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The Pleistocene of Iowa: An Engineering Appraisal

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Abstract. Pleistocene deposits from wind, water and ice dominate the soil engineering in Iowa. For foundation engineering the *in situ* properties of soils must be evaluated, the most pertinent properties being shear strength and compressibility. Alternately if the soil is to be used as a construction material, for example in embankment or earth dam construction, disturbed properties such as grain size and plasticity are utilized and are reflected in engineering classification. The shear strength, compressibility, and engineering classification of the various types of Pleistocene and Recent deposits in Iowa are presented and discussed, and examples of the use of shear strength data in slope stability and bearing capacity problems are presented.

Pleistocene and Recent materials literally form the foundation for nearly all engineering structures in Iowa. In some cases such as earth dams or highways, they practically comprise the structure. Use of such non-indurated materials in construction constitutes the field of soil engineering.

Because of the sometimes obscure geological boundary between the Pleistocene and the Recent, for example in alluvial deposits, the Recent will be included but not separated in this discussion.

Materials of the Iowa Pleistocene are primarily loess and glacial till, each of which accounts for about 40 percent of Iowa's land surface. Alluvium comprises practically all the remaining 20 percent.[11] Residual soils on bedrock probably account for less than a percent.

The duality of engineering usage of soils—either undisturbed as a foundation material, or disturbed, selected, proportioned, and compacted as a construction material—is reflected in the method of engineering evaluation. The two purposes may be generally categorized as foundation engineering on the one hand, and highway, airfield, and earth dam engineering on the other. There is some overlap since highways *et cetera* need adequate foundations, and foundations are occasionally built up in layers in the manner of highway embankments.

PROPERTIES IN SITU

The main concerns in foundation engineering are bearing capacity, which is safety from shear failure, and settlement due to consolidation of the soil.

Bearing capacity depends on soil unit weight and shearing strength. Unit weight may be measured or fairly reliably estimated from a geological identification of the material as indicated in Table 1. The most important single factor affecting unit weight is water. At moisture contents below saturation, water adds to the unit weight of the soil, whereas submergence decreases unit weight 62.4 pcf.

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The role of unit weight in bearing capacity can be seen by reviewing the Terzaghi equation for bearing capacity, q_o:

$$q_o = \frac{\gamma b}{2} N\gamma + cN_c + \gamma DN_q \qquad (1)$$

where γ is the soil unit weight, b the width of the footing, D the depth of the footing, and c, $N\gamma$, N_c, and N_q are functions of shearing strength. For a cohesionless sand the equation simplifies to:

$$q_o = \gamma \left(b \frac{N\gamma}{2} + DN_q\right). \qquad (2)$$

Thus bearing capacity of sand is proportional to its unit weight, γ, and submergence reduces bearing capacity about one-half. Most foundation failures on sand therefore occur during periods of high water. For frictionless clays the equation simplifies to $q_o = cN_c$, and unit weight is much less important.

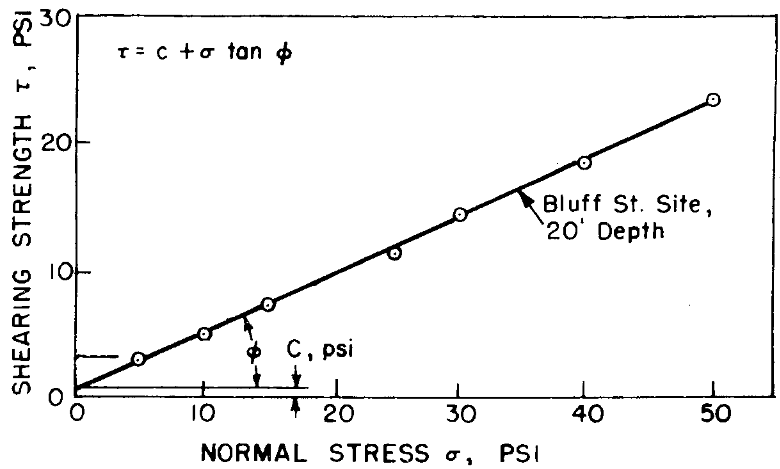

Figure 1. Shear test data for loess at Prospect Hill, Sioux City. A linear failure envelope may be described by the angle of internal friction, ϕ, and the cohesion intercept, c. Data after Lohnes and Handy.[9]

Shearing Strength

The shearing strength of soil depends in part on the pressure normal (perpendicular) to the plane of shear. This relationship is approximately linear, and is analogous to sliding friction. Shearing strength is in part attributed to sliding friction between the soil grains. The proportionality factor between shearing resistance and normal pressure is termed the coefficient of internal friction. If shearing resistance is plotted vs. normal stress, as in Figure 1, the coefficient of internal friction is equal to the tangent of ϕ, where ϕ is the "angle of internal friction."

