Design Manual
Chapter 200
Geotechnical Design
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Engineering Properties of Soil and Rock

Quick Tips:

- The geotechnical designer is responsible for determination of geotechnical parameters.
- Selection of soil/rock engineering properties requires that the designer understand the loading conditions and soil/rock behavior.

This section provides guidance on selecting soil and rock engineering properties for use in geotechnical design. Properties selected for design should be based on the results of field testing, laboratory testing, or back analysis of existing conditions. In addition, local experience, correlations to local geologic formations, and relevant published data should also be considered in the final selection of design parameters.

The geotechnical designer should assess the variability of the engineering properties to determine if the observed variability is a result of laboratory testing variability or subsurface variations across the site. The geotechnical designer is responsible for selecting the geotechnical engineering properties that are appropriate for the analyses required for design of the project.

Geotechnical design parameters should be established for each of the geologic strata identified at a project site. A geologic stratum, consisting of either soil or rock material, is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, etc.

Soil Strata

Soil is a nonhomogeneous, porous material for which engineering behavior is greatly influenced by density, source material, stress history, and hydrogeology. Some engineering properties (e.g., undrained shear strength in normally consolidated clays) may vary as a predictable function of a stratum dimension (e.g., depth below the top of the stratum). Where the engineering property varies within a soil stratum, the engineer should develop the design parameters taking this variation into account. This may result in multiple values within the stratum.

Rock Strata

Determination of design parameters for rock should take into consideration the in-situ rock properties (condition of the rock mass), which are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. A combination of laboratory testing on small samples, empirical analysis, field observations, and engineering judgment should be employed to determine the engineering properties of rock masses, with appropriate emphasis placed on visual observations and quantitative descriptions of the rock mass.

Influence of Existing and Future Conditions on Soil and Rock Properties

Soil properties are not intrinsic to the soil type, but vary with the influence of stress, groundwater, and other environmental conditions. Thus, it is important to determine the existing conditions, as well as how conditions may change over the life of the project. For example, construction of a new embankment places new surcharge loads on the underlying soil profile. As the stress increases due to loading, the engineering properties of the underlying soil strata may also change. For example, normally consolidated clays can gain strength with increases in effective stress. On the other hand, over-consolidated clays or weak rock may lose strength with time when exposed in cuts. The geotechnical engineer is responsible for choosing design parameters representative of existing and future conditions.

Determination of Soil and Rock Properties

Subsurface soil or rock properties are generally determined using empirical correlations related to the testing that is performed during the field exploration program (e.g. Standard Penetration Test (SPT)) outlined in Section 200C-1 and/or the laboratory testing as outlined in Section 200D-1. The laboratory testing program generally consists of index tests (classification) to obtain general information or to use with correlations to estimate design properties, and performance tests (strength and compressibility) to directly measure specific engineering properties.

The observational method or back analysis can be used to determine engineering properties of soil or rock in cases where slope failures occur, or where embankment settlement or excessive settlement of existing structures have been observed. For instance, with landslides or slope failures, the geotechnical engineer determines the geometry of the failure and then selects the soil/rock parameters, aided by correlations with index tests (if available) or experience, that result in a factor of safety approaching 1.0. For embankment or structure settlement, the engineering properties of the soils can sometimes be determined from laboratory tests outside and inside the zone of stress influence resulting from the loading, and then correlating these properties for the magnitude of the known loads. As with slope stability analysis, the pre-existing and final geometry of the subsurface soil must be adequately known, including the history of the groundwater levels.

For the detailed measurement and interpretation of soil and rock properties, follow the guidelines provided in Section 200D-1 and FHWA-IF-02-034, Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002), except as specifically indicated herein.

Engineering Properties of Soil

The selection of soil properties for design and analysis by the geotechnical engineer requires that the designer has a good understanding of the loading conditions and the soil behavior, has high quality soil sampling and testing, and has local geotechnical experience with the various geologic formations. This section provides guidance in the selection of engineering properties for cohesive soils (clays and highly plastic silts) and cohesionless soils (sands and non-plastic silts) for use in geotechnical design.

Unit Weight

The total (wet) unit weight of soils, see Table 1, can be estimated from typical values, or measurements of mass and volume can be performed on Shelby tube samples or California or Modified California rings. Moisture content can provide the necessary data for calculating the dry unit weight of the materials.

