Characterization of Loess for Deep Foundations

Robert L. Parsons, University of Kansas, Lawrence Kansas, USA, (785) 864-2946, rparsons@ku.edu

Rebecca M. Johnson, University of Kansas, Lawrence Kansas

Dan A. Brown, Dan Brown and Associates, Sequatchie, Tennessee

S. Dapp, Dan Brown and Associates, Sequatchie, Tennessee

James J. Brennan, Kansas Department of Transportation, Topeka, Kansas

ABSTRACT

This paper describes the results of a detailed analysis of a loess deposit at the deep foundations test site established by the Kansas Department of Transportation. Multiple CPT, SPT, and pressuremeter tests were conducted as part of the investigation. This paper includes an evaluation of the effectiveness of common correlations used with the CPT, SPT when used for classification of loess and estimating its properties. Results show that common correlations must be used with caution as significant errors in estimation may result from using methods developed for other soil types.

Laboratory testing included direct shear, triaxial, collapse, and consolidation testing, with many of the tests conducted on both vertically and horizontally oriented samples to evaluate anisotropic behavior. The results showed that strength properties could be considered to be isotropic.

INTRODUCTION

This paper describes the site investigation and test results conducted in association with the construction of a set of six drilled shafts in a loess deposit at the Kansas Department of Transportation geotechnical test site. The shafts were constructed for the purpose of lateral load testing for the development of p-y curves. This paper contains a summary of the origin and significant engineering properties of loessal soils, a discussion of soil properties at the test site as determined from laboratory and in situ tests, and an evaluation of existing correlations for in-situ test data when applied to loess. The results of the lateral load testing and resulting p-y curves for both static and repeated loading will be presented in a separate paper.

SCOPE OF STUDY

A series of laboratory and in-situ tests were conducted to characterize the test site. Laboratory testing included saturated and unsaturated triaxial testing, direct shear, consolidation and collapse testing. Field tests included SPT, CPT, and pressuremeter testing along with continuous soil profiling with an A. D. Bull Soil Sampler. Sampling was conducted in the summer of 2004 and in the summer of 2005 within two days of testing.

Two pairs of shafts with diameters of 760 mm and 1070 mm (30 and 42 in) were tested under static loading. A third pair of 0.76 m shafts was tested under repeated loading. Shaft deflections were measured using inclinometer soundings and correlated with CPT cone tip resistance (q_c). A hyperbolic model was developed to correlate ultimate soil resistance (P_{uo}) to the CPT cone tip resistance (q_c) for both static and repeated loading at any given depth and was used to develop a family of p-y curves unique to loess. This model may be entered into commercially available software for design of laterally loaded drilled shafts constructed in loess. A detailed description of all aspects of the project and analysis is provided by Johnson et al. (2006).

LOESS

Origin

Multiple competing theories concerning the origin of loess have been proposed. There are five different theories discussed by Marosi (1975); however, nearly all authors accept and discuss the theory of an aeolian origin. Terzaghi et al. (1996) define loess in general as uniform, cohesive, wind-blown sediment. Loess is a clastic soil mostly made of silt-sized quartz particles and loosely arranged grains of sand, silt, and clay. Cohesion is due to clay or calcite bonding between particles which are significantly weakened upon saturation. When dry, loess has the unusual ability to stand and support loads on nearly vertical slopes.

Loess was formed during arid to semi-arid periods following periods of Pleistocene continental glaciation. As the glaciers retreated, strong winds swept up sediments from the outwash. Larger particles were sorted and deposited near the original riverbeds while silt-size particles were transported downwind. The glacial till continued to be swept up and reworked throughout the arid times, creating a loosely arranged soil mass.

Loess soil particles are often loosely arranged with numerous voids and root-like channels. The coarser particles settle out near the source and finer particles are deposited progressively further away. Therefore, local differences occur in the type and quantity of mineral content. In general, the fabric of loess consists of fine, loosely arranged angular grains of silt, fine sand, calcite, and clay. Most of the grains are coated with thin films of clay and some with a mixture of calcite and clay. Coarser samples are generally better sorted than the finer ones (Swineford and Fry, 1951).

The granular components of loess are quartz, feldspars, volcanic ash shards, carbonates, and micas. The percent of composition varies with each site sampled, but in general quartz makes up around half the total volume of the deposit (Gibbs and Holland, 1960; Paliwal et al., 1965).

Color and particle size are strong identifiers of loess. It is commonly a buff, medium to coarse-grained silt with fine to very fine grains of sand. In general, the median grain size ranges from 0.02-0.05 mm (Swineford and Fry, 1951).

Loess is present in central parts of the United States, Europe, the former Soviet Union, Siberia, and in large parts of China and New Zealand (Bandyopadhyay, 1983). Within the United States, major loess deposits are found in Nebraska, Kansas, Iowa, Wisconsin, Illinois, Tennessee, Mississippi, southern Idaho, and Washington, as mapped on Fig. 1 (Gibbs and Holland, 1960). Thicknesses of loess deposits vary and may exceed 30 meters.

[FIG. 1] Outline of major loess deposits in the United States (Bandyopadhyay, 1983).

GEOTECHNICAL CHARACTERISTICS

Several characteristics are used to separate loess from other silty soils. In its natural state, loess has an open, cohesive particle structure with low density and high dry strength. Non-cohesive silty or clayey soils similar to loess in particle size, deposition, and open particle arrangement are not considered loess. They are considered wind-deposited silts, fine sands, or clays. Loess has a metastable structure due to the high degree of settlement and large loss of strength that may occur upon saturation.

