbly-wheel" effect, where each wheel follows a weaving path. Pneumatic tire rollers are normally equipped with a weight or ballast box, which allows easy adjustment of the roller's weight. Pneumatic tire rollers are available in a wide variety of sizes and weights, the most common being 50-ton (45,000 kg) rollers.

Pneumatic tire rollers are the most versatile type of equipment for general compaction use. They are capable of compacting both cohesive and granular soils. Lighter rollers (20 tons or 20,000 kg) are generally capable of compacting lifts varying from two to six inches thick 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) is 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 drum 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 rollers 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 soil particles into a denser configuration. Vibratory compactors achieve best results when operated at speeds of 2-4 mph (3-6 km/hr). Smooth drum vibrators are most effective when compacting granular soils, where lift thickness up to 3 ft (1m) can compacted to near maximum density. As the percentage of fine material increases, thickness of layer being compacted must be reduced. Vibratory pneumatic wheel rollers have also been used to compact granular soils, but the lift thickness for effective compaction is generally 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, however, be used to effectively seal the soil surface being worked at the end of each day. Sealing provides a smooth upper surface, which causes rainfall occurring during the night to run off. If the surface is not sealed, rainwater will soak into the upper layers, and create a soft, wet working surface for the next day. Table 11 provides a generalized summary that relates soil types to the characteristics of equipment considered suitable for achieving adequate compaction. Table 12 outlines required compaction requirements for typical NDOR projects.

For any method of compaction, maximum density and optimum moisture content vary with the type of soil. Well-graded soils containing gravel, sand, silt and clay have higher maximum densities and lower optimum moisture contents. Poorly graded granular soils and clayey soils have lower maximum densities and higher optimum moisture contents. Figure 16 illustrates twenty-nine moisture density curves representing soils typical of Nebraska.

Table 11 – 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

Table 12 – Recommended NDOR Compaction Requirements.

	Soil Type	Depth Below Finish Subgrade	Percent Density	Moisture Requirements Minimum	Moisture Requirements Maximum
Embankment/Roadway Grading, including driveways which will receive Concrete Pavement	Silt-Clay	Upper 3′	96±3	Opt.-3%	Opt.+3%
	" "	At depths greater than 3′	95 Min.	**	Opt.+2%
	Granular	All depths	100 Min.	**	**
Embankment, including roads, detours, temporary roads, and driveways which will receive Flexible Pavement	Silt-Clay	Upper 3′	100 Min.	**	Opt.+1%
	" "	At depths greater than 3′	95 Min.	**	Opt.+2%
	Granular	All depths	100 Min.	**	**
Embankment of Roads which will not be surfaced	All	All depths	95 Min.	**	**
Embankment of driveways not to be surfaced	All	All depths	Class I	(See Specifications)	
Subgrade Preparation (Concrete Pavement)	Silt-Clay	The upper 6″ of subgrade soil	96±3	Opt.-3%	Opt.+3%
	Granular	" "	100 Min.	**	**
Subgrade Preparation (Flexible Pavement)	Silt-Clay	The upper 6″ of subgrade soil	100 Min.	**	Opt.+1%
	Granular	" "	100 Min.	**	**
Bituminous Pavement Patching	All	Underlying Material	100 Min.	**	**
Foundation Course	–	–	100 Min.	(See Specifications)	

** Moisture as necessary to obtain density

The densities and moisture contents of a soil being used as a construction material should be compared with the compaction and moisture specifications. If the moisture content of the soil being placed is higher than specified limits, the contractor must reduce the moisture content. This can be accomplished through aeration or through the use of admixtures. If the moisture content of the soil being placed is lower than specified limits, water must be added. Moisture density samples should be taken from each lift and tested in accordance with one of the methods shown in the NDOR's *Standard Methods of Sampling and Testing Materials*. The frequency of testing should meet or exceed the sampling requirements outlined in NDOR's *Materials Sampling Guide*.

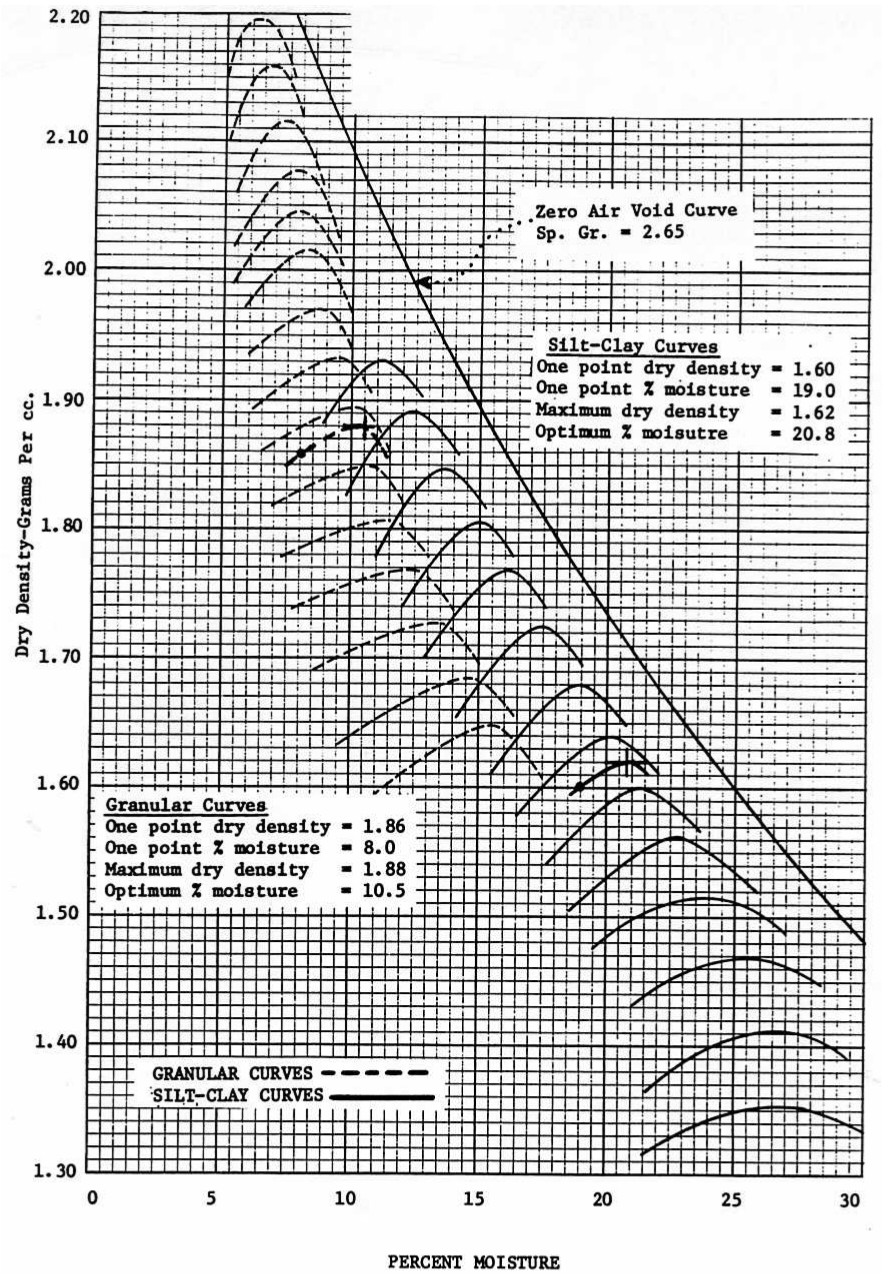


Figure 16 – Moisture Density Curves for Typical Nebraska Soils.

Dry unit density can be obtained by dividing wet unit density by one plus the moisture content. A large number of moisture content determinations will be necessary to ensure that proper moisture and density are obtained when constructing embankments or subgrades. Moisture content can be determined very quickly and inexpensively using the procedure outlined in ASTM D 4643, *Standard Test Method for Determination of Water (Moisture) Content of the Soil by the Microwave Oven Method*. A quick check of the accuracy of field moisture content determinations can be made by comparing the values obtained with the zero air voids (ZAV) curve. Since no known method of compaction decreases air voids in a field environment to zero, actual values must always fall to the left of the ZAV curve. Values falling on or to the right of the ZAV curve must be in error. The actual procedure for calculating and plotting the ZAV curve is discussed in greater detail in Appendix E – Compaction.

Problems with meeting compaction specifications are commonly discovered when lower/ higher densities or lower/higher water contents than specified are measured at the project site. Adding water to the soil immediately before mixing and compacting commonly solves the problem of soil that is too dry. Economic considerations demand that aerating and drying soil by disking/scarifying be considered and carefully evaluated before the decision is made to remove and replace any in-situ soil. Some possible solutions to common density problems are illustrated in Table 13.

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 engineering characteristics for very thick layers of soil. These techniques include vibrocompaction, vibroreplacement and dynamic deep compaction. Deep ground treatment techniques such as these may offer practical, economically viable alternatives to the construction of deep foundations for some projects.

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 outward horizontally. 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 vibrocompaction equipment incorporates water jets directly into the vibrator to assist in penetration and densification of granular material.

Table 13 – Troubleshooting Chart for Compaction Problems.

Problem: *Measured density is too low*.

Possible Cause	Solution
Wrong compaction curve used	Check soil identification
Wrong compaction equipment	Use equipment suited to soil type and compaction specifications
Too few passes of compactor over each lift of soil	Increase number of passes per lift
Lift thickness is excessive	Decrease lift thickness
Error in test procedure	Recheck procedure/Recalibrate apparatus
Granular materials	Mix in-situ soil with silt-clay before compacting
Moisture content is not within specifications	Check moisture content, add or remove water
Change in soil type	Check soil index properties to see if soil type has changed

Problem: *Measured density is too high.*

Possible Cause	Solution
Wrong compaction curve used	Check soil identification
Compaction equipment is too heavy	Remove any auxiliary weight(s); use smaller equipment
Too many passes of compactor over each lift	Reduce number of passes of compactor over each lift
Lift thickness is too thin	Increase lift thickness
Error in test procedure	Recheck procedure/Recalibrate apparatus
Moisture content is not within Specifications	Measure moisture content again; add or remove water
Change in soil type	Check soil index properties to see if soil type has changed

Vibroreplacement works much the same way as vibrocompaction, except crushed stone or gravel is added to the top of a column and vibrated downward 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 vertical soil column partially supported by stone. The overall capacity of a site treated by vibroreplacement depends upon the spacing and size of the soil-stone columns as well the bearing capacity and shear strength of the natural soil.

Dynamic deep compaction is a method where a heavy (2-50 ton) weight is dropped (usually by a crane) 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 performed, each at a different grid intersection. This process can be used successfully with most types of soil, but it is particularly effective for soils consisting of building rubble or buried garbage fills. A depression is created at each location where the weight impacts. This depression must be filled in and compacted using normal surface compaction methods.

The intent of deep compaction is to improve a 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 the need to construct deep foundations or to remove and replace a significant thickness of surface material.

5.5 Pile Driving and Testing.

Pile installation involves furnishing, driving, trimming, and testing of bearing and sheet piles. Specifications for pile driving can be found in Section 703 of NDOR's *Standard Specifications for Highway Construction*, which is available online at http://doroads.nol.org/ref-man. Pile driving contractors are required to submit completed hammer data sheets to the NDOR bridge engineer for wave equation analysis before each project begins. A blank hammer data sheet is shown on p. 406 of website shown above. If wave equation analysis indicates that the hammer system will be unable to drive the pile to minimum penetration without damage to the pile, the contractor will be required to modify the hammer system and submit a revised hammer data sheet. Hammers cannot be changed or replaced without authorization from the geotechnical engineer.