Table 1
Some In Situ Shear Strength Parameters

Material	γw pcf	c, psi	ϕ degrees	Ref.
Loess[3]				
Friable <20% clay	75- 95[1]	1.3 ± 0.3[2, 3]	24.9 ± 0.6[2]	5, 6, 9
Medium 20-40% clay	90-105	4-5[3]	24	5, 6, 10
Plastic >40% clay	95-110[4]	10-22[6, 3]	n.d.	5, 6, 8
Dune sand	100-110	0-0.4	37	7
Alluvium				
Sand	n.d.	0	30-40[5]	4
Clay	n.d.	2-4[6]	low, n.d.	4
Glacial deposits				
Unoxidized till (preconsolidated)	140-150	10[6]-30	9	8
Oxidized till	130-140	3-4	15-21	2, 8
Sand pockets	130-140	0	30	8
Gumbotil (Yarmouth-Sangamon Paleosol)	n.d.	18-36[6]	n.d.	8
Swale filling (Webster soil series)	125	10	5	7

[1]Increases with depth.[6]
[2]± indicates 90 percent limits on the mean.
[3]Cohesion of loess decreases 0 - 0.3 psi on saturation.[1, 10]
[4]Higher mainly because of higher natural moisture content.
[5]Estimated from Standard Penetration Test blow counts.
[6]Rough estimate, one-half the unconfined compressive strength.

Internal friction in soil is not solely due to sliding friction or ϕ would not be increased by artificial compaction. This increase is attributed to interlocking, or more precisely the requirement that grains of a compact soil must roll over one another and increase the total volume to allow shearing. The resulting volume increase is termed dilatancy. Theoretical considerations show that this energy component of shearing strength is also proportional to normal pressure, and therefore may be included in the "angle of internal friction."

A third component of shear strength is called cohesion, and is independent of applied normal pressure. The modern picture of cohesion is that it is usually the result of an earlier loading or preconsolidation. In a "normally consolidated" clay the shearing strength increases with normal pressure, as in sand; however if the normal pressure is released the mineral grains in cohesive soils retain their high regard for one another and therefore retain much of their shear strength (Figure 2). This affinity is due to surface interactions of the clay particles.

Thus the higher the preconsolidation pressures on clay, the higher the cohesion or inherited strength. Some soils, e.g. lake sediments or loess, are preconsolidated only by their own weight and have relatively low cohesion, whereas other soils such as glacial till or shale may be heavily preconsolidated and have a high cohesion. High preconsolidation followed by unloading, as in shale or till, often causes fracturing, weakening the material.

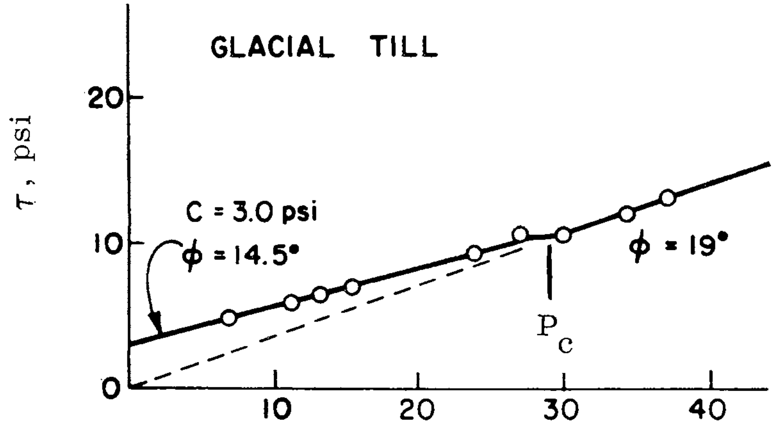

Figure 2. Shear test data for Cary till, Ames, Iowa, showing a two-stage shear envelope indicative of preconsolidation. The inferred effective preconsolidation pressure is about 30 psi. Data after Handy and Fox.[7]

Preconsolidation is not necessarily a purely mechanical phenomenon because the same effect is caused by shrinkage upon drying. Thus an alluvial clay may be more "preconsolidated" at the surface than at depth, giving a superficial cohesive "skin" on the deposit.