Because the vertical permeability of loess is much greater than the horizontal permeability (Bandyopadhyay, 1983), strength and stability decrease for intermediate slope angles. Loess is subject to a large degree of consolidation, poor stability, seepage, erosion, and leaching of carbonates under various moisture and load combinations. Other defining characteristics include grain structure, color, and engineering properties.

Calcite is believed to be a significant cementing material in loess. It can be leached into the soil from above or can be brought into the soil by evaporation of capillary water from the groundwater below. However, clay is more commonly the bonding agent that gives loess its cohesive nature. Bandyopadhyay (1983) found montmorillonite clay to be the major cementing material in Kansas loess, while calcite "usually occurs in distinct silt-sized grains throughout the loess in a finely dispersed state rather than as a cementing material." Gibbs and Holland (1960) found that, in general, intergranular supports were composed mostly of montmorillonite clay with small amounts of illite, and contend that calcite serves as a secondary support structure and clay as the primary soil matrix.

Sheeler researched quantitative properties of loess, including specific gravity, Atterberg limits, permeability, density, shear strength, and natural moisture content as shown in Table 1 (Sheeler, 1968).

Specific gravity is influenced by local variations in the type and quantity of mineral content.

Values range from 2.57 to 2.78 with an average value of 2.66 (Gibbs and Holland, 1960; Bandyopadhyay, 1983; Sheeler, 1968).

Plasticity Indices for loess range from 5 to 37 and Liquid Limits range from 25 to 60 depending on the amount of clay present (Gibbs and Holland, 1960; Sheeler, 1968; Crumpton and Badgley, 1965). High Plasticity Index values correspond to high percentages of montmorillonite in the soil (Bandyopadhyay, 1983).

[TABLE 1] Range in Values of Engineering Properties of Loess in the U.S. Adapted from Sheeler (1968)

Property	Location							
	Iowa	Nebraska	Tennessee	Mississippi	Illinois	Alaska	Washington	Colorado
Specific Gravity:	2.58 - 2.72	2.57 - 2.69	2.65 - 2.70	2.66 - 2.73		2.57 - 2.79		
Analysis								
Sand, %	0 - 27	0 - 41	1 - 12	0 - 8	1 - 4	2 - 21	2 - 10	30
Silt, %	56 - 85	30 - 71	68 - 94	75 - 85	48 - 54	65 - 93	60 - 90	50
Clay, %	12 - 42	11 - 49	4 - 30	0 - 25	35 - 49	3 - 20	8 - 20	20
Atterberg limits								
LL, percent	24 - 53	24 - 52	27 - 39	23 - 43	39 - 58	22 - 32	16 - 30	37
PL, percent	17 - 29	17 - 28	23 - 26	17 - 29	18 - 22	19 - 26		20
PI	3 - 34	1 - 24	1 - 15	2 - 20	17 - 37	NP - 8	<8	17
Classification								
Unified	ML, CL, CH	ML, CL	ML, CL	ML, CL	CL, CH	ML, CL-ML		CL
Field moisture, %	4 - 31		12 - 25	19 - 38		11 - 49		8 - 10
Shear strength								
UU triaxial shear								
c, psi		0 - 67		2 - 10				
c, kPa		0 - 460		14 - 69				
ϕ		31 - 36°		0 - 28°				
CU triaxial shear								
c, psi	0 - 8			3 - 8				
c, kPa	0 - 55			21 - 55				
ϕ	28 - 31°			26°				
CD Direct shear								
c, psi	.3 - 1.8			0				
c, kPa	2 - 12			0				
ϕ	24 - 25°			32 - 33°				
CBR				10 - 13				

Permeability is influenced by soil properties such as particle size and shape, gradation, void ratio and continuity, and soil structure (Howe, 1961). It is a widely varied local feature with in-place vertical permeabilities of loess ranging from 1 x10^{-5} to 1 x10^{-3} cm/s, determined after consolidation was complete under a given load (Sheeler, 1968). Bandyopadhyay (1983) states the vertical permeability of Peoria loess in Kansas is on the order of 9 x10^{-4} cm/s and is much larger than the horizontal permeability, with the higher permeability caused at least partially by "the existence of vertical tubules and shrinkage joints within the soil mass". Terzaghi et al. (1996) viewed permeability in loess as an elusive property because the structure changes when it is saturated. It breaks down, becomes denser, and its permeability is decreased (Sheeler, 1968).

Cohesion and friction angle are controlled by the initial density, moisture content, and clay content of the loess. Reported angles of internal friction range from 28-36 degrees for samples tested with a moisture content below saturation (Sheeler, 1968). The cohesive strength varied from 0-483 kPa (0-70 psi) with the high values of cohesion associated with high unit weights. Also, cohesive strength increases with increasing clay content (Sheeler, 1968). There is a distinct difference in shear strength between wet and dry loess with dry loess having greater shearing resistance under an applied load and greater cohesion than when saturated.

Loess is often associated with terms such as "collapse," "hydroconsolidation," or "hydrocompaction" (Bandyopadhyay, 1983). Consolidation may be the most outstanding physical and structural property of loess. Its susceptibility to settlement makes it a potentially unstable foundation material. Because of the sensitivity of montmorillonite to moisture, an increase in moisture content may cause clay bonds to weaken, reducing the original soil strength. Slight variations in clay content and moisture content may cause localized consolidation or collapse. Saturated loess consolidates under lower stress conditions than when dry. Therefore, an increase in moisture content is often a more important contributor to collapse and consolidation than loading (Gibbs and Holland, 1960; McClelland et al. 1956).