Determination of bearing capacity for driven piles is detailed in 703.03.4 of NDOR's *Standard Specifications for Highway Construction* while load testing for piles is outlined in 703.03.5. The geotechnical engineer monitors driving of plan specified test piles using a pile driving analyzer. Test piles are then incorporated into the foundation as load bearing piles. Specifications defining "practical refusal" for pile driving can be found in 703.03.7.

5.6 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 17.

MSE walls now 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

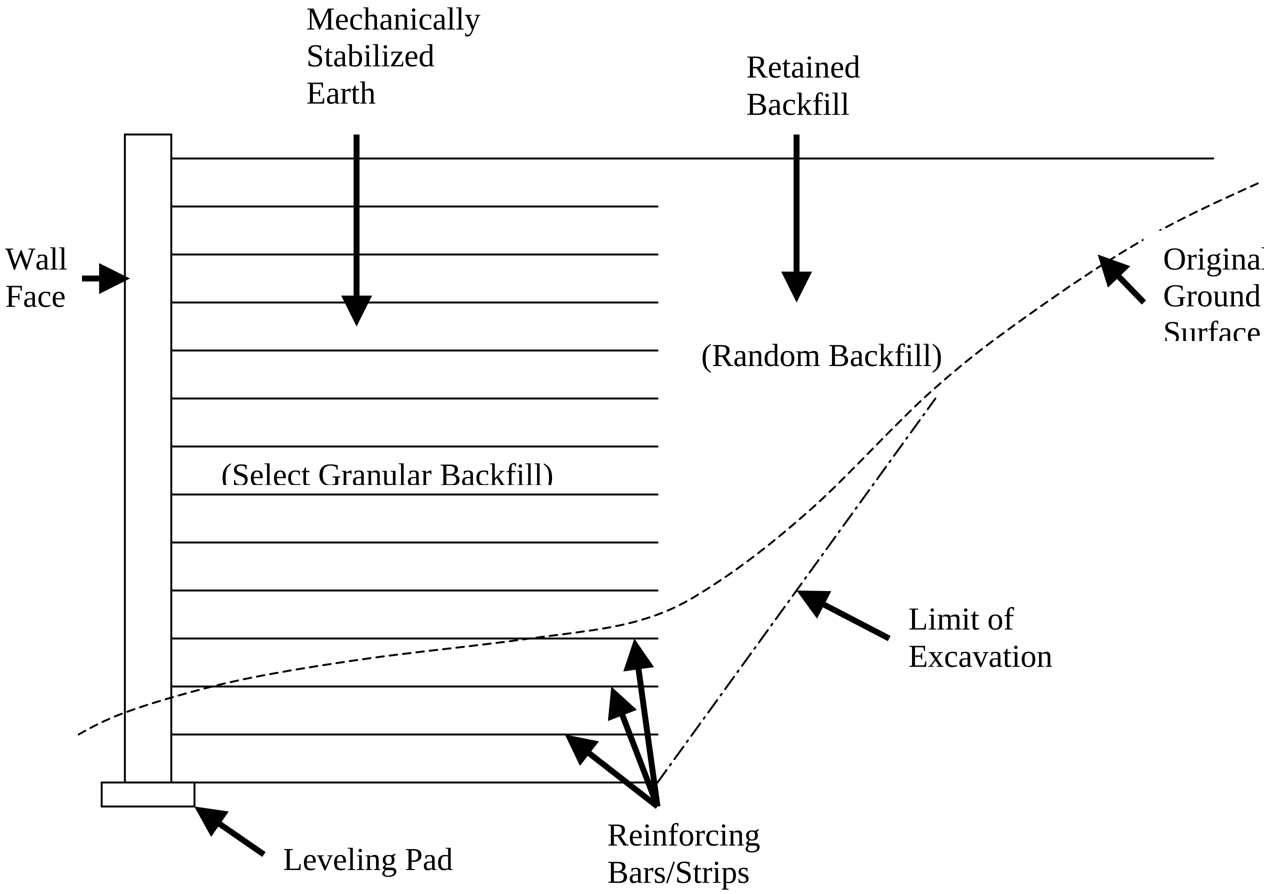


Figure 17 – Cross Section of Mechanically Stabilized Earth Wall.

construction material. 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. MSE walls normally require a large, relatively open area behind the wall to bury reinforcing elements for 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, various types of metallic facing, 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 18.

MSE walls for highways currently require select granular materials for backfill. Backfill serves two functions, providing drainage for the soil mass behind the wall 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.

Use of lower quality backfill is the subject of a current FHWA study. Granular material offers excellent drainage characteristics, which provide increased life to the reinforcing elements, especially when the reinforcing elements are 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 progresses upward.

The construction sequence for MSE walls starts with site preparation. A leveling pad is 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. 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 first layer of reinforcing elements and compacted.

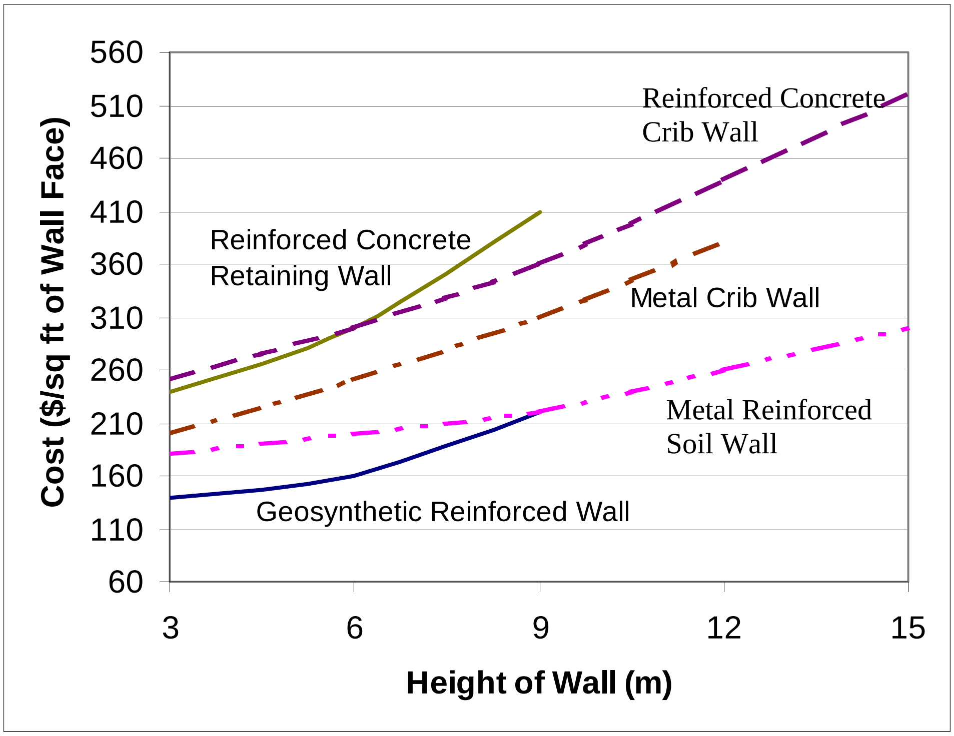


Figure 18 – Cost of Various Types of MSE Walls.

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 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 moisture content, so RSS have many applications for alleviating problems with saturated soils. Some specific applications include preventing sloughing of slopes during periods of saturation, various uses in flood control structures, and maintaining slope stability when increasing the height of earthen 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 (i.e. mowing) are often more hazardous on steeper slopes. The practice of designing and constructing RSS is still evolving and has yet to be standardized.

5.7 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 indicate 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 unstable 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 one or more type of appendage(s) 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.

5.7.1 Inclinometers.

Inclinometers (sometimes called slope indicators) 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 in a fixed location. A probe is lowered down the casing and readings that measure the horizontal deflection of the casing are taken at fixed intervals. Successive readings taken over a period of time provide a chronological record of horizontal deformation in the inclinometer casing as a function of depth.

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 may be taken daily during embankment construction. If an increase in rate of slope movement close to or above one order of magnitude is detected, construction should be halted immediately until the cause is determined and corrective action taken.

5.7.2 Settlement Plates.

The simplest form of settlement indicator is a steel or wooden plate placed in 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 of the top of the settlement plate (or of the top of the reference rod). 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 construction activities. After construction has been completed, readings can be taken at a reduced frequency unless problems are indicated. Settlement data is normally plotted as a function of time. Data is analyzed to determine when the rate of settlement has diminished to the extent that construction of pavement and other structures can begin.

NDOR uses a modified settlement-monitoring device for embankment construction that eliminates the need to determine the elevation at the top of the reference rod. The NDOR modified settlement device uses a fixed steel rod that is installed on a base plate on the original ground surface. This inner rod extends within a shaft that is shielded from the compressible layers constructed around it by a casing or by bentonite grout (at the discretion of the geotechnical engineer). Both the inner rod and shielding are extended upward as fill material is placed in layers. The relative downward movement of the outer casing compared to the inner fixed rod provides an accurate measurement of magnitude of settlement for the embankment material (see Figure 19 for details).

5.7.3 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 peizometers allows construction to be halted or slowed before soil failure due to buildup of excessive pore water pressure occurs.

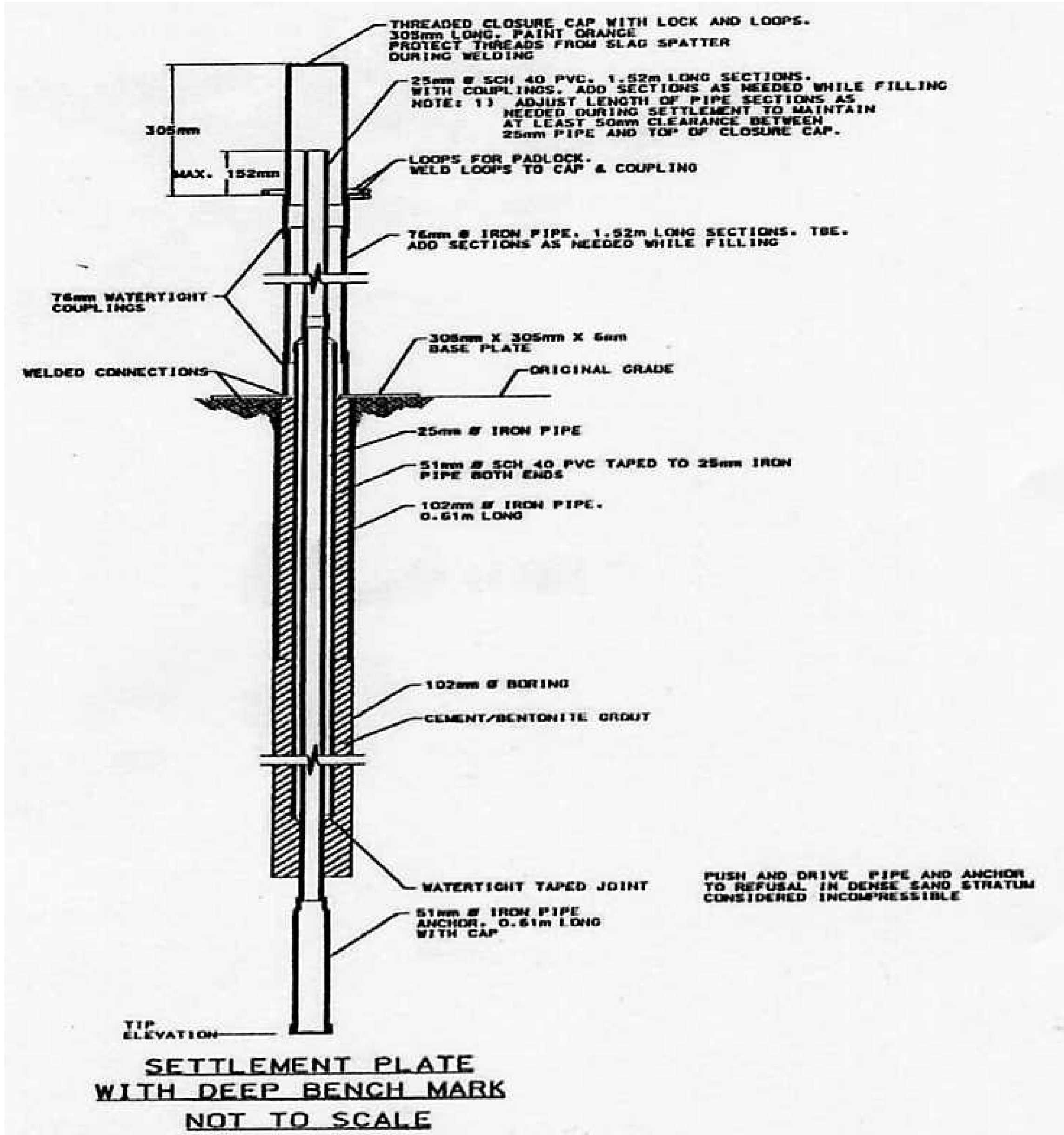


Figure 19 – NDOR Modified Settlement Monitoring Device.