Still another factor affecting shear strength is the pore water pressure. Positive pore pressure acts to oppose the applied normal stress and decreases the apparent angle of internal friction. Strength parameters corrected for pore pressures are said to be on an "effective stress" basis, indicated by c' and ϕ'. Positive pore pressure results from "trapping" of water during consolidation, and therefore is highest in saturated clays. Negative pore pressures occur in unsaturated soils due to capillarity, and also in compact soils that are dilating during shearing.

Consolidation

Settlement of structures occurs when soil underneath the structure consolidates under the load. Such consolidation (incorrectly referred to by some geologists as "compaction") is time dependent, since time is required to squeeze the interstitial water out. The lower the permeability or the thicker the consolidating layer, the longer the time required for a given degree of consolidation. The soil property describing this rate is determined in the laboratory, and is the "coefficient of consolidation."

When time is sufficient, as in laboratory testing of very thin samples, an empirical relationship is obtained between the final density, or rather "void ratio," and the applied pressure. There is a more-or-less linear relationship to log of pressure, as shown in Figure 3. The slope of the line is a measure of soil compressibility, and is termed the "compression index."

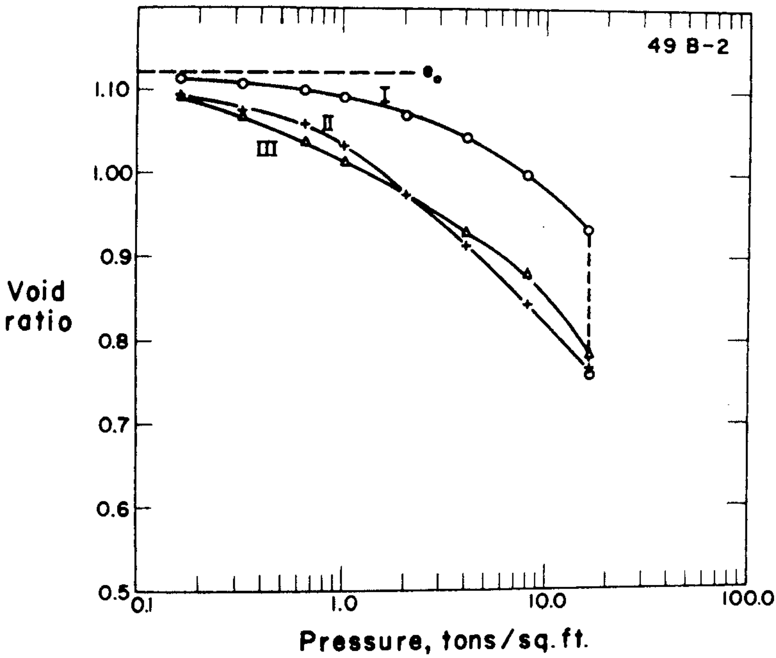

Figure 3. Consolidation void ratio vs. log pressure curves for Wisconsin loess at 75-foot depth at the Loveland type section. Curves show effects of different moisture contents: Curve I, 8.5 percent and then saturated, causing collapse (dashed line); II, 16.2 percent moisture; and III, 29.5 percent moisture (saturated). Data after Olson.[10]

Data and Discussion

For foundation engineering, soils are generally used "as is." Building sites are therefore investigated individually by consulting engineers, and there has been little attempt to systematically study undisturbed engineering properties of Iowa soils.

From the above discussion, pertinent properties include cohesion, angle of internal friction, preconsolidation pressure, compression index, coefficient of consolidation, etc. Unfortunately there is an absolute paucity of published data. Consolidation tests are seldom performed because of the time and cost, and the vast majority of strength tests performed do not differentiate between c and ϕ. For the most part the data in Tables 1 and 2 are estimates intended only as guidelines, and certainly are not to be construed as safe design values. Furthermore, regardless of the precision of such values, each proposed building site must be investigated because of the expectation of the unexpected.