In general, settlement is expected to be large for loess with a dry unit weights below 12.6 kN/m^3 (80 pcf) and small for those exceeding 14.1 kN/m^3 (90 pcf) (Gibbs and Holland, 1960; Bandyopadhyay, 1983). Therefore, loessal soils with low field densities and clay cementation can be expected to have a high consolidation and collapse potential (Bandyopadhyay, 1983). For dry loess, bearing capacity may exceed 480 kPa (10,000 psf) but may drop to 24 kPa (500 psf) upon saturation.

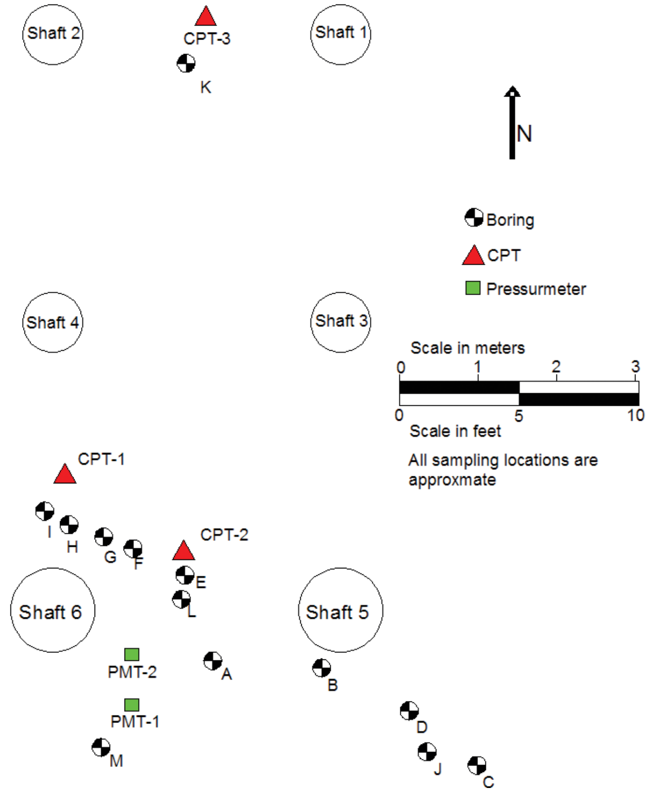
In-situ moisture contents of loess range from 4 to 49 percent. There is a strong correlation between regional average annual rainfall and the natural moisture content. Because the structure of loess is loosely arranged and filled with voids, rainfall quickly infiltrates and loess may remain dry within a few feet of the surface, unless there is a water table near the surface. Gibbs and Holland concluded that maximum dry strength occurs at moisture contents below 10 percent and high resistance to settlement should be expected. Soils with moisture contents between 10 to 15 percent have moderately high strength, with strength declining as moisture approaches 20 percent. Moisture contents above 20 percent are considered high and will permit full consolidation to occur under load. Saturation occurs at about 35 percent moisture (Gibbs and Holland 1960).

SITE INVESTIGATION

A deposit of loess located on the northwest corner of I-435 and State Highway 32 in Wyandotte County, Kansas was selected by the University of Kansas (KU) and KDOT for the full scale drilled shaft lateral load tests. The site is part of the Loveland member and was chosen for its deep, uniform deposit of loess and deep groundwater table.

Nine borings were drilled during June of 2004 and one was drilled in June of 2005 during the week of load testing. Locations of the borings and drilled shafts are shown in Fig. 2. Field tests included Standard Penetration Tests (SPT) in Borings A-D using an automatic hammer, a total of three cone penetration tests (CPT), two pressuremeter tests (PMT), and two continuous soil profiles obtained using an A.D. Bull Soil Sampler. CPT tests were conducted using an acoustic cone manufactured by Geotech AB of Sweden and PMT tests were performed using a Rocktest pressuremeter, Model G-AM, at depths of 0.6, 2, 1.5, and 3 meters (2, 6.6, 5 and 10 ft). All in-situ tests performed in 2005 were conducted within two days of the final lateral load test to provide the most accurate soil profile possible when determining the soil's response to loading. Undisturbed soil was sampled using 89 mm (3.5 in) diameter Shelby tubes at depths of 0.3 to 7.6 meters (1-25 ft).

[FIG. 2a] Test Shafts



[FIG. 2b] In-situ Testing and Drilled Shaft Layout

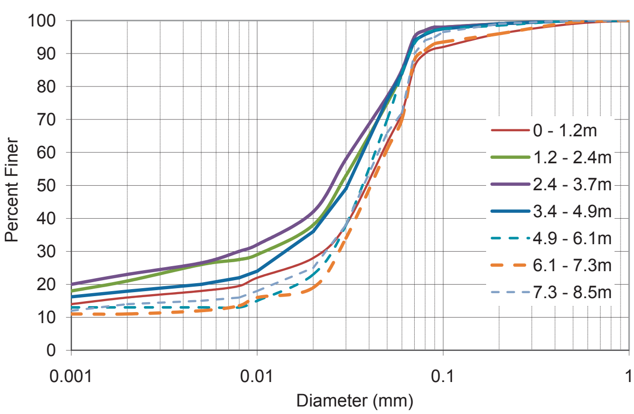
KDOT and KU conducted consolidated-undrained triaxial compression tests, unconsolidated-undrained triaxial compression tests, and unconfined compression tests on samples 71 mm (2.8 in) diameter with a height to diameter ratio of approximately 2.2:1. Direct shear, consolidation, and index property tests were also performed. Testing was conducted on samples obtained in June 2004.