During project construction, piezometers are used to evaluate increases in pore water pressure resulting from construction activities. Piezometers are normally checked at least daily 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 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 changes.

Piezometers are normally installed prior to or during construction at any location where excess pore water pressure may develop. 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.

Chapter 6

Geotechnical Reports & Forms

Chapter 6 consists of geotechnical reports and forms used to keep accurate and complete records of the progress of work performed and materials tested. Blank copies of these forms are available online or in paper format from NDOR offices.

Page

STATE OF NEBRASKA
DEPARTMENT OF ROADS

SAMPLE IDENTIFICATION FORM

Name of Material: Soil for Compaction Test

Project Number: F-2-1(000)

County: Lancaster

Name of Road: Lincoln East

Date Sampled: Feb. 1, 1990

Sample taken from: Excavation/ Borrow Pit No. 1

Material for use in (type of work or kind of structure): Embankment

Location to be used (Station or other information): Sta.153+00 to Sta. 200+00

Sampled by: J. Doe — Title: EA-II — Address: Box 2306 Lincoln Ne.

Report to be sent to: D. Jones — Title: Proj. Mgr. — Address: " " "

Contractor (performing work): Contractors Inc. — Address: Lincoln, Ne.

AGGREGATE-SOIL-FILLER – ROAD GRAVEL

Field Ident. No.	S-1	S-2	S-3	
Location (Hole No., etc.)	40' Rt.	Sta. 159+00		
Depth of Sample	0-1.5'	1.5'-3'	3'-9.0'	
Depth of Overburden				
Thickness of Stratum	topsoil	subsoil	peorian	
Type or Class				

Pit Location ______ ¼ Sec. ______, T- ______, R- ______

Kind of Pit (Dry or Wet)

Quantity, cu. yd.

Owner of Pit

Produced by

CONCRETE CYLINDERS (6x12)

Field Ident. No.

Specified Class

Mix Used

Water, lbs./lb.

Air Voids, per cent

Slump, inches

Quantity, cu. yd.

Method of Curing Structure or Pavement

Method of Curing Cylinders

Days Cured in Field

Min. Temp. during Cure

Cement: Brand & Type

Location of Mill

Admixture: Name

Quantity per 100 lbs. Cement

S.G. or Fine Aggregate: Source

Pit Location

Coarse Aggregate: Source

Pit Location

ASPHALT AND ASPHALTIC OIL

Field Ident. No.				
Type				
Car or Truck Ident.				
Ref. Insp. No.				
Quantity, gal.				
Manufactured by				
Location of Refinery				

CEMENT

Field Ident. No.		
*Type		
Car No., etc.		
Quantity, tons		

Brand Name

Manufactured by

Place of Storage

Length of Storage

*Type I - Regular; Type III - High Early; Type IA - Air-Entraining.

BITUMINOUS AGGREGATE – ASPHALTIC CONCRETE

Field Ident. No.		
Sampled from sta.		
Lift, Lane, Qtr. Point		
Stations Rep: From		
To		
Asph. Oil gal./sta.		
Aggregate, tons/sta.		
Type of Asph. Conc.		
Job Mix No. (Asph. Conc.)		
Asph. % (Asph. Conc.)		
Type Asph. or Asph. Oil		
Mfr. Asph. or Asph. Oil		
Car or Truck Ident.		

OTHER MATERIALS

Field Ident. No.

Kind of Material

Brand

Quantity Represented

Size and Weight specified

To comply with Spec. No.

Manufactured by

Jobber

OTHER INFORMATION

TO BE FILLED IN AT LABORATORY

Date received at Laboratory

Laboratory Identification

Submitted by: John Doe

Title: EA-II

Address: Box 2306 Lincoln Ne 68506

DR Form 12, Oct 74

2800

STATE OF NEBRASKA DEPARTMENT OF ROADS

FIELD GRADATION TESTS OF AGGREGATES

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

PROJECT NUMBER: F-111(1)

REPORT NUMBER: 2

KIND OF MATERIAL: SELECT SOIL	FOR USE IN: EMBANKMENT
NAME OF ROAD: LINCOLN - NORTHEAST	COUNTY: LANCASTER
PIT NUMBER: 514 · LOCATION: NW ¼ · SECTION: 6 · T. 12 N · R. 7 E	3.5 Miles N of Davey (TOWN, CITY, LANDMARK)
NAME OF PRODUCER: CONTRACTORS INC.	CONTRACTOR: CONTRACTORS INC.
DELIVERY POINT: STA. 65+00 to STA. 90+00	SAMPLED FROM: (Pit, Car, Truck, Window, Stockpile) WINDROW
SPECIFICATIONS · SECTION: · SUBSECTION:	SPECIAL PROVISIONS · SECTION: · SUBSECTION:

MECHANICAL ANALYSIS OF MATERIAL

Spacer sieves *(were) (were not)* used

DATE	SAMPLE NUMBER	WASH OR DRY TEST		DRY WEIGHT OF SAMPLE	*	*	*	3/4	3/8	4	10	20	*	50	100	200	QUANTITY OF MATERIAL REPRESENTED BY THIS TEST & CAR NUMBER	STATION
					TOTAL WEIGHT RETAINED ON SIEVE / TOTAL PER CENT RETAINED ON SIEVE													
			Weight	245						0	2	5	37	76	184	231		
4/13	FS-31	W	Percent							0	1	2	15	31	75	94	5 STATIONS	65+00
			Weight	208							0	4	29	62	156	196		
"	FS-32	W	Percent								0	2	14	30	75	94	5 "	70+00
			Weight	223						0	2	7	36	62	163	205		
"	FS-33	W	Percent							0	1	3	16	28	73	92	5 "	75+00
			Weight	198							0	4	30	65	150	188		
4/14	FS-34	W	Percent								0	2	15	33	76	95	5 "	80+00
			Weight	205							0	4	33	62	152	191		
"	FS-35	W	Percent								0	2	16	30	74	93	5 "	85+00
			Weight	217							0	9	37	74	165	204		
"	FS-36	W	Percent								0	4	17	34	76	94	5 "	90+00
			Weight															
			Percent															
			Weight															
			Percent															
			Weight															
			Percent															
			Weight															
			Percent															
SPECIFICATION RANGE									0						-50	80+		

*Insert appropriate Sieve Size

Duplicate of Sample Number FS-35 Submitted to Lincoln Laboratory for Correlation.

REMARKS: *(Indicate Disposal of material failing Specification Requirement)*

DISTRIBUTION
1 - Materials and Tests Division
2 - District Engineer
3 - Project Manager

INSPECTOR: John Doe

APPROVED: *(Project Manager)* Gary Kelley

DR Form 63, Feb 72

THIS FORM REPLACES DR FORM 63, JUL 70. PREVIOUS EDITIONS WILL NOT BE USED.

330

Distribution:
1. Materials & Tests Division
2. District Engineer
3. Project Manager

STATE OF NEBRASKA
DEPARTMENT OF ROADS

WEEKLY REPORT OF MOISTURE AND DENSITY TESTS

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

PROJECT NO.: I-80-2( ) & S-130( )

NAME OF ROAD: - EXAMPLE -

REPORT NO. 2

WEEK ENDING: 10-27-89

TYPE OF WORK *(embankment, subgrade, foundation course, soil-aggregate base, etc.)*: SUBGRADE COMPACTION

METHOD OF COMPACTION — CLASS: III — TYPE:

		COLUMN NUMBER	1	2	3	4	5	6	7	8
		Date	10-22	10-22	10-22	10-22	10-23	10-23	10-22	10-33
		Sample No.	6	7	8	9	10	11	8-A	11-A
		Curve No.	E-4	E-4	E-2	E-3	E-3	E-3	E-2	E-3
		Opt. Moist. From Curve	10.3	10.3	9.0	14.9	14.9	14.9	9.0	14.9
(1)		Max. Dens. *(Dry)* from Curve	1.97	1.97	2.02	1.63	1.63	1.63	2.02	1.63
(2)		Required Moist. Limits	0-11.3	0-11.3	0-10.0	0-15.9	0-15.9	0-15.9	0-10.0	0-15.9
(3)		Required % of Max. Dry Dens.	100	100	100	100	100	100	100	100
(4)	SAMPLE TAKEN AT	Station	574+00	571+00	564+00	554+00	550+50	560+00	564+00	560+00
(5)		Ft. Rt. or Lt. of ¢	¢	15'RT.	20'RT.	15'RT.	12'LT.	15'LT.	20'RT.	15'LT.
(6)		Depth Below Finish Grade	GRADE	GRADE	GRADE	GRADE	GRADE	GRADE	GRADE	GRADE
(7)	ACTUAL VOLUME	Final Reading *(cc or cu ft)*	2400	3050	3325	3325	3600	3325	3450	3400
(8)		Initial Reading *(cc or cu ft)*	1500	1500	1450	1425	1450	1450	1450	1475
(9)		Difference *(cc or cu ft)* (7 - 8)	900	1550	1875	1900	2150	1875	2000	1925
(10)		Volume of Sample *(cc)*	255	439	531	538	609	531	566	545
(11)	ACTUAL MOISTURE	Wet Wt. *(gms)*	200	200	200	200	200	200	200	200
(12)		Dry Wt. *(gms)*	180	181	188	181	182	182	188	182
(13)		Loss *(gms)* (11 - 12)	20	19	12	19	18	18	12	18
(14)		Moisture (%) [(13)÷(12)] 100	11.1	10.5	6.4	10.5	9.9	9.9	6.4	9.9
(15)	ACTUAL DENSITY	Wet Wt. *(gms)* (entire sample)	579	968	1114	1023	1160	914	1214	1029
(16)		Wet Dens. *(gms/cc)* [(15) ÷ (10)]	2.27	2.20	2.10	1.91	1.91	1.72	2.15	1.89
(17)		Dry Dens. *(gms/cc)* $\left[\frac{(16)}{100+(14)}\right]100$	2.04	1.99	1.98	1.73	1.74	1.56	2.02	1.72
(18)		Maxium Density *(% of)* [(17)÷(1)] 100	103	101	98	106	107	96	100	105
		Thickness *(GFC, SABC, etc.)*								

REMARKS: TEST No. 8 & 11 NEEDS MORE ROLLING TEST No.-8A & 11A CHECK TESTS FOR 8 & 11

INSPECTOR: John B. Smith

APPROVED: (Manager) Gary A. Saylor

DR Form 86, Feb 72

THIS FORM REPLACES DR FORM 86, SEP 70. PREVIOUS EDITIONS WILL NOT BE USED.