Table 1: Typical unit weight values for soils in their natural state (after Peck, Hanson, and Thornburn, 1974).

soil description	dry unit weight (pcf)	wet unit weight (pcf)
uniform sand (loose)	90	118
uniform sand (dense)	109	130
well-graded sand (loose)	99	124
well-graded sand (dense)	116	135
windblown silt (loose)	85	116
glacial till	132	145
soft glacial clay	76	110
stiff glacial clay	106	129
soft slightly organic clay	58	98
soft very organic clay	43	89

Shear Strength

The shear strength of a soil can be evaluated by one of the following methods:

- Geotechnical laboratory testing.
- Correlations with in-situ field testing results.
- Correlations based on index parameters.

The laboratory test that is selected and used to evaluate the shear strength of a soil should be the method that is best suited to model the loading condition (undrained total stress or drained effective stress) and the soil response. Shear strength laboratory testing methods are described in Section 200D-1. The selection of peak, fully softened, or residual shear strength for design analyses should be based on review of the expected or tolerable displacements of the soil.

Undrained Cohesive: The short term, end of construction design of the project should be based on undrained (total) shear strength parameters. The undrained shear strength (Su) of cohesive soils (clay and highly plastic silts) can be measured using unconfined compression (UC) tests, unconsolidated undrained (UU) triaxial tests, or consolidated undrained (CU) triaxial tests of undisturbed samples. Typically, the total internal friction angle (ϕ) is negligible and assumed to be zero ($\phi = 0$) in cohesive materials. However, if required for the analyses, the undrained (total) friction angle (ϕ) and cohesion components of the shear strength can be determined using appropriate laboratory testing methods.

The undrained shear strength (Su) of cohesive soils may also be correlated to results of in-situ testing such as Standard Penetration Test (SPT), Cone Penetrometer Test (CPT), Flat Plate Penetrometer Test (CPT), Flat Plate Dilatometer Test (DMT), or Vane Shear Test (VST). For example, undrained shear strength (Su) can be correlated to the N_{60} value obtained from the Standard Penetration Test (SPT). The undrained shear strength (Su) can be estimated for low plasticity clays (PI $\leq$ 10) and medium to high plasticity clays (11 $\leq$ PI $\leq$ 40) using the relationship developed by McGregor and Duncan, 1986, see Figure 1.

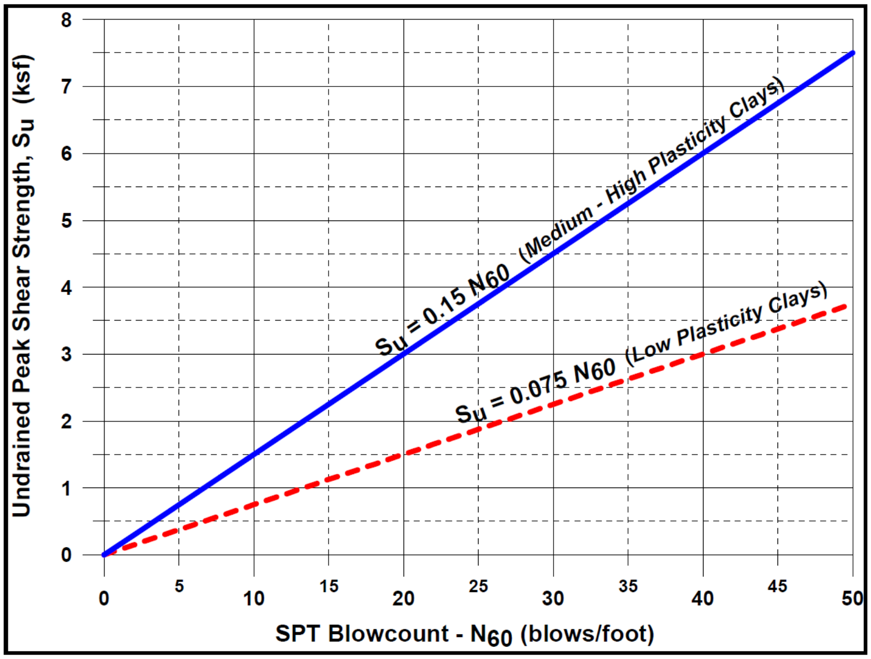

Figure 1: Undrained Shear Strength – SPT Relationship (McGregor and Duncan, 1986).

<u>Drained Cohesive:</u> The long-term design of the project's geotechnical components should be based on drained (effective) strength parameters. The use of peak or residual effective shear strength should be based on the expected or tolerable soil displacements. For normally-consolidated cohesive soils, the cohesive intercept (apparent cohesion) should be zero. For over-consolidated cohesive materials, such as many glacial tills, the cohesive intercept can be non-zero and may be established by consolidated undrained triaxial (CU) testing. The relationship between the peak/residual friction angle and the plasticity index as reported in NAVFAC DM7.1 generally works well for estimating the shear strength, see Figure 2.