Triaxial testing was conducted on unconsolidated-compression samples at the in-situ moisture content on 36 mm (1.42 in) diameter samples and consolidated undrained testing with pore pressure measurements was conducted on 71 mm (2.80 in) samples. Selected direct shear samples were trimmed with the vertical axis of the sample parallel to the long axis of the sampling tube and others with the long sample axis perpendicular to the sampling tube to determine if strength characteristics were anisotropic. Consolidation, collapse, and index property testing were also conducted.

Table 2 contains a description of the subsurface profile at the test site along with soil index parameters. Fig. 3 shows the range of grain size distributions to 8.5 meters (28 ft) and Fig. 4 shows the moisture content and saturation profile for both 2004 and 2005. Despite substantial rainfall in the days prior to testing in 2005, the moisture profiles are quite consistent between 2004 and 2005, with a higher moisture content near the surface in the more weathered, clayey soil and lower moisture contents below a depth of 4 meters (13 ft).

[TABLE 2] Specific Gravity and Atterberg Limits with Depth

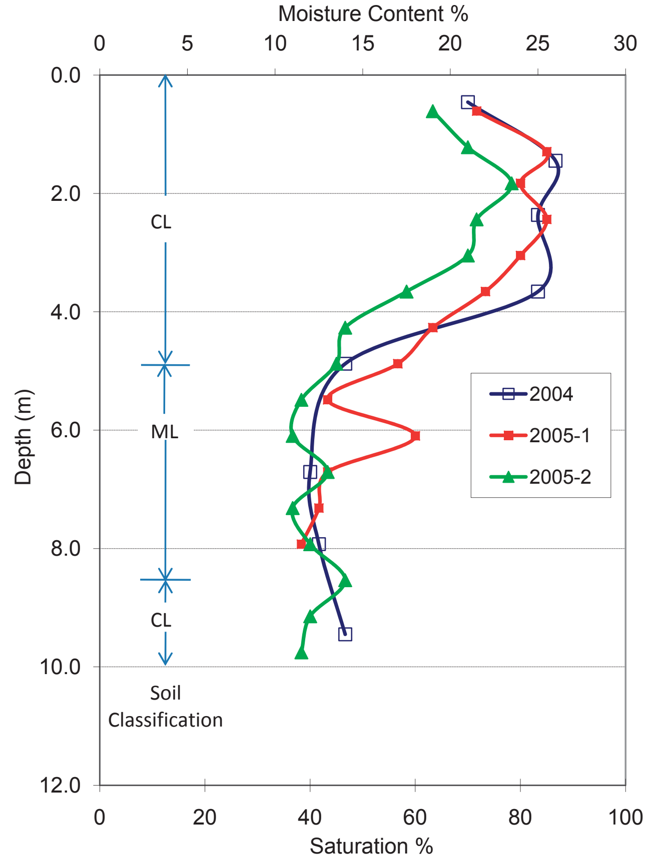
Depth (m)	G_s	Atterberg Limits LL	PL	PI	Classification
0-1.2	2.63	31	18	13	CL
1.2-2.4	2.68	36	17	19	CL
2.4-3.7	2.62	36	16	22	CL
3.7-4.9	2.62	33	18	15	CL
4.9-6.1	2.61		np		ML
6.1-7.3	2.61		np		ML
7.3-8.5	2.63		np		ML
8.5-9.8	2.63	38	17	21	CL



[FIG. 3] Grain size distributions with depth

COMPARISON OF SOIL CLASSIFICATION BY STANDARD METHODS AND CPT METHODS

The soil was classified based on laboratory testing as low plasticity clay (CL) from the ground surface to a depth of 4.9 meters (16 ft). The soil was classified as non-plastic to low plasticity silt (ML) from 4.9 to 8.5 meters (16 to 28 ft) below the surface. The soil was again classified as low plasticity clay (CL) from 8.5 to 9.8 meters (28 to 32 ft) below the surface. A more detailed summary is provided in Table 2.



[FIG. 4] Profile of moisture content and saturation for 2004 and 2005

The soil was also classified using a computer generated CPT profile based on the correlation of Robertson and Campanella (1983). CPT analyses 1 and 3 indicated a clay layer approximately 2.7 to 3.7m (9 to 12 ft) thick just below the ground surface. This CPT method does not distinguish between high and low plasticity clay. Below the clay layer, the computerized CPT analysis for these soundings indicated alternating layers of silty sand, sandy silt, and clayey silt. While not exactly correct, this automatic analysis was considered reasonable. However the analysis for CPT 2 presented a soil profile consisting of alternating layers of sand, clayey sand, gravely sand, and no silt.

A soil classification was also performed manually using correlations from Robertson and Campanella (1983). These profiles consisted of alternating layers of sand, silty sand, and sandy silt. These results from the automatic and manual analyses suggest the CPT correlations tend to see loess as a sand, and occasionally as a gravel; therefore confirmation of soil classification through sampling is recommended for areas with loess.

CONSOLIDATION, COLLAPSE, AND UNCONFINED COMPRESSION TESTING

Consolidation testing was conducted on soil samples at intervals of approximately 3 meters (10 ft). Results show the soil behaves as a lightly overconsolidated soil with a decreasing OCR with depth as shown in Table 3.