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STATE OF NEBRASKA
DEPARTMENT OF ROADS

MOISTURE DENSITY TEST

AASHO T-99, METHOD

DENSITY DETERMINATION								
DETERMINATION NUMBER	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. GM	2036							
WT. WET SOIL, GM								
VOLUME OF MOLD, CC.	940							
WET DENSITY, GM/CC								
DRY DENSITY, GM/CC	1.610	1.668	1.726	1.701	1.642			
MOISTURE DETERMINATION								
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	369.6			
WT. OF WATER, GM								
WT. OF CONTAINER, GM	82.1	82.1	82.9	81.4	81.7			
WT. OF DRY SOIL, GM								
PERCENT MOISTURE	12.7	14.9	17.2	19.2	21.3			

PROJECT: 6-6(126) and L80H(105)

LAB. IDENT.: S95-381

SPECIFIC GRAVITY:

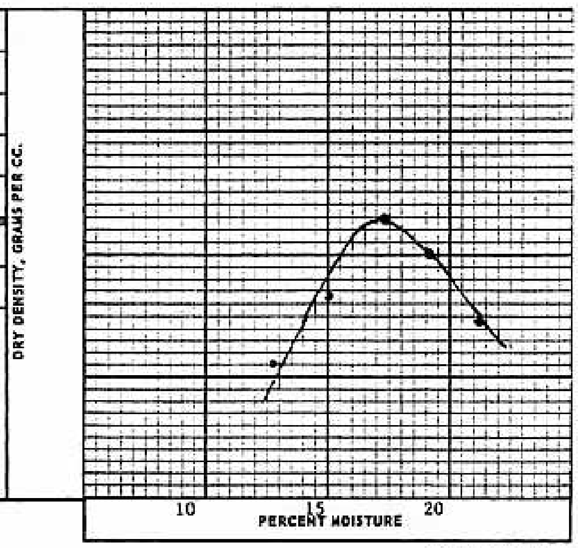
MAX. DENSITY, GM/CC: 1.73

OPT. MOISTURE, PERCENT: 17.2

OPERATOR: Allen

DATE: 7-27-95

REMARKS: MDC#G-119



DR Form 210, Sep 72

printed on recycled paper

Check One

☐

☒

STATE OF NEBRASKA
DEPARTMENT OF ROADS

FIELD GRADATION TESTS OF GRAVEL FOR SURFACING AND MINERAL AGGREGATE FOR ARMOR COAT

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

PROJECT NUMBER: Sheet 1 of 4

REPORT NUMBER: P. O. #545329

NAME OF ROAD: Hwy. 59 M. P. 24.8 ½ mile West of Jct. 121/59

COUNTY: KNOX

PIT NUMBER: | LOCATION: NE ¼ | SECTION: 18 | T 26 N | R 1W | 6 Miles NE of Pierce (Town, City, Landmark)

NAME OF PRODUCER: Backus Sand & Gravel

CONTRACTOR: SAME

DELIVERY POINT: Stockpile

SAMPLED FROM: SAME

SPECIFICATIONS — SECTION: | SUBSECTION: | SPECIAL PROVISIONS — SECTION: | SUBSECTION:

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

DATE	SAMPLE NUMBER	WASH OR DRY TEST		DRY WEIGHT OF SAMPLE	1"	3/4"	3/8"	No. 4	No. 10	No. 50	No. 200	QUANTITY OF MATERIAL REPRESENTED BY THIS TEST & CAR NUMBER	STATION
					TOTAL WEIGHT RETAINED ON SIEVE / TOTAL PERCENT RETAINED ON SIEVE								
06/20	1	DRY	Weight	50			1		37	49	50	50 cu. yd.	Stockpile
			Percent				2		74	98	100		
06/20	2		Weight	75			1		57	74	75		
			Percent				1		76	99	100		
06/20	3		Weight	37			1		28	37	37		
			Percent				1		76	100	100		
06/20	4		Weight	46			1		33	46	46		
			Percent				1		72	100	100		
06/20	5		Weight	61			1		44	60	61		
			Percent				2		72	98	100		
Computation for Deduction =				Sum of % Retained on No. 10 Sieve = divided by Number of Tests Performed =					370 / 74	= Ave %	$	___ % Deduction (From Tables in Specifications) x Contract Unit Price $ ___ = Per C.Y. for Lot No. ___ .	
06/21	6	DRY	Weight	57			1		43	57	57	50 cu. yd.	Stockpile
			Percent				2		75	100	100		
06/21	7		Weight	62			0		46	61	62		
			Percent				0		74	98	100		
06/24	8		Weight	35			0		24	34	35		
			Percent				0		69	97	100		
06/24	9		Weight	42			1		30	41	42		
			Percent				1		71	98	100		
06/24	10		Weight	54			1		41	54	54		
			Percent				2		76	100	100		
Computation for Deduction =				Sum of % Retained on No. 10 Sieve = divided by Number of Tests Performed =					365 / 73	= Ave %	$	___ % Deduction (From Tables in Specifications) x Contract Unit Price $ ___ = Per C.Y. for Lot No. ___ .	
	SPECIFICATION RANGE						0-6		59/81	90/100	96/100		

DISTRIBUTION

1 - Materials and Tests Division
2 - CHECK ONE
☐ Construction Division (Submit with Final Computations)
☒ Maintenance Division (Submit with Purchase Order)
3 - District Engineer
4 - Project Manager

Duplicate of Sample Number MA-96-2 Submitted to Lincoln Laboratory for Correlation.

REMARKS: (Indicate disposal of material failing Specification Requirement)

INSPECTOR: Amy Rezac

APPROVED: (Project Manager)

DR Form 264, Jan 77

THIS FORM REPLACES DR FORM 264, OCT 74.
PREVIOUS EDITIONS WILL BE USED UNTIL EXHAUSTED.

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

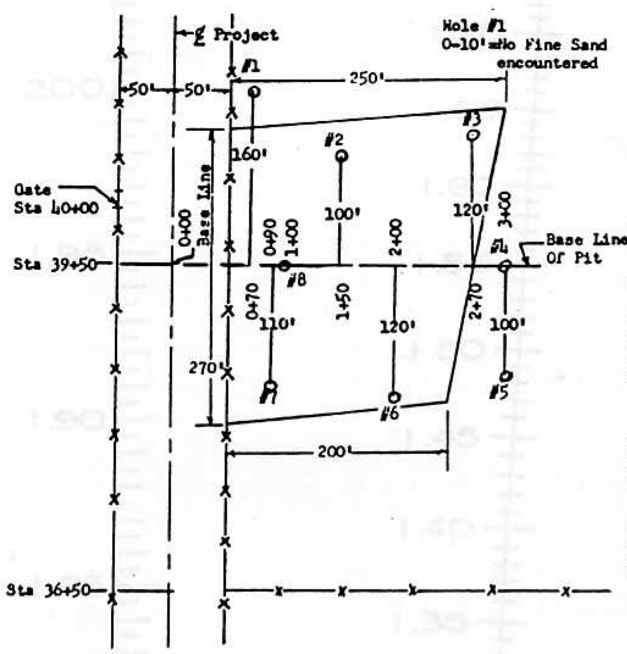
Type of road between pit and project Adj. to Project

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".)



Hole #1
0-10'=No Fine Sand encountered

Hole #2
0-1'=O.B.
1'-10'=Fine Sand Samp. FS-1
10'+=Silty Soil

Hole #3
0-1'=O.B.
1'-10'=Fine Sand Samp. FS-2
10'+=Silty Soil

Hole #4
0-10'=No Fine Sand encountered

Hole #5
0-10'=No Fine Sand encountered

Hole #6
0-1'=O.B.
1'-5'=Fine Sand Samp. FS-3
5'-10'=Fine Sand Samp. FS-4
(slightly coarser)
10'+=Silty Soil

Hole #7
0'-1'=O.B.
1'-10'=Sim.to Samp. FS-3
10'+=Silty Soil

Hole #8
0-1'=O.B.
1'-10'=Fine Sand Samp. FS-5
10'+=Silty Soil

N

SOIL COMPACTION CURVES

Project No. **STPD-4-6(108)**
Project Name **Plymouth West**

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. S-753(3)

Year constructed 1965

Curve No.	Horizon No. Name	Location Depth *(feet)*	% Retained			P.I.	G.I.	Max. Density	Opt. Moisture
			#10	#50	#200				
A-103	Topsoil (lean clay)	0-1.2		0	1	11	8	1.60	19.6
A-104	Subsoil (lean clay)	1.2-4.0		0	1	39	25	1.56	23.2
A-101	Peorian Loess (lean clay)	3.0-10.0	0	1	1	22	14	1.65	20.7
A-102	Glacial Till (lean clay)	6.0-10.0		0	1	21	13	1.69	17.3
A-100	Glacial Till (sandy silt)	11.0-25.0		0	45	NP	4	1.77	13.6

Nebraska Department of Raods
Materials and Research Division
Geotechnical Section-Soil Survey Unit
Tabulation of Soil Survey Borings

Project Number: S-275-6(1025) **Control Number:** 32026
Project Name: West Point North & South **Date:** 8/21/2002

Location or Hole No.	Depth of Material (feet)	Sample Ident	Similar To	USCS Symbol	Soil Description	Depth to Water	% Moist.	% Ret. # 200	P.I.	G.I.
64+00 C.L.										
	0-0.2				Topsoil					
	0.2-16.0	W-100		SM	Peorian Loess (tan sandy silt dry & powdery to 8.0')			69	NP	0
	15.0	WM-1					4.2			
	16.0-20.0	W-1		CL	Peorian Loess (brown sandy silt)			39	9	5
67+00 C.L.										
	0-0.2				Topsoil					
	0.2-20.0		W-100	SM	Peorian Loess (tan sandy silt, dry, powdery)			69	NP	0
	7.0	WM-7					5.1			
	15.0	WM-8					8.4			
70+00 C.L.										
	0-0.2				Topsoil					
	0.2-6.0		W-100	SM	Peorian Loess (tan sandy silt)			69	NP	0
	6.0-12.0	W-101		SM	Sand (tan-orange fine sand)			80	NP	0
	12.0-20.0	W-102		CL	Glacial (gray silty clay contains gravel peices)			36	25	12
	15.0	WM-2					12.6			
72+00 C.L.										
	0-0.2				Topsoil					
	0.2-10.0	W-4		ML	Peorian Loess (tan sandy silt, dry, powdery)			30	NP	7
78+00 30' Rt.										
	0-0.2				Topsoil					
	0.2-15.0		W-2	SM	Peorian loess (tan sandy silt)			50	3	3
	8.0	WM-9					11.1			
85+00 C.L.										
	0-0.2				Topsoil					
	0.2-30.0	W-103		CL	Peorian Loess (tan silty clay, friable)			11	11	8
	20.0	WM-4					6.9			

Note:

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

Project Number: S-275-6(1025)
Project Name: West Point North & South

Control Number: 32026
Date: 8/21/2002

Lab Ident	Sample Ident	Location or Hole No.	Depth of Material	Sieve Analysis Percent Retained											% Silt	% Clay	Liquid Limit	Plasticity Index	Group Index	Insitu Moisture Content	Maximum Density	Optimum Moisture	Moisture Density Curve No.	Organic Content %
				1 1/2	1	3/4	3/8	4	10	20	40	50	100	200										
S02-276	W-100	64+00 C.L.	0.2-16.0						0		1	3	36	69	24	7	NP	NP	0		1.82	12.0	N-166	
S02-284	WM-1	64+00 C.L.	15.0																	4.2				
S02-277	W-1	64+00 C.L.	16.0-20.0						0		2	4	24	39	44	17	24	9	5					
S02-291	WM-7	67+00 C.L.	7.0																	5.1				
S02-292	WM-8	67+00 C.L.	15.0																	8.4				
S02-278	W-101	70+00 C.L.	6.0-12.0						0		2	10	56	80	12	8	NP	NP	0		1.88	11.7	N-167	
S02-279	W-102	70+00 C.L.	12.0-20.0					0	2		6	11	28	36	31	33	37	25	12		1.76	14.1	N-168	
S02-285	WM-2	70+00 C.L.	15.0																	12.6				
S02-294	W-4	72+00 C.L.	0.2-10.0								0	1	11	30	59	11	NP	NP	7					
S02-293	WM-9	78+00 30' Rt.	8.0																	11.1				
S02-280	W-2	80+00 30' Rt.	0.2-12.0								0	1	22	50	36	14	20	3	3					
S02-281	W-3	80+00 30' Rt.	12.0-22.0								0	1	5	14	65	21	29	9	8					
S02-286	WM-3	80+00 30' Rt.	20.0																	20.4				
S02-282	W-103	85+00 C.L.	0.2-30.0					0	1		3	4	7	11	67	22	32	11	8		1.79	15.1	N-169	
S02-287	WM-4	85+00 C.L.	20.0																	6.9				
S02-288	WM-5	90+00 C.L.	25.0																	16.8				
S02-283	W-104	103+00 C.L.	0.2-7.0						0		1	3	46	85	8	7	NP	NP	-2		1.79	12.7	N-170	
S02-289	WM-6	103+00 C.L.	15.0																	6.7				
S02-431	W-5	105+00 C.L.	0.2-7.0						0		1	1	9	16	63	21	30	9	8					
S02-437	WM-10	105+00 C.L.	10.0																	6.6				
S02-432	W-105	110+00 C.L.	3.0-10.0								0	1	3	8	64	28	38	19	12		1.73	18.5	N-219	
S02-438	WM-11	110+00 C.L.	10.0																	22.7				
S02-433	W-6	117+00 C.L.	0-15.0						0		2	7	46	88	6	6	NP	NP	-2					
S02-439	WM-12	117+00 C.L.	10.0																	4.1				
S02-434	W-106	140+00 C.L.	2.0-5.0								0	1	3	7	60	33	42	21	13		1.68	20.2	N-220	
S02-440	WM-13	140+00 C.L.	5.0																	14.2				
S02-435	W-107	146+00 C.L.	0-2.0						0		1	2	10	19	55	26	32	11	8		1.67	19.2	N-221	
S02-441	WM-14	151+00 C.L.	10.0																	23.1				
S02-442	WM-15	160+25 C.L.	20.0																	18.7				
S02-436	W-108	165+00 C.L.	2.0-20.0									0	1	3	68	29	36	16	10		1.71	17.9	N-222	
S02-443	WM-16	165+00 C.L.	15.0																	16.7				

Field Report of Nuclear Density Tests for Soils

NDOR
Nebraska Department of Roads

Project:	F-20-5(1009)	Control No.	31073A
Location:	Royal-Brunswick	Report No.	7

Type of Work: ☑ Grading ☐ Sewer ☐ Paving ☐ Backfill ☐ Base ☐ Misc.

Gauge Number	Method of Compaction		Remarks:	For Week Ending 6-23-01
26880	Class	Type		
	III			

Date	6/18/2001	6/18/2001	6/19/2001	6/19/2001	6/20/2001
Test Number	26	27	28	29	30
Curve Number	B-103	B-100	B-100	B-100	B-100
Optimum Moisture %	16.5	10.8	10.8	10.8	10.8
Maximum Density (lb/ft^3)	1730.0	1880.0	1880.0	1880.0	1880.0
Moisture Limits	0.0	As Needed	As Needed	As Needed	As Needed
Req. % of Max. Density	95.0	100.0	100.0	100.0	100.0
Station	817+16.99	802+53.53	815+33.08	817+47.47	804+97.37
Offset (ft) Lt/Rt Centerline	10.8m ☑ Lt ☐ Rt	2.4m ☐ Lt ☑ Rt	6.7m ☑ Lt ☐ Rt	7.0m ☑ Lt ☐ Rt	2.4m ☐ Lt ☑ Rt
Depth Below Grade	1.2m	Grade	.3m	.5m	Grade
Density Standard Count	2832.0	2832.0	2840.0	2840.0	2840.0
Density 1	96.7	100.9	99.5	99.8	99.9
Density 2	94.5	100.4	100.2	99.7	99.8
Average % Density	95.6	100.7	99.9	99.8	99.9
Moisture Standard Cnt.	656.0	656.0	655.0	655.0	654.0
Moisture 1	12.3	6.8	6.5	7.9	5.9
Moisture 2	13.1	7.4	6.4	7.7	6.1
Average % Moisture	12.7	7.1	6.5	7.8	6.0
Test Status	☑ Pass ☐ Fail	☑ Pass ☐ Fail	☑ Pass ☐ Fail	☑ Pass ☐ Fail	☑ Pass ☐ Fail

Notes:

Inspector:		Project Manager:	

Distribution: Project Manager
QA Manager
District Engineer
Materials & Research Division

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Appendix A

AASHTO AND ASTM PRACTICES/TEST METHODS

AASHTO Specifications/Test Methods

Subject	Number
Sizes of Aggregate for Road and Bridge Construction	M 43
Materials for Embankments and Subgrades	M 57
Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	M 145
Materials for Aggregate and Soil-Aggregate Subbase, Base and Surface Courses	M 147
Geotextiles Specifications for Highway Applications	M 288
Drilling for Subsurface Investigations – Unexpectedly Encountering Suspected Hazardous Material	R 21
Standard Guide for Decommissioning Geotechnical Exploratory Boreholes	R 22
Materials Finer than 75 µm (No. 200) Sieve in Mineral Aggregates by Washing	T 11
Sieve Analysis of Fine and Coarse Aggregate	T 27
Specific Gravity and Absorption of Coarse Aggregate	T 85
Test Method for Particle-Size Analysis of Soils	T 88
Determining the Liquid Limit of Soils	T 89
Determining the Plastic Limit and Plasticity Index of Soils	T 90
Moisture Density Relations of Soils Using a 2.5 kg (5.5 lb) Rammer and a 305 mm (12-inch) Drop	T 99
Specific Gravity of Soils	T 100
Moisture Density Relations of Soils Using a 4.54 kg (10 lb) Rammer and a 454 mm (18-inch) Drop	T 180

Penetration Test and Split-Barrel Sampling of Soils	T 206
Thin-Walled Tube Sampling of Soils	T 207
Unconfined Compressive Strength of Cohesive Soil	T 208
Permeability of Granular Soils (Constant Head)	T 215
One-Dimensional Consolidation Properties of Soils	T 216
Field Vane Shear Test in Cohesive Soil	T 223
Direct Shear Test of Soils Under Consolidated Drained Conditions	T 236
Measurements of Pore Pressures in Soils	T 252
Installing, Monitoring and Processing Data from the Traveling Type Slope Inclinometer	T 254
Determining Expansive Soils	T 258
Laboratory Determination of Moisture Content of Soils	T 265
Determination of Organic Content in Soils by Loss on Ignition	T 267
Resilient Modulus of Subgrade Soils and Untreated Base/ Subbase Materials	T 292
Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	T 296
Consolidated, Undrained Triaxial Compression Test on Cohesive Soils	T 297
Determining the Resilient Modulus of Soils and Aggregate Materials	T 307
In-Place Density and Moisture Content of Soil and Soil-Aggregate By Nuclear Methods (Shallow Depth)	T 310

ASTM Practices/Guides/Test Methods

Subject	Number
Test Method for Specific Gravity and Absorption of Coarse Aggregate	C 127
Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as Mineral Admixture in Concrete	C 618
Guide to Site Characterization for Engineering Design and Construction Purposes	D 420
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^3 or 600 kN-m/m^3)	D 698
Test Method for Specific Gravity of Soils	D 854
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^3 or 2,700 kN-m/m^3)	D 1557
Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586
Practices for Thin-Walled Tube Sampling of Soils For Geotechnical Purposes	D 1587
Practice for Diamond Core Drilling for Site Investigation	D 2113
Test Method for Unconfirmed Compressive Strength of Cohesive Soil	D 2166
Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass	D 2216
Practice for Wet Preparation of Soil Samples for Particle Size Analysis	D 2217
Test Method for Permeability of Granular Soils (Constant Head)	D 2434

Test Method for One-Dimensional Consolidation Properties of Soils	D 2435
Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487
Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488
Test Method for Field Vane Shear Test in Cohesive Soil	D 2573
Test Method for Unconsolidated-Undrained Triaxial Compression Tests on Cohesive Soils	D 2850
Test Method for Density of Soil and Soil-Aggregate In Place by Nuclear Methods (Shallow Depth)	D 2922
Test Methods for Moisture, Ash, and Organic Matter of Peat And Other Organic Soils	D 2974
Test Method for Water Content of Soil and Rock In Place by Nuclear Methods (Shallow Depth)	D 3017
Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D 3080
Practice for Classification of Soils and Soil-Aggregate Mixtures For Highway Construction Projects	D 3282
Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D 3385
Test Method for Mechanical Cone Penetration Tests of Soil	D 3441
Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D 4186
Practices for Preserving and Transporting Soil Samples	D 4220
Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D 4254
Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D 4318
Test Methods for One-Dimensional Swell or Settlement	D 4546

Potential of Cohesive Soils

Test Method for Determining the Water (Moisture) Content of Soil by the Microwave Oven Method	D 4643
Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648
Test Method for Pressuremeter Testing in Soils	D 4719
Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750
Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767
Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084
Guide for Field Logging of Subsurface Explorations of Soil And Rock	D 5434
Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778
Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151
Test Method for Determining Dispersive Characteristics Of Clayey Soils by the Crumb Test	D 6572

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 text (body of the report), a tabulation of soil properties and soil borings, a tabulation of test results and a diagram 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. Topography, drainage characteristics and location of the water table are outlined next. Information concerning soil formations and geology of the project area follows. The soil horizons and soil formations encountered in the project area are then detailed. 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 for selective handling 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 granular 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 will always specify the planned project surface.

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. 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. It 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 four feet (1.2 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 four feet (1.2 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) Compaction 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 SOILS

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.

S**ubsoil** 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**, 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. 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 20) shows some possible variations in soil development due to topography. Development of a soil profile depends on the action of percolating water to leach material from the topsoil and redeposit the leached material into 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. These deposits are commonly encountered in stream channels and are referred to as alluvium or colluvium.

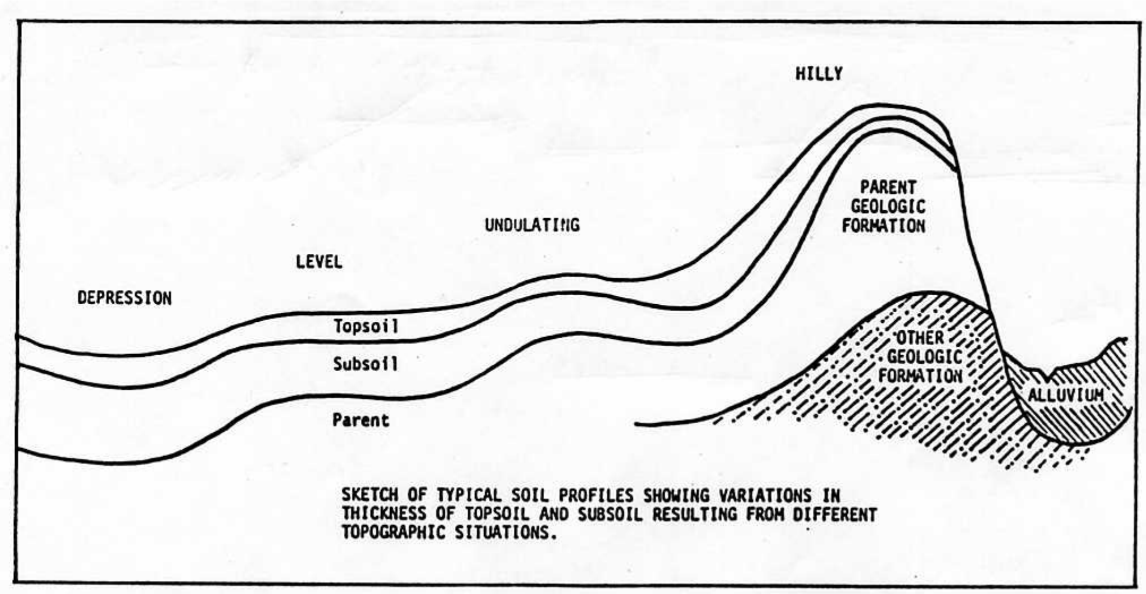


Figure 20 – Variations in Soil Development Due to Topography.