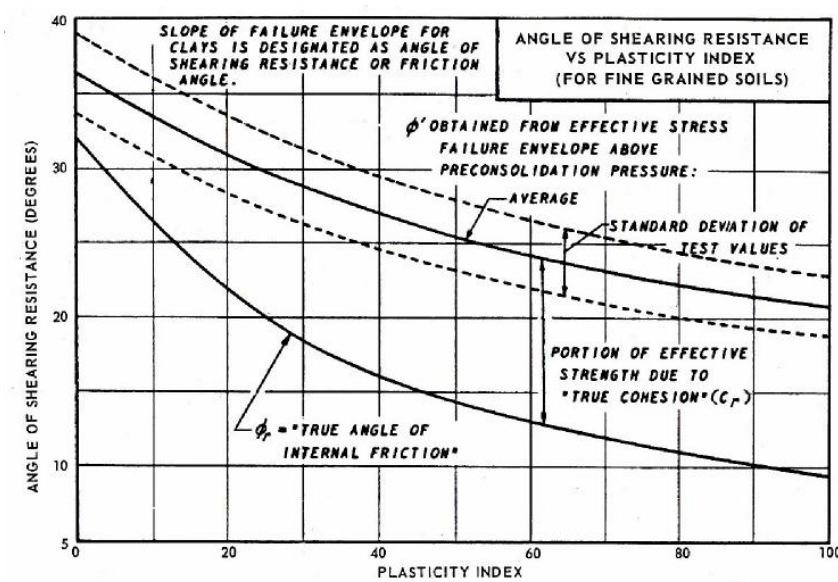

Figure 2: Correlation between residual shear strength of over-consolidated clays and plasticity index (after NAVFAC, 1971).

Drained Cohesionless: The drained friction angle (ϕ') of granular deposits may be evaluated by correlation to the results of a Standard Penetration Test. The drained friction angle (ϕ') of granular deposits may also be determined based on the relationships provided in Table 2. Table 2 is based on data for relatively clean sands, thus, sands with a significant amount of clay/silt-sized material will fall in the lower portion of the range. The reported values of effective friction angle (ϕ'), based on SPT N60 values, should be reduced by 5° for clayey sands and should be increased by 5° for gravelly sands. Engineering judgment should be used in selecting a specific value.

Table 2: Relationship among relative density, SPT N value, and internal friction angle of cohesionless soils (after Meyerhof, 1956).

state of packing	relative density (%)	standard penetration resistance, N (blows/ft)	friction angle, ϕ' (°)
very loose	<20	<4	<30
loose	20-40	4-10	30-35
compact	40-60	10-30	35-40
dense	60-80	30-50	40-45
very dense	>80	>50	>45

Note: N = 15 + (N’ – 15)/2 for N’>15 in saturated very fine or silty sand, where N’ = measured blow count and N = blow count corrected for dynamic pore pressure effects during the SPT.

Compressibility

Compressibility characteristics of the soils can be determined using laboratory testing or correlations to field/in-situ testing and index properties. Compressibility of soils is typically divided into two primary categories: 1) elastic settlement and 2) time-dependent settlement. Elastic settlements typically dominate in the cohesionless soils, while time-dependent settlements dominate in cohesive soils.

Cohesionless Soils: Obtaining high quality samples for laboratory testing can be extremely difficult in cohesionless soils. Therefore, indirect methods (correlations) of measuring the elastic parameters are used to develop compressibility parameters (elastic modulus, Poisson’s ratio, etc.), see Sabatini et al. (2002). Where evaluation of elastic settlement is critical to the design of the foundation or selection of the foundation type, in-situ methods such as Cone Penetrometer (CPT), Pressuremeter (PMT), or Dilatometer (DMT) should be considered for evaluating the elastic modulus of the soil strata. Typical elastic modulus values of predominately granular materials are shown in Table 3.

Table 3: Typical elastic modulus values for granular soil (modified after Sabatini, 2002).

soil type	range of equivalent elastic modulus ksf / (kPa)
fine sand • loose • medium dense • dense	 160 to 240 / (8,000 to 12,000) 240 to 400 / (12,000 to 20,000) 400 to 600 / (20,000 to 30,000)
sand • loose • medium dense • dense	 200 to 600 / (10,000 to 30,000) 600 to 1,000 / (30,000 to 50,000) 1,000 to 1,600 / (50,000 to 80,000)
gravel • loose • medium dense • dense	 600 to 1,600 / (30,000 to 80,000) 1,600 to 2,000 / (80,000 to 100,000) 2,000 to 4,000 / (100,000 to 200,000)

Note: 1ksf ≈ 50kpa.