Loess. Iowa is a classic area for loess, and this is one of the most interesting soil deposits in the state because of its unique properties and dramatic occurrences. Preconsolidation of friable loess is less than

Table 2

Consolidation Properties of Iowa Pleistocene Materials

Material	C_c	P_c Tsf	C_v,[3] ft^2/yr	Ref.
Loess (saturated)[1]	0.3	0-1[2]	2-6	10
Till	0.17	2.5	n.d.	8
Gumbotil	0.22	0.6	0.7-1.4	8

[1]Below saturation, consolidation is impeded by apparent cohesion from negative pore pressure.
[2]Usually less than the overburden pressure: See footnote 1.
[3]Lower for higher consolidation pressures and lower permeabilities.

the present overburden pressure, possible only if the loess has never been saturated.[10] For example, in Figure 3,[11] the inflection point indicative of preconsolidation load is less than 1 Tsf (14 psi) at a depth where the overburden pressure is about $75 \times 90 \div 2000 = 3.2$ Tsf. Thus preconsolidation must have been inhibited by a continuously lower moisture content. Curve I run at a lower moisture content shows a maximum curvature at about 3-4 Tsf, which is the proper range. Saturation, indicated by the dashed line on Curve I, presents a unique danger in that the loess then will collapse under its own weight giving severe settlement.

Foundation troubles are common in loess, and almost inevitably can be traced to:

1. Footing levels located near the bottom of the loess layer, in the zone of either an occasional or continuous perched water table.
2. Interruption of surface drainage: damming, ponding, etc.
3. Other sources of water: broken mains, improper downspouts, lawn watering, etc.

Loess is usually referred to as the "clay" that stands majestically in vertical cliffs for no reason at all, or for reasons that defy the meager intelligence of man. Actually Iowa loess cliffs are seldom vertical except in the upper 7-10 feet where they represent either a cut or a tension crack (Figure 4). Furthermore the maximum heights and angles of cuts are consistent with measured densities and shear strength.[9] Modal slope angles occur at about 77 degrees and 51 degrees, also predictable from shear strength parameters,[9] and corresponding to maximum stable heights of about 25 feet and 65 feet, respectively. For steep slopes $H_{max} \cong 0.1$ c, where H is in feet and c is in psf.

Relatively little shear strength data are available on more clayey loess or loess-derived soil profiles. As the clay content increases, cohesion in non-saturated soil probably increases while ϕ stays about the same. Saturation reduces cohesion to near zero, as in the case of friable loess, probably because the loess derives its cohesion not from

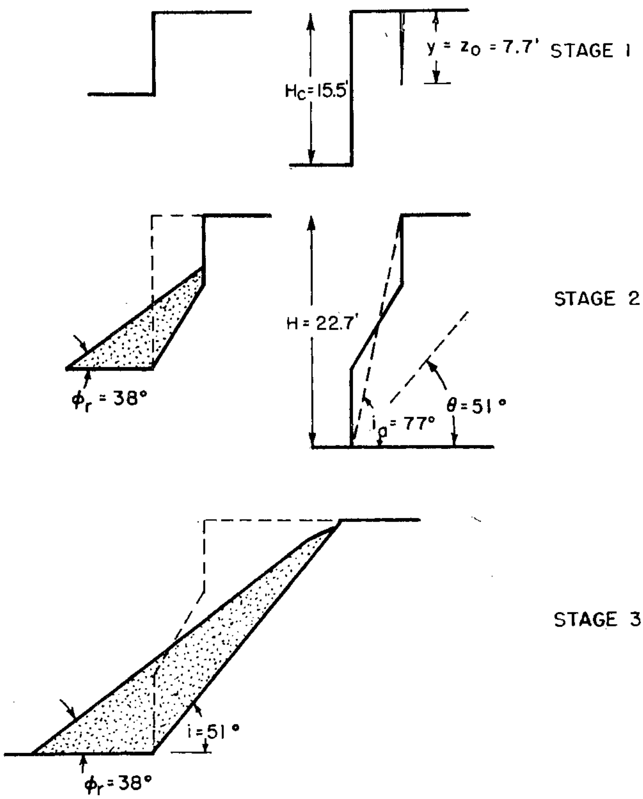

Figure 4. Inferred sequence of slope failures from vertical downcutting in friable loess: Stage 1, vertical tension crack leading to a shear failure at $45° + \phi/2 = 57°$. Stage 2, further downcutting leading to a shear failure at an angle $\phi/2 + i_a/2 = 51°$. Stage 3, $\phi\gamma$ = angle of repose. After Lohnes and Handy.[9]

consolidation, but from capillary and electrostatic attractions in the clay-water system. Loessal B horizons probably should exhibit the more permanent preconsolidation cohesion because of drying shrinkage, evidenced by a blocky structure.