Collapse testing was also conducted and the soil was determined to have a only a slight degree of collapse, which is consistent with reported results for soils with a unit weight greater than 14.1 kN/m³ (90 pcf) (Paliwal et al., 1965).

Unconfined compression tests were conducted on 71 mm (2.8 in) specimens trimmed from 89 mm (3.5 in) Shelby tube samples. Results are reported in Table 4. The loess was very stiff near the surface and medium stiff for depths below 1.6 meters. The modulus/shear strength ratio was calculated for each sample had an average value of 84. Modulus values determined using unconfined compression were the lowest of all test methods.

[TABLE 3] Preconsolidation Stress and Collapse Index

Depth (m)	φ_p' (kPa)	OCR
2.1	103	3.1
4.6	241	3.4
7.6	152	1.3

Depth (m)	I_e (%)	Degree of Collapse
0.3	0.5	slight
7.6	0.4	slight

TRIAXIAL AND DIRECT SHEAR TESTING

Both UU and CU triaxial tests were conducted on the loess throughout the soil profile. UU tests were conducted at in-situ moisture contents because the loessal soil is not expected to become saturated during the engineering life of the structure and it was considered likely that saturating the samples

[TABLE 4] Unconfined Compressive Strength Results, 2004

Depth (m)	Q_u (kPa)	E_m (kPa)	Consistency	Lab	S_u (kPa)	E_s/S_u
0.8	130	4830	very stiff	KDOT	65	74
1.6	30	1820	medium stiff	KDOT	15	121
3.3	24	820	medium stiff	KDOT	12	68
5.3	28	1020	medium stiff	KDOT	14	73
					Average	84

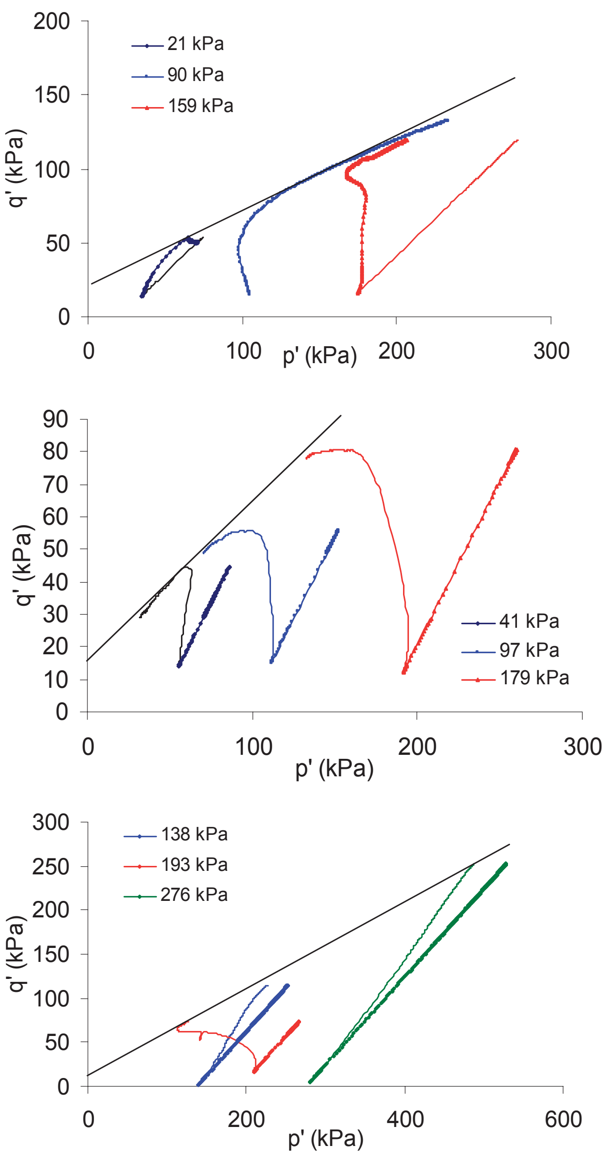
[TABLE 5] Triaxial Compression Results

2004 Data					
Depth (m)	**c (kPa)**	**ϕ (degrees)**	**E_m (kPa)**	**Test**	**Lab**
0.3	34	18	9690	UU	KU
0.9	21	20	13000	CU	KDOT
1.5	31	25	5950	UU	KU
2.1	12	26	29500	CU	KDOT
4.6	24	30	7800	UU	KU
7.6	10	30	8150	CU	KDOT

2005 Data					
Depth (m)	**c (kPa)**	**ϕ (degrees)**	**E_m (kPa)**	**Test**	**Lab**
0.3	28	22	7290	UU	KU
1.5	10	23	7180	UU	KU
4.6	0	32	10900	UU	KU

UU tests were conducted at natural water contents (unsaturated) during testing would damage or destroy the interparticle cementation, rendering the samples unrepresentative of field conditions. The CU tests were saturated and pore pressure was recorded during CU testing. Triaxial testing results are shown in Table 5.