Cohesive Soils: To determine the amount of consolidation settlement (primary and secondary) in cohesive soils, the following soil parameters are required: compression index (C_c), recompression index (C_r), secondary compression ($C\alpha$) index, consolidation coefficient (C_v), and the preconsolidation pressure (σ'_p). Compressibility parameters of cohesive soils can be determined from the results of one-dimensional consolidation tests. For preliminary analyses or where accurate prediction of settlement is not critical, values can be obtained from published correlations. If correlations for prediction of settlement are used, their applicability to the specific geologic strata under consideration should be evaluated. Refer to Sabatini et al. (2002) for discussion of the various available correlations of which some are outlined below.

Compression Index (C_c): Primary consolidation occurs as the pore pressures generated during loading dissipate. The amount of primary consolidation will be related to the compressibility characteristics of the soil, and can be determined from a one-dimensional consolidation test. In lieu of one-dimensional consolidation test data, the compression index can be estimated using a correlation established by Terzaghi and Peck (1967), which is valid for inorganic clays with sensitivity up to 4 and a LL less than 100.

$$C_c = 0.009 \times (LL - 10) \quad \text{(Equation 200E-1\_1)}$$

where:

LL = Liquid Limit (%).

Recompression Index (C_r): The Recompression Index (C_r) can be determined from the one-dimensional consolidation test data or estimated using empirical correlations that relate it to the C_c values, in that the C_r value is approximately 10 to 20 percent of the C_c value (Ladd, 1973).

Secondary Compression Index (C_α): Secondary compression occurs after the completion of elastic and primary consolidation settlements, and should be included in the estimate of total settlement. The Secondary Compression Index (C_α) is best determined from consolidation testing; however, Figure 3 can be used for preliminary estimates of secondary compression settlement.

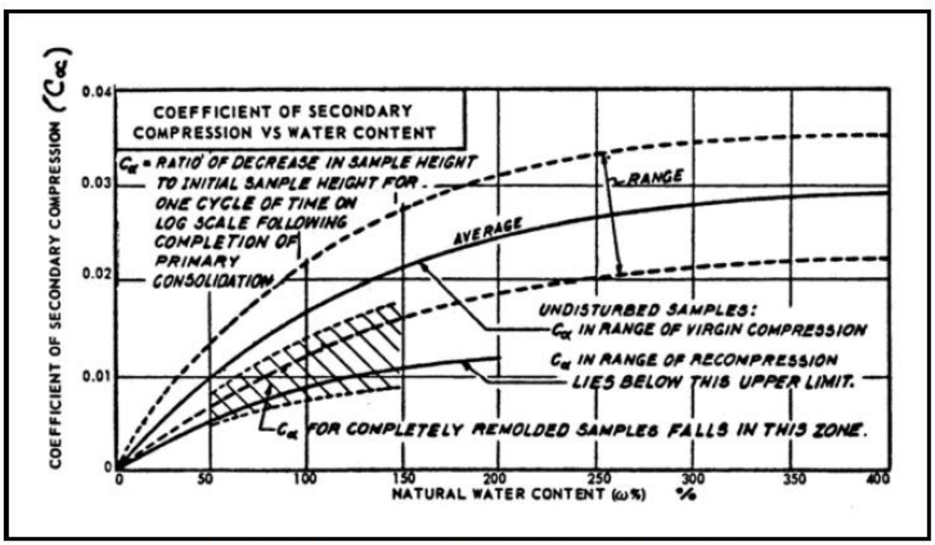

Figure 3: Secondary Compression Index Chart (NAVFAC DM-7.1).

Consolidation Coefficient (C_v): The rate of consolidation is directly related to the permeability of the soil and can be determined from a one-dimensional consolidation test. If laboratory test data is not available, an estimate of the Consolidation Coefficient (C_v) can be determined using Atterberg Limit test results and Figure 4.