Very few consolidation tests are available on undisturbed loess samples, and indicate a high and rapid compressibility, indicated by high values of C_c and C_v (Table 2).

Dune Sand. Dune sand is common as the first bluff along outwash streams, often as a windward facies of loess. The low clay content means a cohesion of almost zero. In active dunes ϕ is about equal to the angle of repose exhibited on the leeward slope, and is approximately equal to the angle of sliding friction (24 degrees for quartz, 32 degrees for feldspar).[12] Iowa Pleistocene sand dunes apparently have consolidated sufficiently to give an additional interlocking component, increasing ϕ (Table 1).

Glacial Till. Glacial till in Iowa varies from a hard, rock-like material to a relatively loose, friable mixture, depending on the degree of preconsolidation under ice and soil overburden pressures. Measured preconsolidation pressures probably do not reflect the true ice thickness because consolidation was inhibited by lack of drainage. For example, the P_c for Table 2, 2.5 Tsf, corresponds to an ice thickness of only 80 feet for Kansan till at Cedar Falls, Iowa.

Till cohesion therefore varies at least 10-fold depending on preconsolidation, with minor opposite variations in friction angle (Table 1). Preconsolidation of till is also reflected by a low compressibility (Table 2).

Soils developed on glacial till usually have lower shear strength than the till and are best avoided for foundations when the stronger material is close below. Webster and associated swale soils are soft and contribute to wet basements. Gumbotil (Yarmouth-Sangamon paleosol) is thick and is a better foundation stratum than the loess immediately above. These soils have medium to high cohesions and low friction angles, and high to medium compressibility depending on degree of preconsolidation either from weight or from drying.

Alluvium. Engineering properties of Missouri River floodplain soils were studied by Dahl[4] and summarized by Dahl et al.[3] Ten "standard penetration" tests, whereby an 18-inch sampler is driven with a 130-pound weight falling 30 inches, gave 10 to 74 blows per foot, averaging 38 bpf on point bar and channel sands and on the floodplain sand substratum. Blow counts tended to be higher in point bars and lower in braided channel sands below the mouth of the Platte. These tests indicate that the sands are of medium density, with ϕ in the neighborhood of 30-36 degrees.

Clay plugs, which are clay-filled oxbow lakes, are as much as 30-50 feet deep north of the mouth of the Platte and 5-15 feet deep south of the Platte, reflecting the shift from a meandering to braided river. These highly compressible fills contain 60 to 75 percent montmorillonite clay and are avoided for foundations, either by site selection or by use of pile to the underlying sand. Where pile are used, consideration must be given to "negative skin friction" as the weight of the slowly consolidating clay gradually comes to push downward on the pile. Road embankments crossing clay plugs usually settle several feet, necessitating periodic filling and reconstruction of the pavement until settling finally stops. The upper 3 to 5 feet of clay plug material is often deceptively hard due to drying.

Outside the meander belt, the flood basin deposits are extremely variable, and range from 25 feet thick near the meander belt to 45 feet at the valley wall. The weakest soils are clay backswamp deposits farthest from the river, with properties not unlike clay plugs although the thickness is more uniform. Natural levee and other overbank deposits close to the meander belt are more silty and somewhat stronger.

Deposits of other river floodplains in Iowa are probably similar, but on a smaller scale. For example, thickness of the clay plugs relates to the maximum depth of scour, which depends on size of the river.

Usefulness of Shear Strength Data. Some examples of calculations utilizing shear strength data in Table 1, are shown in Tables 3 and 4. These are strictly for purposes of illustration, and are not be construed as safe design values.

Table 3 was calculated for possible geological interest, and includes no safety factor. The calculated maximum height of a vertical cut is mainly a function of soil cohesion, and ranges from 0 for cohesionless sand to 100 feet for preconsolidated glacial till. Saturation reduces the maximum height for loess from 15 to only about 2 feet.

Table 3

Approximate Maximum Height of Vertical Cut
(No Safety Factor)

$$H_c = \frac{4c}{\gamma} \tan\left(45^\circ + \frac{\phi}{2}\right)$$

	Feet
Dune sand	0
Friable loess, natural moisture content	15
Friable loess, saturated	2
Oxidized and leached till	20
Preconsolidated till	100

Note: *Not to be used for design.* Tension cracks will reduce these heights one-half.