Stress paths were plotted for all triaxial compression tests performed. Fig. 5 contains the stress paths of the three consolidated-undrained tests performed on representative

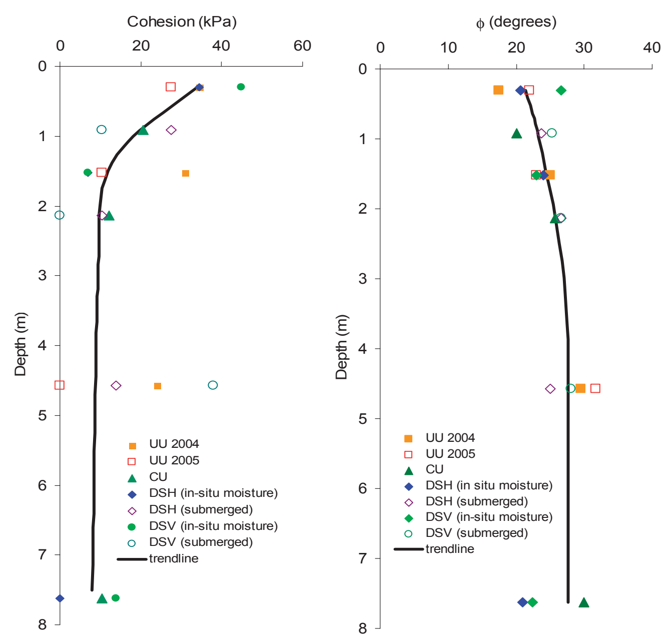


[FIG. 5] Stress paths for CU tests on loess samples from depths of 0.9, 2.1, and 7.6 meters. Confining stresses are shown.

samples taken at depths of 0.9, 2.1, and 7.6 meters (3, 7 and 25 ft). All samples were consolidated under isotropic conditions. These figures show the soil behaving as a contractive material throughout the soil profile, as would be expected as the clay cementing agent softens and yields and the soil grains shift to a denser configuration. Higher pore pressures were generated during loading for the samples of the more weathered soil from the shallow depths (0.9, 2.1m) while two of the three samples of the unweathered loess contracted only moderately prior to failure.

Direct shear tests were conducted on samples throughout the soil profile under submerged and in-situ moisture conditions. Table 6 shows the results for representative samples from a range of depths. Results showed cohesion to be a more significant factor near the surface that decreased in importance with depth. Submerged samples were only slightly less cohesive than soils tested at in-situ moisture conditions. The friction angle generally increased with depth from the low 20's in the more weathered material to approximately 27 degrees for the unweathered loess.

Triaxial and direct shear testing results are plotted with depth in Fig. 6 and show a consistent increase in friction angle with depth and decrease in cohesion (The one high value for cohesion at 4.6m (15 ft) is considered an outlier when compared with the other direct shear test and the two UU tests at that depth as well as the data trends for the other types of tests), which is consistent with the soil profile showing a more weathered, clayey soil with a higher water content near the surface that transitions to a silty soil. UU results on unsaturated samples had only slightly higher values for cohesion and lower values for friction angle than saturated CU tests. Cohesion did not



[FIG. 6] Laboratory cohesion and friction angle results for the soil profile

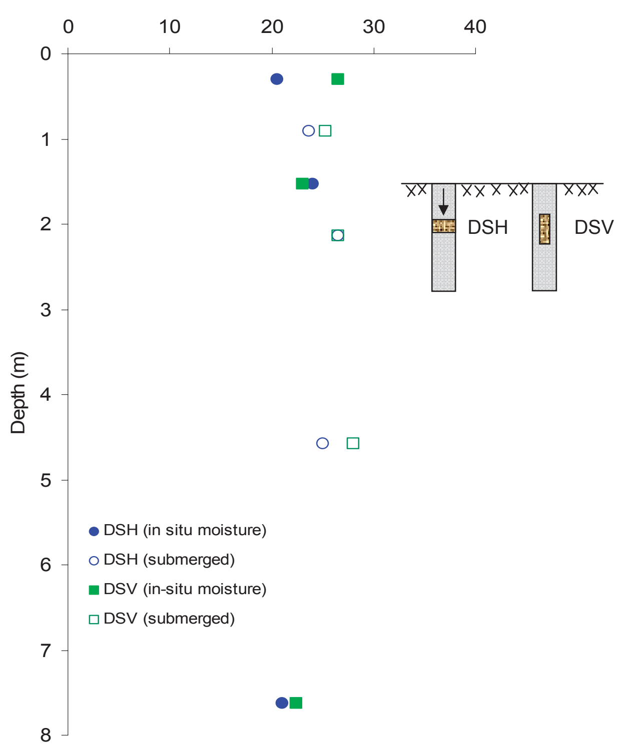
[TABLE 6] Direct Shear Results, 2004

Depth (m)	c (kPa) vertical shear	c (kPa) horizontal shear	ϕ (degrees) vertical shear	ϕ (degrees) horizontal shear	moisture conditions	lab
0.3	44.8	34.5	27	21	in-situ	KU
1.5	6.9	6.9	23	24	in-situ	KU
7.6	13.8	0.0	22	21	in-situ	KU
0.9	10.3	27.6	25	24	submerged	KDOT
2.1	0.0	10.3	27	27	submerged	KDOT
4.6	37.9	13.8	28	25	submerged	KDOT

revert to zero under effective stress conditions, confirming that cohesion for this soil was not solely an artifact of negative pore pressures.

INVESTIGATION OF ANISOTROPIC STRENGTH PROPERTIES

Loess has certain properties that are anisotropic. To evaluate if anisotropy exists with regard to strength parameters, select direct shear samples were trimmed from the 89 mm (3.5 in) Shelby tube samples with the vertical axis of the sample parallel to the long axis of the sampling tube and others with the vertical axis of the sample perpendicular to the sampling tube. Values are plotted with depth in Fig. 7 and show that this loess is effectively isotropic with regard to strength parameters.