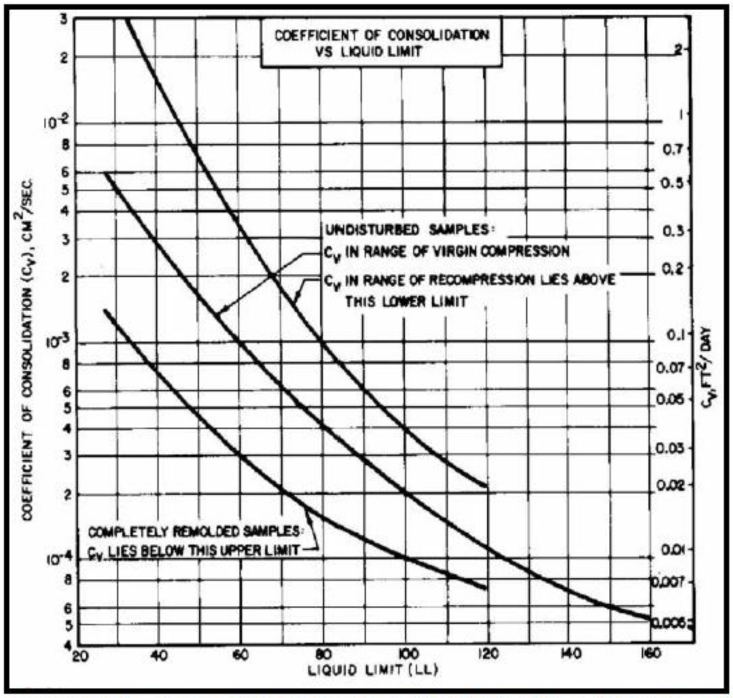

Figure 4: Consolidation Coefficient and Liquid Limit Relationship (NAVFAC DM-7.1)

Preconsolidation Pressure (σ'_p): The effective preconsolidation stress (σ'_p) in soils is used to determine whether to use the Compression and/or Recompression Index. The effective preconsolidation stress (σ'_p) is the maximum past pressure that a soil has been exposed to since deposition. If one-dimensional consolidation test data is not available, the effective preconsolidation stress (σ'_p) can be estimated using a correlation to shear strength (S_u) and Plasticity Index (PI) (NAVFAC DM-7.1).

$$\sigma' = \frac{S_u}{(0.11 + 0.0037PI)} \qquad \text{Equation 200E-1\_2}$$

Engineering Properties of Rock

Engineering properties of rock are generally controlled by the discontinuities within the rock mass rather than the properties of the intact material. Therefore, engineering properties for rock must account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. Rock properties can be divided into two categories: 1) intact rock properties and 2) in-situ rock mass properties.

Intact Rock Properties

Intact rock properties are measured from laboratory tests on samples obtained from coring or field mapping of exposures. Intact rock engineering properties obtained from laboratory tests can include specific gravity, unit weight, compressive strength, tensile strength, elastic modulus, Poisson's ratio and shear strength. Typical property values for different rock types can be found in Goodman (1989), Sabatini, et al. (2002), or AASHTO (2012).

Rock Mass (In-situ) Properties

Rock mass properties are determined by visual examination of rock cores and geologic mapping of discontinuities within the in-situ rock mass. For the majority of projects performed by the Soils Design Section, the rock mass properties are estimated using correlations to the rock type, recovery and rock quality designation (RQD). Recovery is defined as the length of core obtained expressed as a percentage of the total length cored. Rock Quality Designation (RQD) is defined as the total length of core pieces, 4 inches or greater in length, expressed as a percentage of the total length cored. Rock

Quality Designation (RQD) provides an indication of the integrity of the rock mass and relative extent of seams and bedding planes.

Other methods such as the rock mass rating (RMR) system or geologic strength index (GSI) may also be used to assess the design properties such as shear strength and elastic modulus for the rock mass. These alternate methodologies and correlations for determining rock mass properties are described in Sabatini, et al. (2002), Hoek, et al. (2002), and AASHTO (2012).

Properties of Predominant Geologic Units in Iowa

Loess

Loess is a windblown (eolian) soil consisting predominantly of silt-sized particles, and may contain minor amounts of sand and clay. Loess deposits range from almost 300 feet in the Loess Hills of western Iowa to less than 10 feet in south-central Iowa. Loess is typically not found in Iowa floodplain regions; however, loess-derived alluvium is present. Loess-derived alluvium has significantly different engineering properties compared to virgin loess. Loess generally has an increasing clay content as the distance increases easterly from the Missouri River (or other source). The clay component of loess (along with calcium carbonate in certain deposits) has been attributed to the cohesion that allows loess to stand vertical. However, upon wetting, the loess can collapse, resulting in excessive settlements. Claypan, a clay-rich loess, may be present underlying the topsoil in southeast Iowa (and sometimes in south-central Iowa) on the broad loess-capped upland flat areas. Claypan materials are unsuitable for use in roadway projects.

For engineering purposes, loess can generally be classified into three categories based on grain size: clayey loess, silty loess, and sandy loess. Table 4 presents typical engineering properties of Iowa loess.