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 coarse versus fine material in the soil*) is easily determined by touch. Grains of sand and gravel can be distinguished as individual particules 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, clay 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 processes may have significantly altered many of the distinguishing characteristics of these materials and formations. Maps showing the major topographic regions of Nebraska and the parent materials being weathered to form Nebraska soils are included as Figures 21 and 22.

Soil Formations of Recent Age

ALLUVIUM: Water deposited material in stream floodplains. Zones of development may be missing. Local variations in texture commonly denoted on soil maps include 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 soil parent 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.

TODD VALLEY: Reworked Peorian loess, commonly silt with some fine sand. Found in elevated benches along river valleys.

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 occur in Peorian loess deposits.

REDEPOSITED PEORIAN: Loess that has eroded out of its original deposition environment; 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 loess. This is often observed 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 (aka SAPPA 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. NDOR makes no distinction between the Kansas and Nebraskan tills.

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 Sandhills 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 obtained 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; mostly clean sand and fine channel deposits.

REDEPOSITED CONCRETIONS: Transported concretions and coarse material making up coarse gravel deposits. This is the source of Class "D" limerock.

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

REDEPOSITED BRULE: Slumped and weathered Brule Formation. It is commonly 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 sandstone 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.

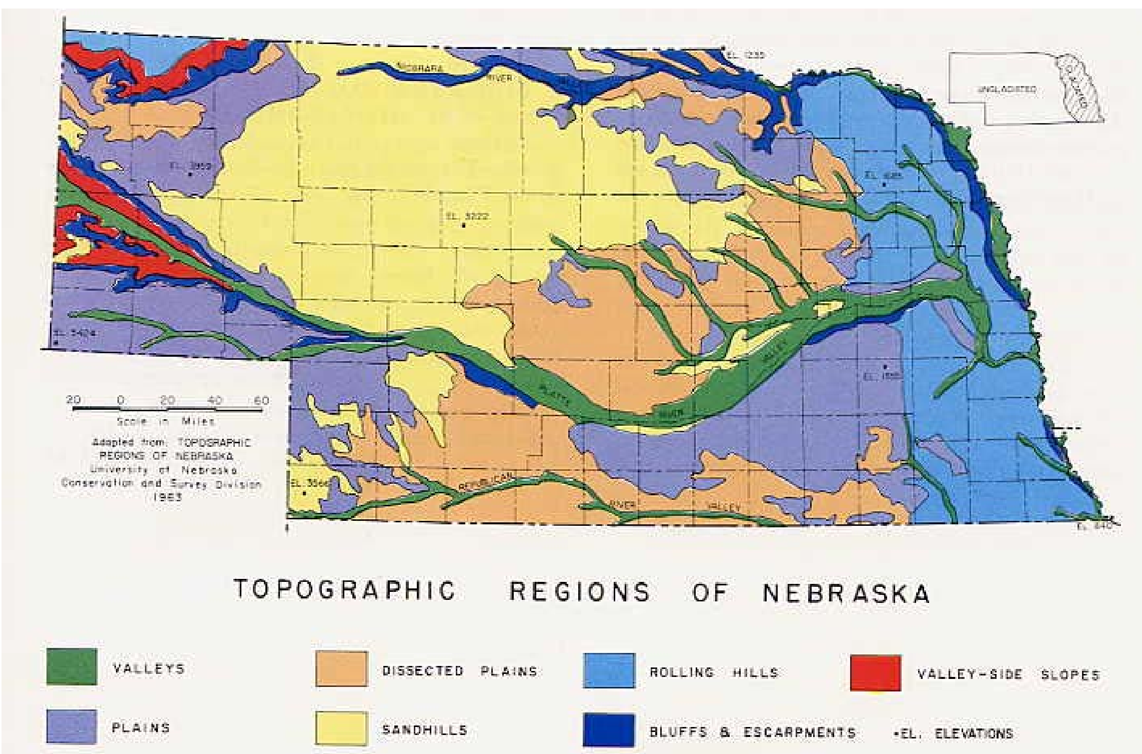
GREENHORN LIMESTONE: Thin, medium soft gray limestones interbedded with gray shales. The presence of many oyster shell-like fossils marks the upper portions and makes it easy to identify.

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

DAKOTA SANDSTONE AND DAKOTA SHALES: A source of fine sand, the formation 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 fine-grained silty clay shales, usually interbedded with sand. Clay in the Dakota Shales generally has high swell characteristics, thus making Dakota Shale generally unsuitable for use as subgrade material. Dakota shale usually has a glossy or soapy appearance and may be multicolored.

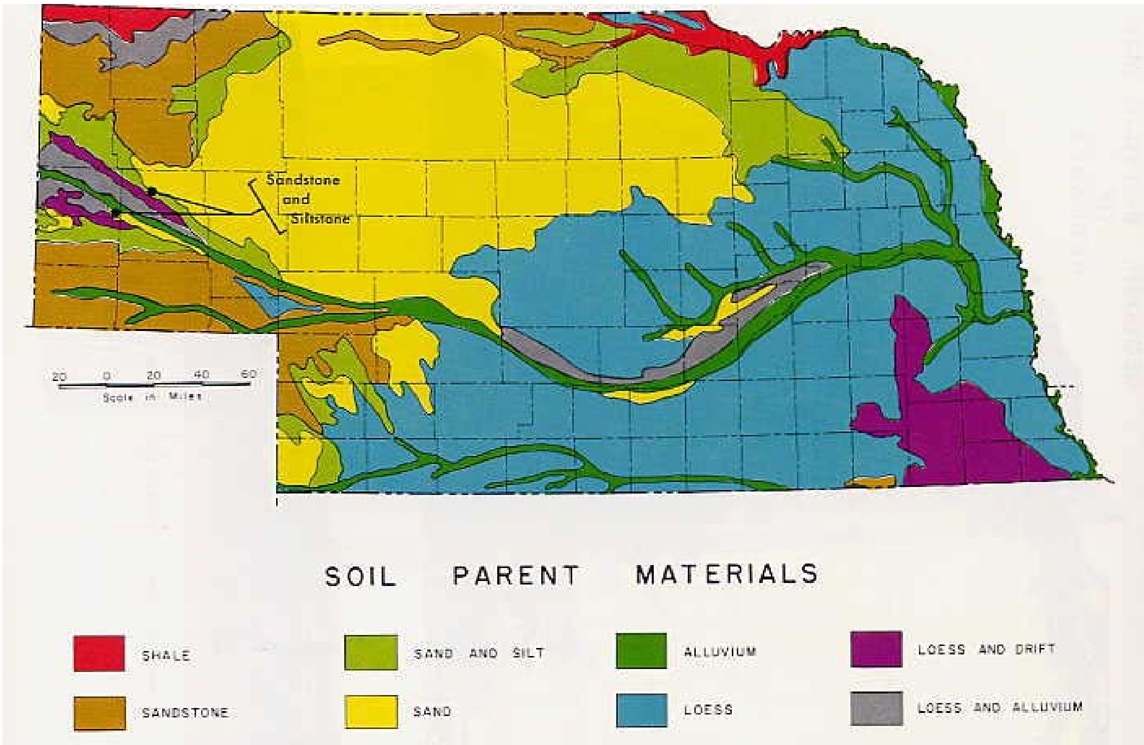
Formations of Permian-Pennsylvanian Age

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.



(Source: Elder, 1969)

Figure 21 – Major Topographic Regions of Nebraska.



(Source: Elder, 1969)

Figure 22 – Parent Materials of Nebraska Soils.

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 23. 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 24. To obtain data from which to plot these curves, several identical samples were compacted 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%

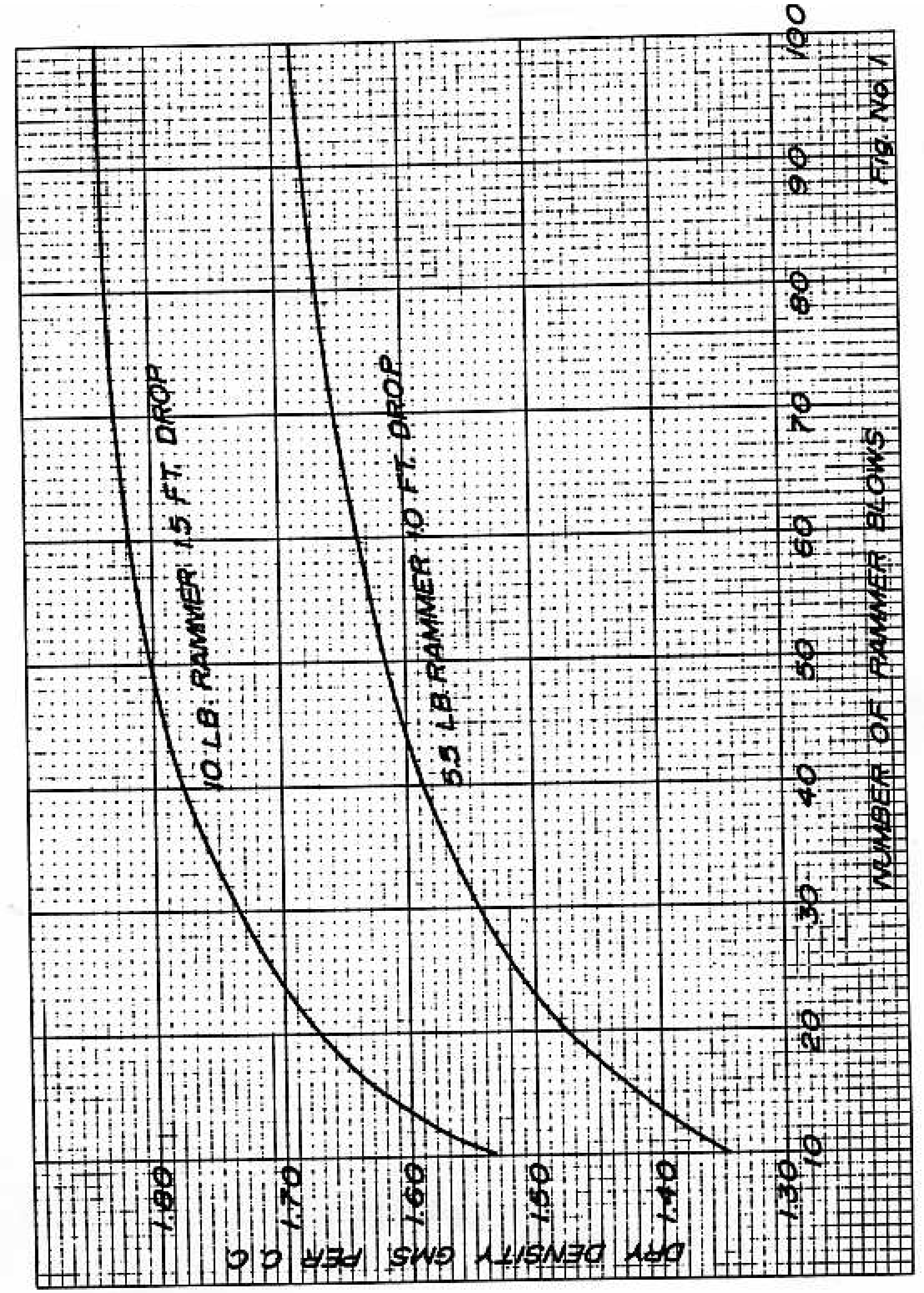


Figure 23 – Relationship between Compaction Effort and Soil Density
at Constant Moisture Content.