Table 4 shows calculated bearing capacities for a 1-foot wide footing at a depth of 4 feet, and apply only to footing with this width and depth. The bearing capacity per unit area can be increased by making the footings wider or deeper or both. Both friction and cohesion are important, so the dune sand has as high a bearing capacity as preconsolidated till. The effect of saturation on loess is not so pronounced if internal friction stays the same; however, these calculations say nothing about consolidation, which not only will cause settlement, but also will induce temporary positive pore pressures that can drastically reduce the effectiveness of internal friction. Thus data such as in Table 4 must be tempered with judgment or more refined test data. A commonly used "safe" bearing capacity for saturated loess is only 0.25 T/ft.

Other uses for shear strength and density data are for calculations of slope stability, landslides, pressures on retaining walls or underground conduit, bearing capacity of piles, etc.

Table 4

Shear Bearing Capacities of 1-Foot-Wide Continuous Footings 4 Feet Deep (Factor of Safety = 3.0)

$$q_o = \frac{\gamma b}{2} N\gamma + cN_c + \gamma DN_q$$

	T/ft.
Dune sand	5
Friable loess, natural moisture content	1.5
Friable loess, saturated	0.5
Oxidized and leached till	1.5
Preconsolidated till	4.5

Note: *Not to be used for design.* No allowances made for settlement or positive pore pressures.

Disturbed Properties

In contrast to foundation engineering where shear strength and consolidation are direct design criteria, for highway embankments, road bases, and similar uses, soils are selected and sometimes even proportioned and mixed to obtain better engineering qualities; the moisture content is carefully controlled; and they are compacted with an equivalent standardized compactive effort. Because of the latter procedures, soils with similar gradations and clay mineralogy tend to have similar shear strength, consolidation and volume change properties. Therefore instead of expressing these data directly, materials engineers prefer to express soil properties indirectly by means of an engineering classification.

Engineering soil classifications are based on particle size distribution and plasticity. The latter depends on clay mineralogy and is defined by the liquid limit, or minimum water content for a soil to behave as a liquid; and the plastic limit, or minimum water content for a soil to be moldable without breaking. The "plasticity index" or PI is the numerical difference between the two and describes the range of moisture contents through which a soil exhibits plastic behavior.

The oldest and still one of the most widely used engineering classification systems is the BPR (Bureau of Public Roads) or AASHO classification, originally proposed by Terzaghi and Hogentogler in the 1920's, and subsequently revised and modified by various committees. In this system: A-1 soils are the best, being well-graded (i.e., poorly sorted) sandy gravels; A-2 soils are sands or gravels with appreciable fines; A-3 soils are cohesionless sands; A-4 soils are very fine sands and silts, highly susceptible to frost action; A-5 soils are similar, but contain mica or other minerals making them difficult to compact; A-6 and A-7 soils are expansive clayey soils and clays; and A-8 soils are peat and muck, which are unstable. Relative plasticity properties within each group is indicated by a "group index."

A simple but somewhat less definitive classification was devised by

Casagrande in the 1940's and is gaining wide acceptance. It is now termed the Unified Classification. Most classes combine two letters: G = gravel, S = sand, M = silt, C = clay, O = organic, Pt = peat; and W = well graded, P = poorly graded, L = low PI and H = high PI. Thus ML is a silt with a low PI, CH a clay with a high PI, SP a poorly graded sand, etc.

A third classification used by the Federal Aviation Agency rates soils as E-1 through E-13, again on the basis of gradation and plasticity. These soil classes are used directly in pavement design.

The AASHO classification of Iowa Pleistocene deposits is shown on a map by Welp and Anderson[14] from Iowa State Highway Commission data. This map is reproduced in Figure 5. For a more detailed discussion, particularly in relation to soil series, readers are referred to that report.

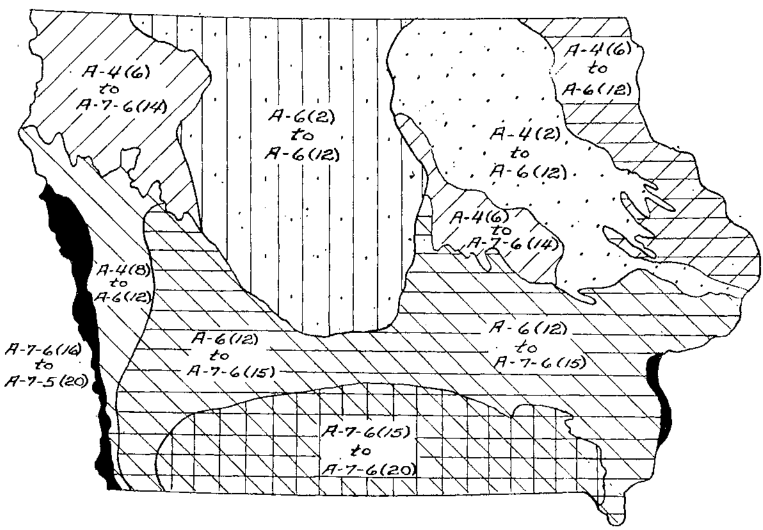

Figure 5. General soil classification areas in Iowa. After Welp and Anderson.[13]