Table 4: Typical engineering properties of Iowa loess.

material	soil type (USCS/AASHTO classification)	undrained		drained		dry unit weight (pcf)
		angle of internal friction (degrees)	cohesion (psf)	angle of internal friction (degrees)	cohesion (psf)	
claypan (loess derived) LL = 30 to 80 PI = 10 to 40	CL, CH A-4, A-5 , A-6 and A-7	8 to 12	50 to 600	15 to 25	0 to 200	75 to 95
clayey loess LL = 30 to 50 PI = 10 to 25	CL A-4, A-5 , A-6 and A-7	8 to 11	50 to 300	20 to 25	50 to 100	75 to 95
silty loess LL = 20 to 30 PI = 0 to 10	CL, CL-ML, ML A-4, A-5 , A-6 and A-7	11 to 14	50 to 150	22 to 28	0 to 50	70 to 90
sandy loess LL = nonplastic PI = nonplastic	ML A-3	13 to 16	0	26 to 32	0	70 to 90

Glacial Deposits

Glacial deposits consist of all material deposited or transported by a glacier or a glacier's meltwater stream and are found throughout Iowa. Glacial deposits that may be encountered within Iowa include: 1) the Des Moines Lobe, which consists of the most recent glacial deposits (Wisconsin age) and extends south to the site of modern-day Des Moines; 2) Illinoian glacial deposits that are found only in southeast Iowa; and 3) Pre-Illinoian glacial deposits that are found throughout the state (generally buried beneath loess or alluvium or sporadically exposed in northeast Iowa). See Figure 5.

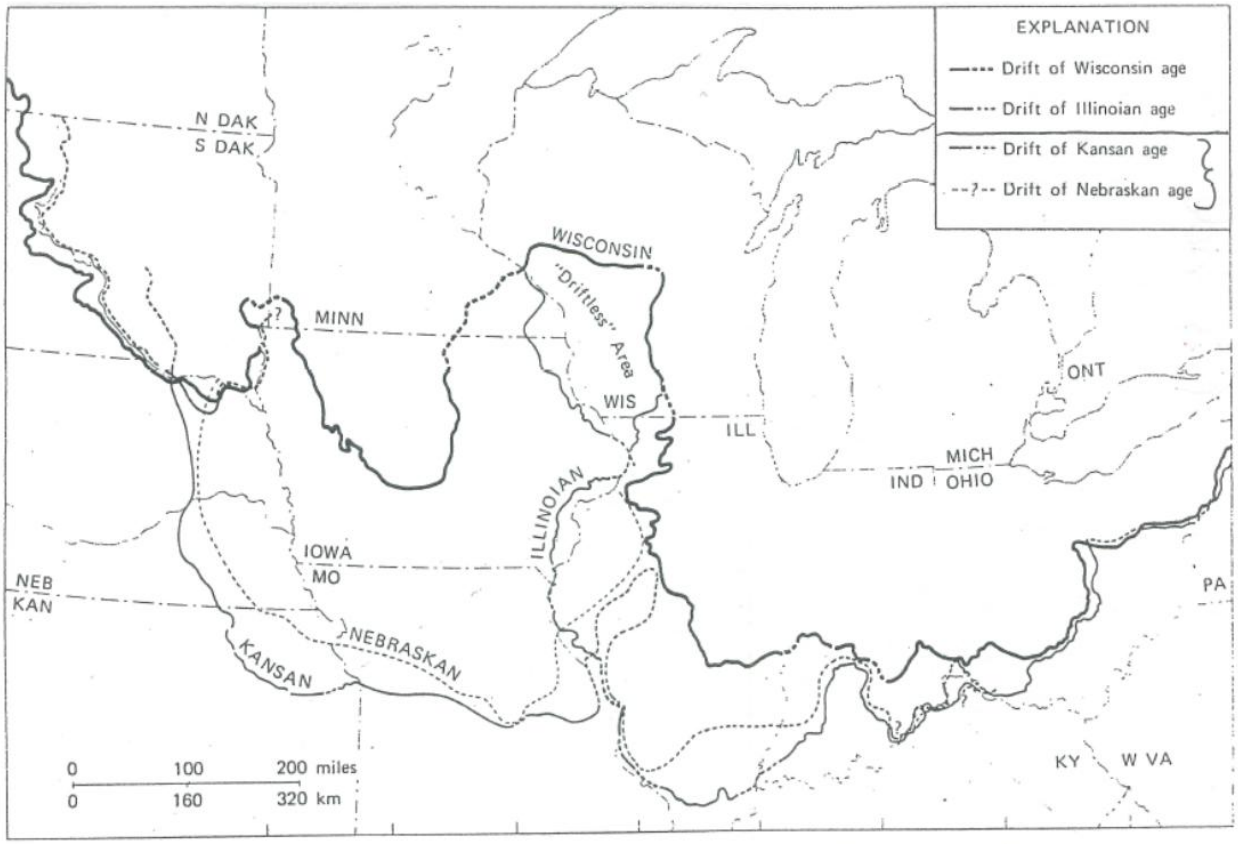

Figure 5: Limits of four glacial stages in Central North America (Prior 1976).