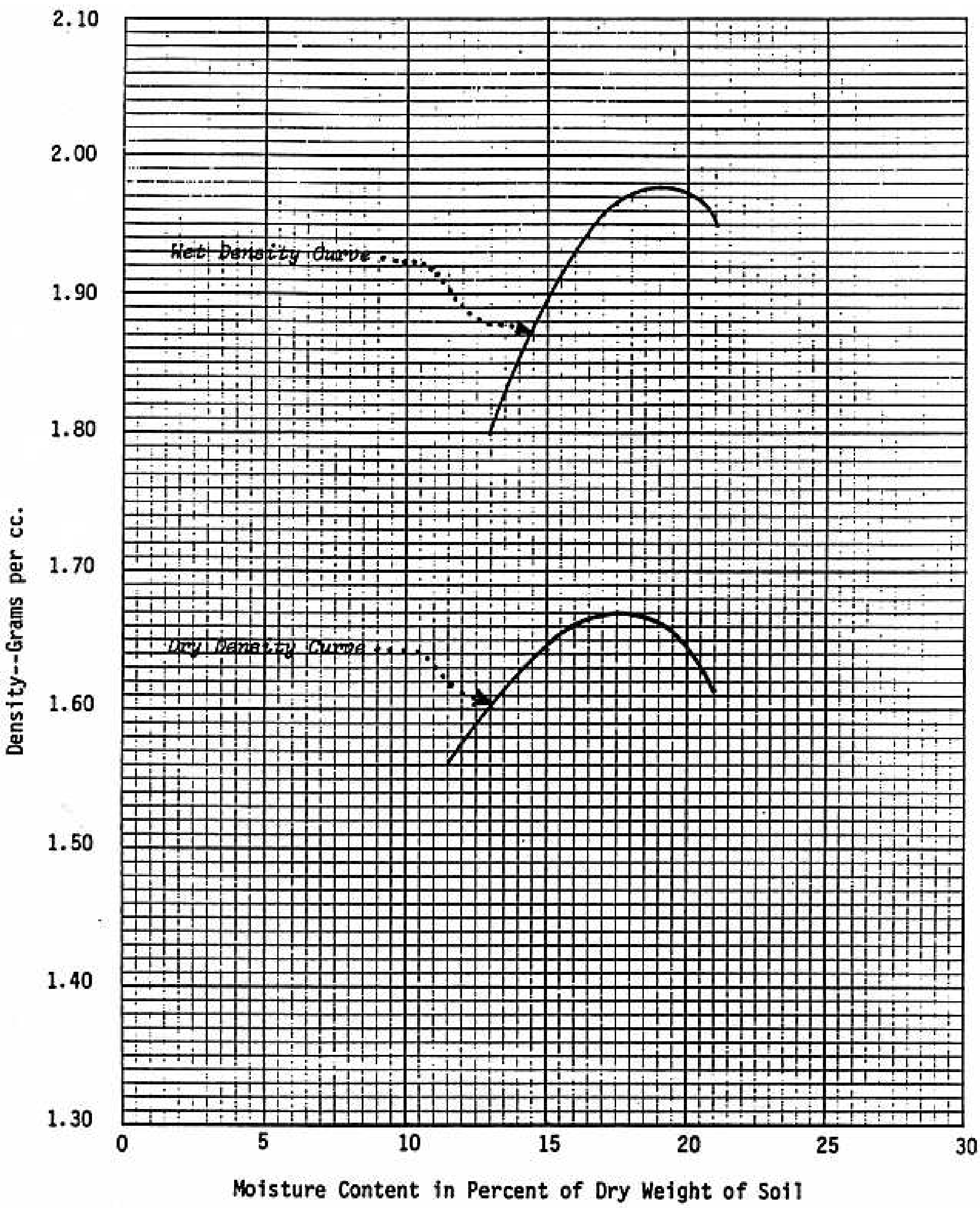


Figure 24 – Relationship Between Soil Moisture Content and Density For Constant Compaction Effort.

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 24, 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 (OMC) 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 25.

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

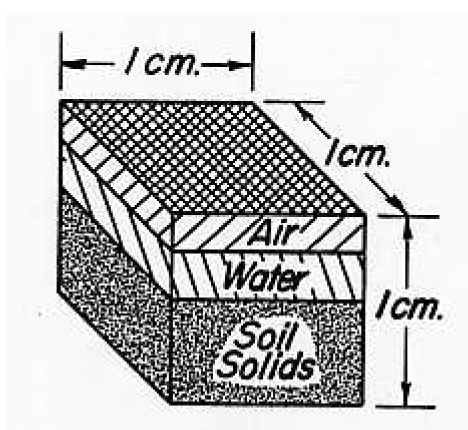


Figure 25 – One Cubic Centimeter of Soil Divided into
Solid, Liquid and Air Components.

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^{o} C
> % moisture with all voids filled = 0.394/1.6 x 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 26 is for soil solids with a specific gravity of 2.64. Since the position of the ZAV curve with respect to the moisture density curve is significant, the moisture density curve for dry density is also plotted in Figure 26.

A range of ZAV curves can be plotted for the same soil over the range of specific gravities associated with common soil minerals. ZAV curves commonly shift slightly right and left corresponding to changes in mineral content. Left and right shifts are very slight for the range of specific gravities normally encountered when working with Nebraska soils. Since the ZAV 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 compaction is capable of removing all of the air voids from a soil under field conditions. 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 24 is plotted again on Figure 27. A curve showing resistance to penetration, a ZAV curve and a curve for 3% air voids are also superimposed on Figure 27. 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 27 shows that resistance to penetration becomes weaker as moisture content increases, suggesting that soil at higher moisture contents will carry less load.

Examination of Figure 27 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 27 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.

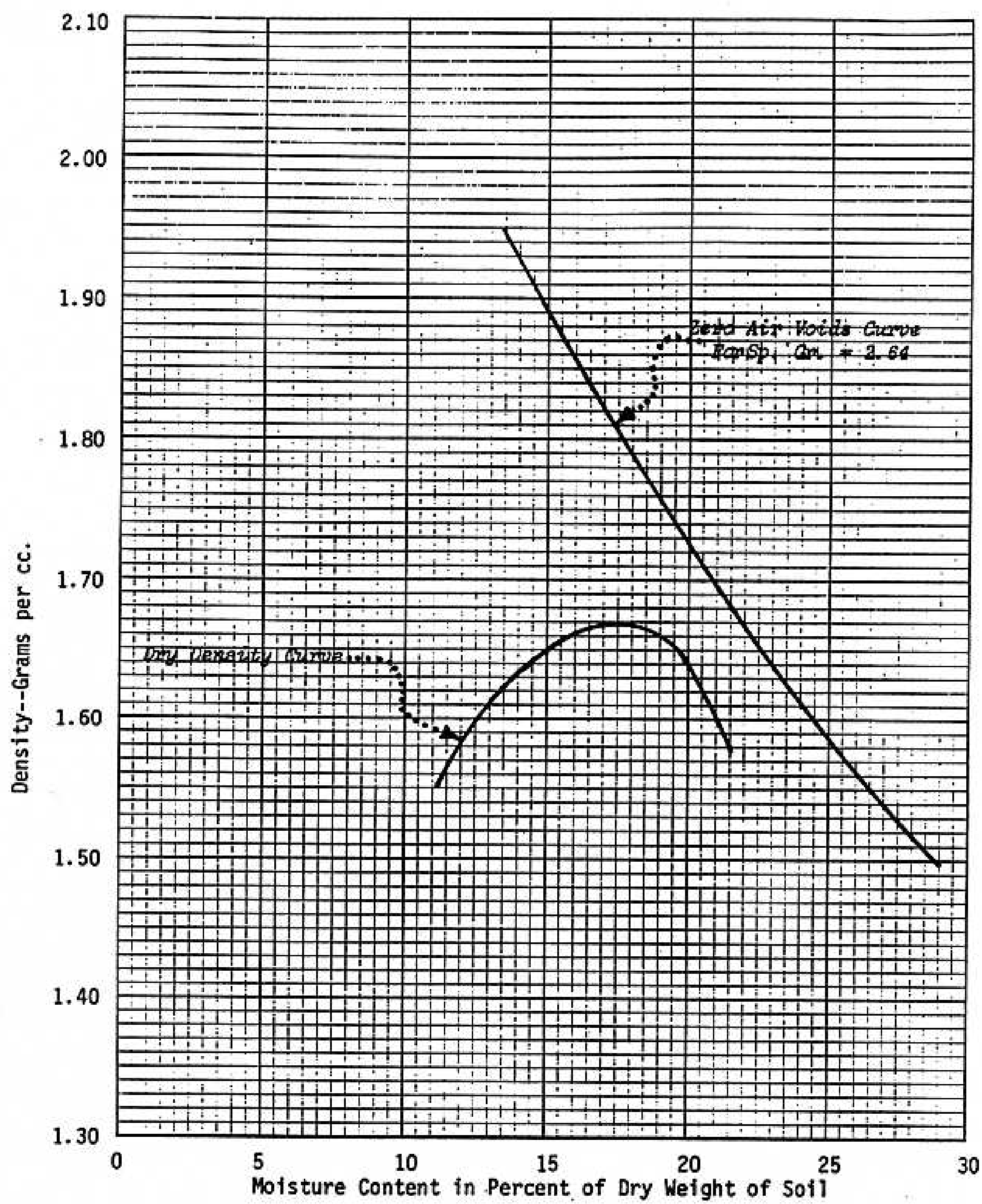


Figure 26 - Relationship Between Standard Dry Density Curve and Zero Air Voids Curve.