More recently in the Soil Conservation Service county soil mapping program, soil samples have been routinely submitted to the Bureau of Public Roads for engineering tests. Results are reported in the county soil survey reports and should form a valuable catalogue of information on disturbed samples. Included are series, depth and horizon data, sieve and plasticity analyses, AASHO classification, and maximum compacted density and optimum moisture content with standard AASHO compactive effort. Attempt is also being made to develop a qualitative soil series rating system pertinent to rural and urban development; in addition to strength and compressibility, the rating system includes permeability factors pertinent to use of septic tanks, depth to water table pertinent to basement construction, shrink-swell potential, and potential for flooding.

A-4 Soils in Iowa

The A-4 soils, which are friable silts subject to severe erosion or frost action, include the thick loess deposits occurring along the outwash fringes of Iowa. These usually classify as A-4(8) or, if sandy, as A-4(6). Sandy till-derived alluviam of the "Iowan" plain classifies as A-4(2). Minor A-4 deposits occur as natural levees along major rivers. A-4 soils are satisfactory for embankments and may be stabilized with Portland cement for road bases.

A-6 Soils in Iowa

Farther from the loess sources the loess is more clayey and classifies as A-6(12). B horizons are better developed and usually classify even higher. A second and more important occurrence of A-6 soils is glacial till, regardless of age. This material differs from plastic loess in being more sandy, as reflected in the group index. Glacial till usually classifies as A-6(2) to A-6(12), with pockets of A-1, A-2, and A-3 gravels or sands. Soil profiles on till or medium loess classify as A-6 or higher, depending on the clay content. A-6 soils are less permeable than A-4 and not so severely affected by frost action. They are satisfactory for embankments, and may be stabilized with hydrated lime or Portland cement for subbase or base courses.

A-7-6 Soils in Iowa

The A-7-6 soils are heavy-textured clays, and include floodplain backswamp or slackwater deposits, loess or till-derived B horizons, plastic loess C horizon, and on late Wisconsin gravel moraine, clayey swale fillings such as the Webster soil series. The gumbotil paleosols usually classify a A-7-6(20), which is the maximum group index, indicating a highly plastic expansive clay.

A-7-6 soils are usable but not very suitable for embankments, in that they tend to swell several percent upon saturation and conversely shrink upon drying. Their use therefore depends on maintaining a nearly uniform moisture content. Flexible pavements may be preferred, both for their flexibility and their water-tightness. A-7-6 soils are very susceptible to stabilization by lime, which acts as a strong flocculent for montmorillonitic clay. Thus where such soils are apparently an insurmountable problem, the problem often may be surmounted with a few bags of lime.

A-8 Soils in Iowa

A-8 soils, peat and muck, are very minor in Iowa and occur primarily on the Cary lobe and on backswamp areas of floodplains. They are avoided wherever possible.

A-3 Soils in Iowa

A-3 soils are clean, well-sorted sands occurring in Iowa primarily on floodplains as alluvial point bar and channel sand deposits, and on uplands as stable dunes associated with Pleistocene outwash or alluvium. A-3 sands form a useful resource for both Portland cement and asphaltic concrete mixtures. Deposits are extensive in eastern Iowa, where they were mapped by Wickstrom and Davidson[14] (Figure 6).