Glacial Till: Glacial till is predominately cohesive and consists of non-stratified deposits of clay, silt, sand, and gravel with cobbles and occasional boulders. In many instances in Iowa, typical glacial till is composed of roughly one third clay, one third silt, and one third sand with a few percentages of gravel. Sand and sand/gravel seams are common in many glacial till deposits. Boulders are common in these deposits and can range from a foot or two in diameter to the size of a bus. Glacial till has a relatively low permeability because of the fines content and the density. However, artesian conditions may be encountered within the water-bearing sand/gravel lenses in the glacial till.

Glacial Outwash: Glacial outwash is a partly-sorted mixture of clay, silt, sand, and gravel. Glacial outwash is predominately granular and consists of clayey/silty sands, fine sands, or clayey/silty gravels. Glacial outwash properties are similar to glacial till; however, glacial outwash tends to be predominately granular and consists of clayey/silty sands, fine sands or clayey/silty gravels. Glacial outwash is quite frequently stratified, mixed and non-uniform. Glacial outwash materials generally have a relatively high permeability, are water bearing, may have artesian conditions, and are susceptible to caving in open excavations or drilled holes.

Gumbotil: Gumbotil is a highly weathered soil that developed from glacial till during interglacial periods. Gumbotil is predominately found in the southern portion of the state. Gumbotil is frequently distinguished from the dark gray and brown glacial till by its light gray color and minimal sand content. Freshly excavated exposures are typically very hard, but will swell and soften rapidly upon exposure to moisture. Gumbotil is unsuitable for use in roadway projects.

Table 5 outlines the typical engineering properties of Glacial Till, Glacial Outwash, and Gumbotil. Engineering judgment and laboratory testing should be used in the evaluation and assignment of engineering properties.

Table 5: Typical engineering properties of glacial deposits.

material	soil type (USCS classification)	undrained		drained		dry unit weight (pcf)
		angle of internal friction (degrees)	cohesion (psf)	angle of internal friction (degrees)	cohesion (psf)	
Glacial Till	CL, CH, CL-ML A-4, A-5 , A-6 and A-7	10 to 15	100 to 1500	20 to 30	50 to 500	110 to 120
Gumbotil	CL, CH A-4, A-5 , A-6 and A-7	8 to 11	100 to 1000	16 to 22	50 to 500	90 to 115
Glacial Outwash	GW, GP, GW-GM, GP-GM, SW, SW-SM, SP, SC, SM A-1, A-3, A-3	25 to 34	0 to 50	28 to 38	0	90 to 110

Alluvium: Alluvium is all material deposited by streams. Alluvial soils are typically composed of clayey over-bank deposits underlain by silts, sandy silts, clayey silts, and silty sands. These soils generally have no continuity in their layering at shallow depths across a given site due to the complex process of erosion and deposition by flowing water and the varying depositional processes. However, continuity of soil layering does usually increase with depth in larger streams and rivers. Near surface soils normally consist of a combination of deposits deposited by slack water during flooding and tend to be finer-grained cohesive and moderately cohesive materials. Below the more cohesive blanket of surface and near surface alluvial soils, the deeper alluvial soils generally become less cohesive and grade from sandy silts and silty sands to medium to coarse sands, which become coarser grained and more dense with depth. Locally, clay filled abandoned channels and other features are often present and may contain isolated zones of clay and other cohesive soils.

The engineering properties of alluvial soils can vary significantly, so characterization of these materials through laboratory testing is preferred. Preliminary engineering properties for the cohesive and non-cohesive materials can be estimated using the correlations presented earlier in this section.

Bedrock

The bedrock in Iowa generally consists of sedimentary limestone, dolomite, shale, and sandstone. Thin layers of coal and gypsum are also found. With the exception of Sioux Quartzite exposed in far northwestern Iowa and the Manson Impact Structure in Calhoun and Pocahontas County, metamorphic and igneous rocks are found deeply buried beneath sedimentary rock and unconsolidated surficial deposits. The bedrock generally dips to the southwest at 2 degrees. The engineering properties of rock vary, so characterization of these materials through laboratory testing is preferred. Preliminary engineering properties for the bedrock can be estimated using the referenced publications presented earlier in this section.