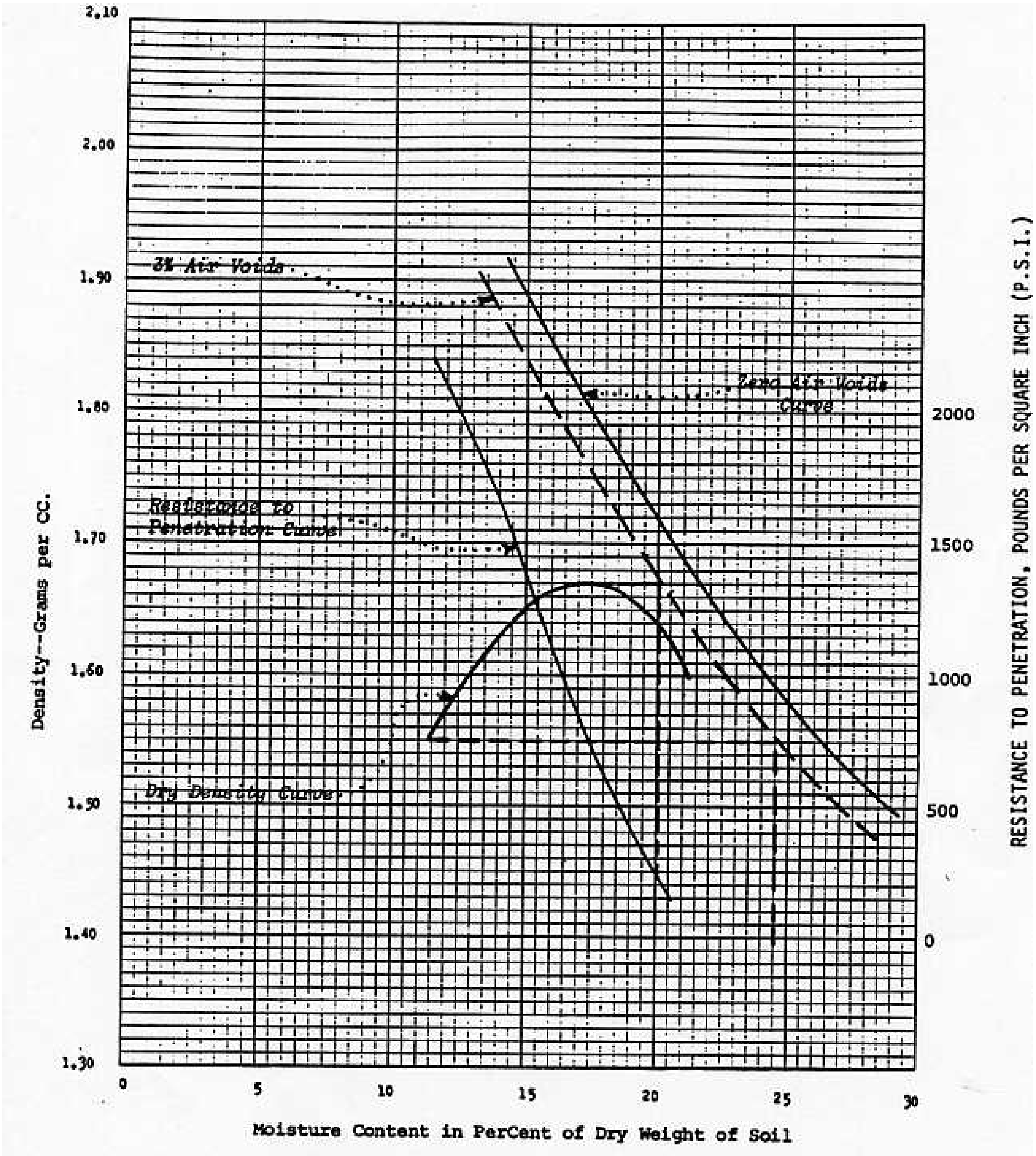


Figure 27 – Resistance to Penetration at Different
Moisture Contents and Dry Densities.

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 28 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 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 then 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 the 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

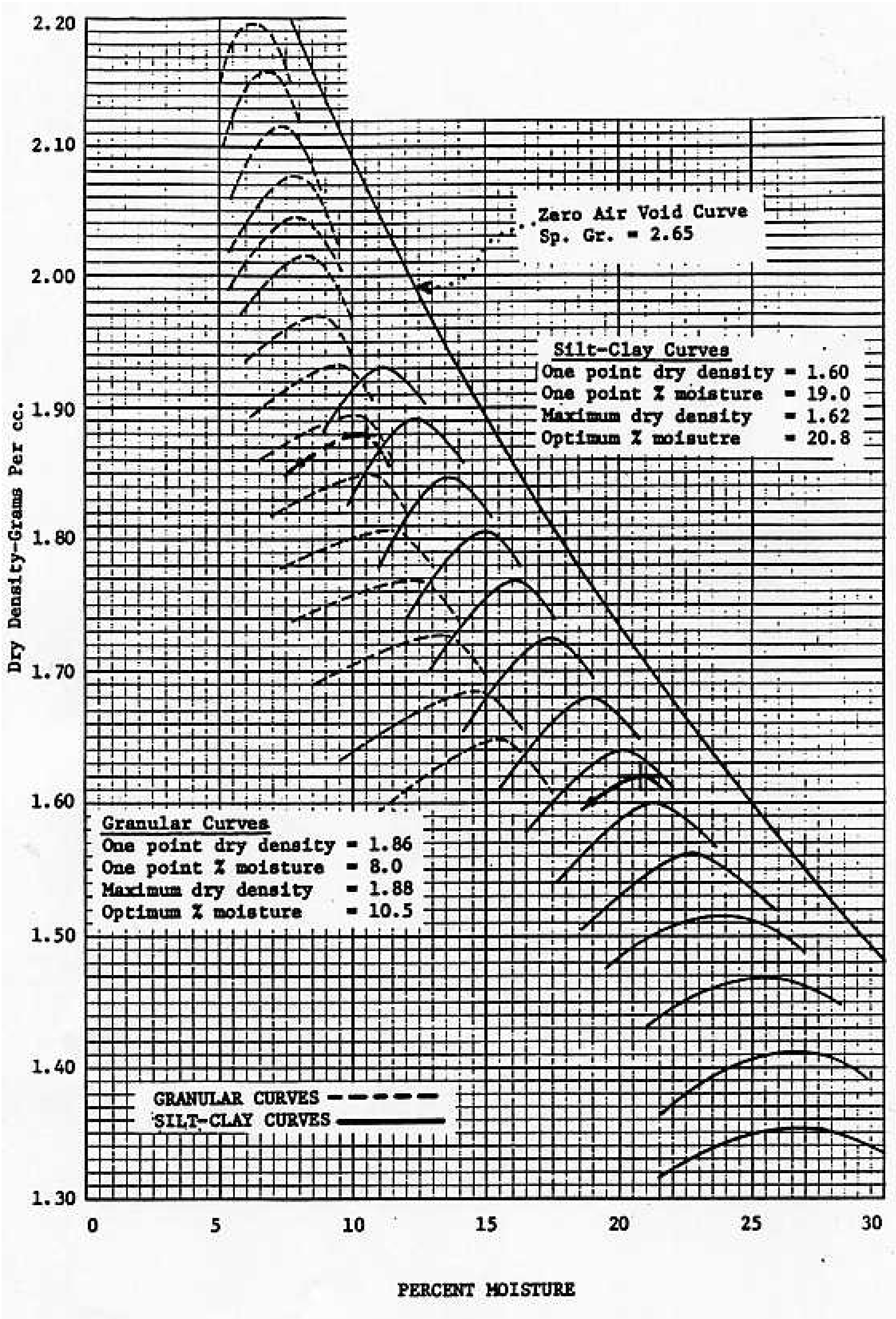


Figure 28 – Moisture Density Curves for Typical Nebraska Soils.

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 planned, the density and moisture requirements will be shown on the plans. As an example of a Class III embankment, 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	MAXIMUM
Embankment/Roadway Grading to be surfaced, including driveways	Silt-Clay	Upper 3'	96±3	Opt.-3%	Opt.+3%
	Silt-Clay	At depths greater than 3'	95 Min.	**	Opt.+2%
	Granular	All depths	100 Min.	**	**
Embankment/Roadway Grading not to be surfaced	All	All depths	Class II	(See Specifications)	
Subgrade Preparation	Silt-Clay	The upper 6" of subgrade soil	96±3	Opt.-3%	Opt.+3%
	Granular	The upper 6" of subgrade soil	100 Min.	**	**
Embankment of drive-ways not surfaced	All	All depths	Class I	(See Specifications)	
Bituminous Pavement patching	All	Underlying Material	100 Min.	**	**

** Moisture as necessary to obtain density.

The contractor will be 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 always meet or exceed project specifications.

Field moisture density sampling is now frequently done using a nuclear moisture density gauge. ASTM D 2922 details standards for determination of soil density using nuclear equipment while ASTM 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

GUIDELINES FOR USING GEOSYNETHETICS 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 ~ 0.5 tons/ft^2 (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 29.

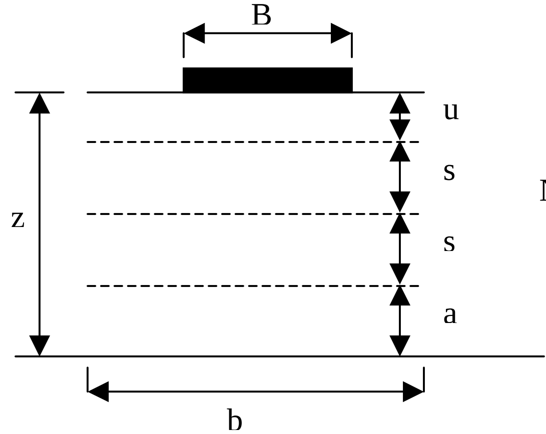


Figure 29 – Geogrid Spacing and Length Parameters.

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 14.

The design values shown in Table 14 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 30 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 31.

Table 14 – Recommended Parameters for Geogrid Used in Foundation Reinforcement.

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

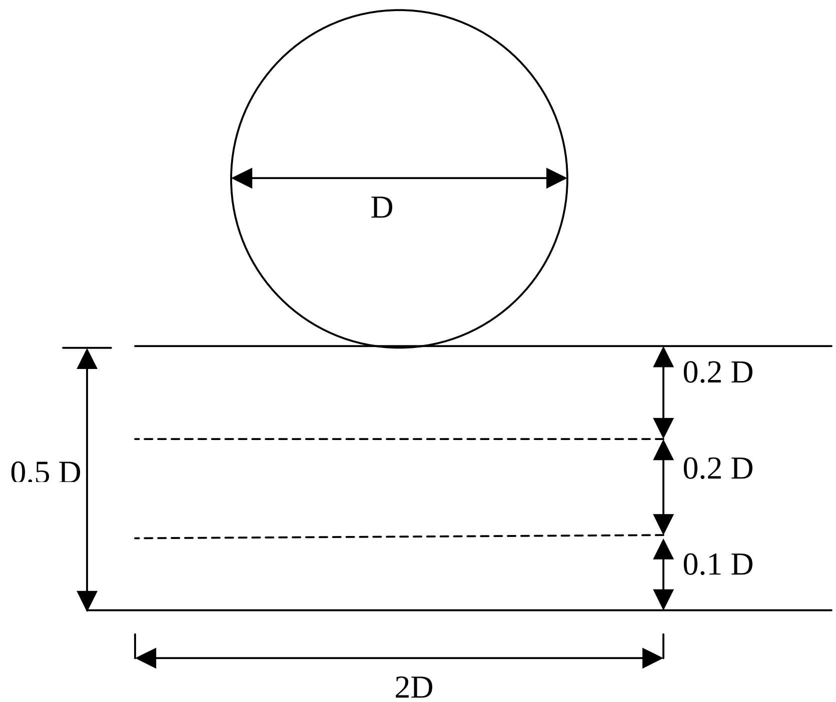


Figure 30 – Minimum Thickness of Replacement Material with Geogrid Reinforcement.

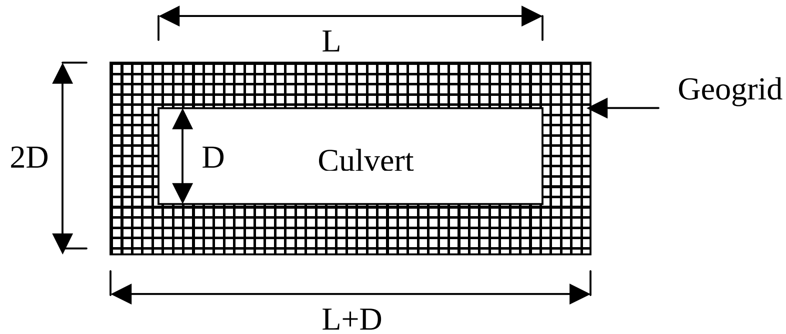


Figure 31 – Plan View Showing Lateral Extent of Culvert and Geogrid.

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^2. 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^2. 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.