[FIG. 7] Friction angle with shear plane orientation and moisture.

PRESSUREMETER TEST

Three pressuremeter tests were conducted by KDOT, two in 2004 and one during the week of the full scale load test in 2005. Tests were performed at depths of 0.6, 1.5, and 3 meters (2, 5, and 10 ft) below the ground surface. The following two equations from Terzaghi et al. (1996) were used to determine the elastic modulus (E_m):

$E_{pm} = 2(1+\nu)(V_0+v_m)(\Delta p/\Delta v)$ .... (4.1)

Where $v_m = (v_o + v_f) / 2$

E_{pm} = pressuremeter elastic modulus

ν = Poisson's ratio = 0.33

V_0 = initial volume of pressuremeter cell

Δp = change in pressure corresponding to Δv

Δv = change in volume corresponding to Δp

$E_m = E_{pm} / \alpha$ ....(4.2)

Where $\alpha = 0.5$ (value from Terzaghi et al. for normally consolidated silt)

Pressuremeter moduli were the highest for any test type for unsaturated soil. Table 7 lists the values recorded.

[TABLE 7] Pressuremeter Results

Depth (m)	E_m (ksf) 2004	E_m (ksf) 2005	E_m (kPa) 2004	E_m (kPa) 2005	K_o 2004	K_o 2005
0.6	276	208	13200	9940	2.6	3.1
1.5	159	446	7620	21400	1.3	1.5
3.0	225	240	10800	11500	0.7	1

CPT AND SPT TESTING

KDOT performed three CPT tests; one in 2004 and two during the week of the lateral load test. The software used to collect and analyze the field data was CPT-LOG Ver. 2.15a and CPT-pro Ver. 5.22, respectively. Fig. 2 shows the location of the CPT tests in relation to the testing shafts.

The correlation by Schmertmann (1975) was used to determine the elastic modulus and the correlations by Kulhawy and Mayne (1991) and Robertson and Campanella (1983) was used to determine the effective friction angle of the soil. The equations are as follows:

$E_m = 2 * q_c$ (ksf) ……………………………………… (4.3)

$\phi' = \tan^{-1}(0.1 + 0.38 * \log(q_c / \sigma_z'))$ ………….. (4.4)

Friction angles for depths with similar tip and sleeve values are reported in Table 8. Pore pressure values were recorded but were essentially zero and neglected in the analysis.

KDOT performed 14 SPT tests in 2004 using a 197 mm (7-¾ in) hollow-stem auger and an automatic hammer with an efficiency of 90%. Values for N_{60} and $N1_{(60)}$ were calculated and used to calculate soil modulus and friction angles using the correlations by Schmertmann (1975), Wolff (1989), Peck (1974), and Kulhawy and Mayne (1991) reported in Table 9 to evaluate the applicability of those correlations in a coarse grained soil with true cohesion.

Friction angle values for SPT and CPT testing are reported with the trendline of values from laboratory testing in Fig. 8. For reasons of clarity and given the similarity of values, only the CPT results calculated using the Robertson and Campanella values are shown in Fig. 8. This figure shows that friction angle values calculated using existing correlations for CPT and SPT were consistently higher, and in most cases much higher, than those determined in laboratory testing. CPT values were generally consistent with each other except for one of the 2005 soundings for depths between 2 and 4 meters (6 and 13 ft). This deviation may have been a function of weathering or moisture content variations at shallow depths. SPT values were consistent with depth throughout the soil profile, however estimated friction angles varied widely based on the correlation used. Friction angles using the correlation by Peck (1974) were the most consistent with the lab data. Reported values for Peck are slightly lower than Peck probably intended (1 - 3°) because the correlation uses an uncorrected N value that has not been adjusted for the higher energy input from the automatic hammer.

[TABLE 8] Selected CPT Data

CPT 2004 Depth (m)	q_t (kPa)	f (kPa)	E_m (kPa)	ϕ' (R & C)	ϕ' (K & M)
1.3	2394	120	4788	42	37
4.8	4309	43	8618	38	36
7.2	7661	108	15322	39	38
8.3	7661	108	15322	38	38
9.6	6703	156	13406	36	37
11.1	8379	156	2394	37	37
12.4	9576	156	19152	37	38

CPT 2005 Depth (m)	q_t (kPa)	f (kPa)	E_m (kPa)	ϕ' (R & C)	ϕ' (K & M)
0.4	1197	86	2394	44	36
1.8	958	57	1915	34	31
2.9	1915	53	3830	36	34
4.0	2873	38	5746	36	35
5.3	4549	53	9097	37	36
6.2	4309	53	8618	36	36
6.9	4549	57	9097	36	36
7.7	4788	60	9576	35	36
8.4	5985	60	11970	36	36
9.6	5698	77	11395	35	36