Properties of Engineered Materials

Compacted soils are used to construct roadway embankments and bridge approaches. Selection of soil parameters for compacted soils should be based on laboratory testing or conservatively estimated from past test data and field correlations. Table 6 provides a guideline for engineering property selection on engineered fills based on the Soils Design Section’s experience with soil materials typically found in Iowa.

The engineering properties are based primarily on classification, gradation, and compaction requirements, with consideration of the geologic source of the fill material. For suitable materials, where the material types vary, additional laboratory testing will be required to evaluate the range of engineering properties to be considered. For existing and new cohesive highway embankments, the Soils Design Section's standard of practice has been to assume a cohesion of 600 psf and a 12 degree friction angle for shear strength parameters; however, engineering judgment and project specific considerations must also be used in all instances.

Table 6: Typical engineering properties of engineered materials.

material	Iowa DOT standard specification	soil type (AASHTO/USCS classification)	friction angle (degrees)	cohesion (psf)	total unit weight (pcf)
suitable materials	2102.02,A	AASHTO T-99 Proctor Density of 95 pcf or greater, and AASHTO M 145 Group index less than 30 Liquid Limit (LL) < 50	Perform lab testing	Perform lab testing	Perform lab testing
select soils (cohesive)	2102.02,D,1,a	A-6 or A-7-6 soils of glacial origin (excluding Gumbotil and Palesols) Must have a AASHTO T-99 proctor density of 110 pcf or greater. Must have 45% or less silt content. Must have a plasticity index (PI) > 10	20 to 30	50 to 600	120 to 135
select soils (granular)	2102.02,D,1,b	A-1, A-2, or A-3 (0) / GW, GP, GW-GM, GP-GM, SW, SW-SM, SP Must have a AASHTO T-99 proctor density of 110 pcf or greater. Must have a combined silt and clay content of 15% or less (finer than 0.074 mm or #200 sieve). Must have a plasticity index (PI) of 3 or less	30 to 45	0	125 to 135

Final Property Selection

Engineering judgment and project specific considerations, combined with parametric analyses as needed, should always be used in selection of engineering properties. Previous experience with geologic units and other laboratory and field data should also be considered in the evaluation. The geotechnical engineer must recognize the variability of materials, and the influence of the uncertainty on the design level of safety. If the impact of this uncertainty is likely to be significant, parametric analyses should be conducted, or more data could be obtained to help reduce the uncertainty. The selection of engineering properties to be used for design is outlined in Sabatini, et al. (2002).

References

1. AASHTO, 2012. LRFD Bridge Design Specifications, Sixth Edition.
2. Goodman, R. E., 1989. Introduction to Rock Mechanics, 2nd Edition, John Wiley & Sons, New York, NY.
3. Hoek, E., Carranza-Torres, C., and Corkum, B., 2002, "Hoek-Brown Criterion – 2002 Edition," Proceedings NARMS-TAC Conference, Toronto, 2002, 1, pp. 267-273.
4. Ladd, C .C., 1973, Estimating Settlement of Structures on Cohesive Soils. Presentation prepared for the Foundations and Soil Mechanics Division of the ASCE, April 9-10, 1973.

5. McGregor, J. A. and J. M. Duncan, 1998. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice. Virginia Polytechnic Institute and State University.
6. Meyerhof, G. G., 1956. Penetration Test and Bearing Capacity of Cohesionless Soils. Journal of the Soil Mechanics and Foundation Division, Proc. ASCE, 82, SM-1, pp. 866-1 to 866-19.
7. NAVFAC, 1971. Design Manual: Soil Mechanics, Foundations, and Earth Structures. DM-7.
8. Peck, R.B., Hansen, W.E. and Thornburn, T.H., 1974. Foundation Engineering. Wiley.
9. Prior, J.C., 1976. A Regional Guide to Iowa Landforms. Iowa Geological Survey Education, Series 3. State of Iowa.
10. Sabatini, P. J., Bachus, R. C., Mayne, P. W., Schneider, T. E., Zettler, T. E., 2002. Evaluation of Soil and Rock Properties. FHWA-IF-02-034, Geotechnical Engineering Circular No. 5.

Chronology of Changes to Design Manual Section:

200E-001 Engineering Properties of Soil and Rock

5/19/2015	Revised Correction in Glacial Till on page 9: glacial till is composed of one third clay, one third silt, and one third sand with a few percentages of clay.
1/15/2014	NEW New