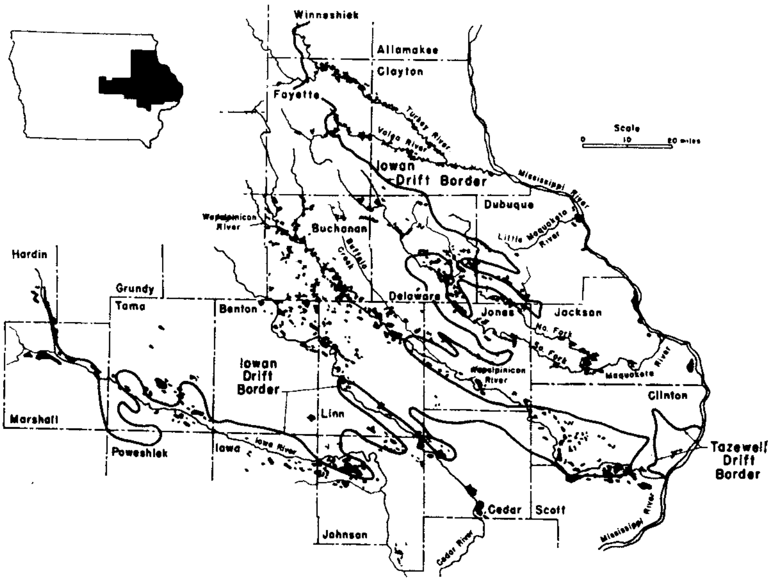

Figure 6. Fine sands in eastern Iowa. After Wickstrom and Davidson.[14]

A-2 Soils in Iowa

A-2 soils are sands or gravels with moderate plasticity. They are further classified as A-2-4, A-2-6, or A-2-7 depending on plasticity of the silt-clay fraction, analogous to A-4, A-6 and A-7 soils. They thus include bimodal mixtures of gravel plus silt or clay, or sand plus silt or clay, and are not common as primary deposits except as a result of composite sampling. An exception is A-2-4 silty sand, which sometimes occurs as natural levee or fine dune sand deposits.

A-1 Soils in Iowa

The coarse granular soils classify as A-1 for road building, A-1-a if predominantly gravel, or A-1-b if predominantly gravel plus coarse sand. These are primary outwash sands occurring either as terraces or as a substrataum to modern finer textured alluvium. They occur mainly in the northern half of the state, associated with the Wisconsin glaciation (Figure 7).

Summary

In general, Iowa's Pleistocene deposits rate good to poor as foundation soils, averaging fair. Most heavy structures require pile or mat foundations, even in areas of relatively good soils.

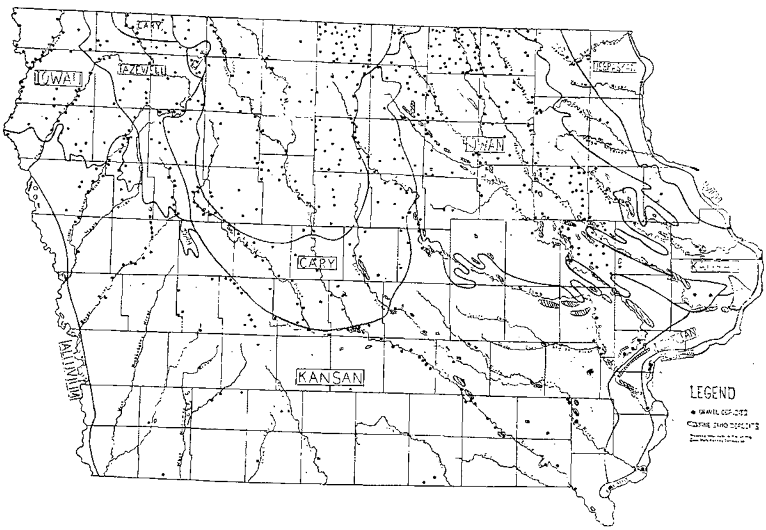

Figure 7. Gravel and sand deposits in Iowa. After Welp and Anderson.[13]

As a construction material the Pleistocene deposits rate as excellent to poor. Those rating excellent are commercially exploited sand-gravel deposits, but because of the predominance of materials at the other end of the spectrum, much attention has been given to chemical stabilization of soils for use as road subbase or base courses. Successful methods include soil-cement, which is best suited for sandy soils, and soil-lime, which is best suited for clays. Cement-treated sands or lime-treated clays have been successfully used in lieu of crushed limestone for base courses for blacktop roads and airfields.

Thus we engineers are learning to live or work with Pleistocene materials mainly as an accommodation to the agriculturists who could not live without them.

Acknowledgment

Most data in this paper were obtained as a result of continuing Highway Research Board sponsorship of soil engineering projects at the Iowa State University, Engineering Research Institute Soil Research Lab. The author also expresses his appreciation for encouraging words and criticism from Dr. Robert A. Lohnes, who originally conceived and organized this symposium.

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