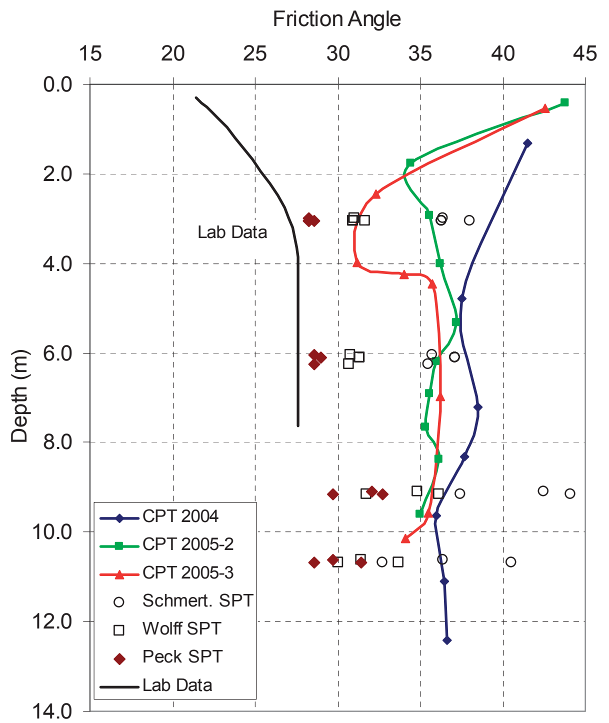
CPT 2005 Depth (m)	q_t (kPa)	f (kPa)	E_m (kPa)	ϕ' (R & C)	ϕ' (K & M)
0.5	1197	77	2394	43	35
2.4	958	48	1915	32	31
4.0	1341	50	2681	31	31
4.2	2155	57	4309	34	33
4.5	2969	48	5937	36	35
7.0	5027	55	10055	36	36
9.6	6224	65	12449	36	36
10.1	5267	67	10534	34	35

[TABLE 9] SPT Correlations for sand

Author	Equation
Schmertmann (1975)	$\emptyset' = \tan^{-1}\left[\left[\frac{N_{60}}{12.2 + 20.3\frac{\sigma_z'}{p_a}}\right]^{0.34}\right]$
Wolff (1986)	$\emptyset' = 27.1 + 0.3(N_1)_{60} - 0.00054[(N_1)_{60}]^2$
Peck et al. (1974)	$\emptyset' = 53.881 - 27.6034e^{-0.0147N}$
Kulhawy and Mayne (1991)	$E_m = 5N_{60} * 100\text{kPa}$

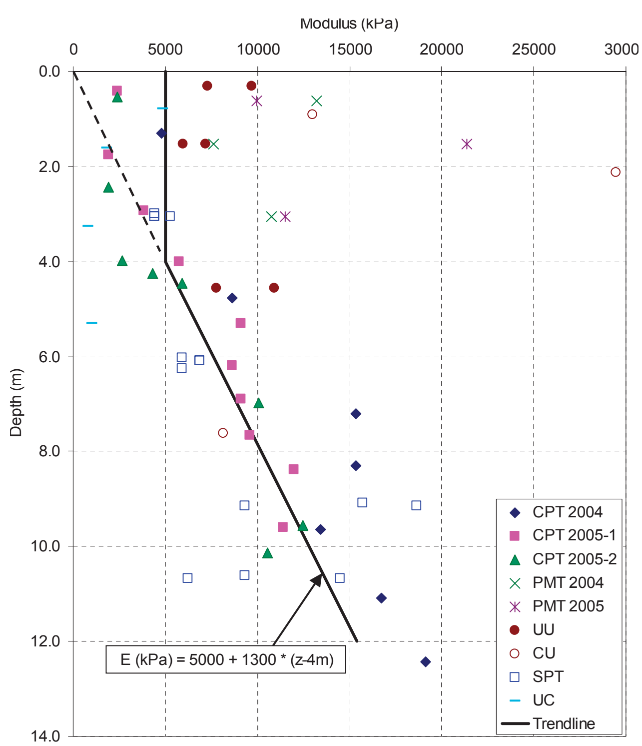
ELASTIC MODULUS

Fig. 9 presents in-situ and laboratory elastic modulus values. The values from the consolidated-undrained triaxial test were relatively high and pressuremeter values



[FIG.8] Friction angle from CPT and SPT compared with lab data

were very high when compared with the other tests. Elastic moduli from the unconfined compression tests were consistently low, which is consistent with the expectation that moduli will be lower for soil samples with no confinement and large strains. Elastic moduli computed from CPT test results test had limited variability and reflected an intermediate value



[FIG. 9] Elastic modulus values and trend from in-situ and laboratory results

between laboratory and SPT correlated values. The greater consistency of the CPT data made it the most promising data set for p-y curve development.

The unweathered loess at depth had a modulus trend that increased linearly with depth. Modulus values for the more weathered soil near the surface (depth <4 m or 13 m) were widely scattered and did not appear to be related to depth as shown in Fig. 9.

CONCLUSIONS

A detailed site investigation of the loess deposit at the Kansas geotechnical test site was made as part of a full scale investigation of the lateral load bearing capacity of loess. A series of conclusions were developed based on this investigation.

- All tests results and soil properties analyzed indicate loess at the test site behaved as a frictional soil with some true cohesion.
- The friction angle varied from approximately 22 degrees at the surface in the more weathered, clayey soil to approximately 27 degrees at depths of 8 meters (26 ft).
- Soil cohesion was at a maximum near the surface and decreased with depth to approximately 8-10 kPa for depths greater than 2 meters (6.6 ft).
- Moisture conditions had little effect on the soil's shear strength and collapse was not a concern for this soil.
- The loess was essentially isotropic with regard to strength parameters.
- The soil's elastic modulus increased linearly with depth below the more weathered surface soils.
- CPT moduli appeared to have the most promise for use in p-y curve modeling because the data had limited variability and moduli values were in the middle of the range of measured values.
- Automatic soil classification methods using the CPT should be used with caution because of the possibility of inaccurate interpretation of soil type.
- Commonly used correlations for SPT and CPT significantly overestimated friction angle, presumably because strength contributed by cohesion was interpreted as a higher friction angle.

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