CONCLUSIONS

To incorporate structural variations into rate-process theory, a primitive or ideal clay-water system is defined by a linear strain-time relation implicit in the theory. The deviations of the real-clay strain-time curve from ideal behavior are attributed to the structural changes that accompany the deformation of real clay. To quantify the structural variations a strain ratio, called the mobilization ratio, is defined. When creep curves of clay specimens at various shear stresses or temperatures are compared, the structures of the specimens are identical at equivalent mobilization ratios.

The method of analysis reveals that the rate-process parameters (β), (ΔH^*), and (A) are structure dependent. Because these parameters vary with creep deformations they can be treated as structural parameters to characterize the structural variations. The test results indicate that strain hardening is associated with a rearrangement of particles that tends toward a more ordered system and with a tendency in the primary particles to be welded into larger particles with stronger bonds. The structure attains its coarsest texture and most strongly bonded and most orderly particle association at the inflection point of the creep curves, as indicated by maximum values of β and ΔH^* and minimum value of A.

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Sampling a Glacial Silty Clay

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Over a period of 10 years a total of 42 borings were made in a glaciolacustrine deposit on the western boundary of Detroit, Michigan. The soil profile consisted of yellow-brown mottled silty clay underlain by gray silty clay. The undrained shear strength, moisture content, and dry density of 329 soil specimens of the gray silty clay are statistically analyzed. The variables are randomly distributed in both the vertical and the lateral directions. The lognormal distribution is the most likely fit for the data but, on engineering grounds, the normal distribution is preferred. Statistical estimation theory indicates that as few as 5 borings arranged in an X pattern and containing at least 30 specimens could adequately estimate the values for design at this site.

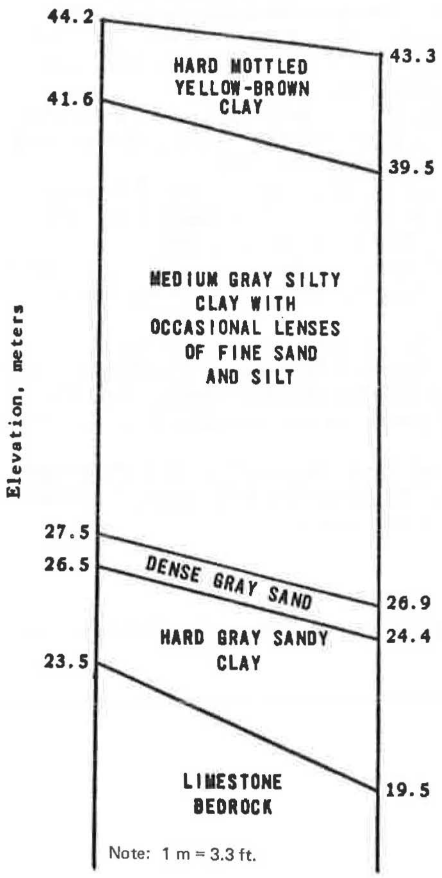
Any program to determine the properties of a natural soil deposit requires answering the following questions:

1. How many borings should be made?
2. Where should the borings be located?
3. How many field tests or laboratory tests or both should be performed?

Once these questions are answered, usually somewhat arbitrarily, it is still necessary to adopt a method for calculating the value of the design parameter. For example, a procedure that has been recommended for analysis of bearing capacity in clays is to take several borings in the area of the footings, average the values in each boring within the significant depth, and take the minimum boring average divided by the factor of safety for design (4). Although this procedure is generally effective, the actual factor of safety is not known. Because the various properties of natural soil deposits behave like random variables, their variations can be analyzed by statistical methods. Such methods make it possible to answer the above questions systematically.

The most important objection to the use of statistical methods of calculation is that they require obtaining more data than are generally considered economical on most soils engineering projects. Examining this need for excessive amounts of data reveals that it is generated by requirements for selecting a suitable probability distribution function to model the distribution of values of random variables, e.g., the undrained shear strength. There may be a way out of this dilemma. Experience and published data suggest that soil deposits that result from similar geological processes and that have similar composition have similar engineering properties. It seems reasonable then to hypothesize that corresponding properties of such deposits might be adequately modeled by the same probability distribution function. In that case, it becomes a matter of identifying the soil type in each stratum and taking sufficient samples to estimate the distribution parameters to the desired confidence level.

Figure 1. Composite soil profile.



SOIL DATA

The soil deposit studied here is a water-worked glacial silty clay formed by the Erie-Huron ice lobe during the glaciation of the Great Lakes. The surface features were formed during the Wisconsin stage of the Pleistocene glaciation (3).

The design of an interchange at the Jeffries and Southfield freeways on the west side of Detroit was changed several times over a 10-year period. Each time the design was changed, additional subsurface information was obtained. A total of 42 borings were made in a 59 458-m^2 (14-acre) area. The main structures were finally supported on point bearing piles to rock.

Figure 1 shows a composite profile of the soil in which the borings were made. The soil is composed of an upper layer of hard yellow-brown mottled clay with a mean thickness of 4.27 m (14 ft) and a standard deviation of 2.13 m (7 ft) underlain by a layer of medium gray silty clay with a mean thickness of 10.67 m (35 ft) and a standard deviation of 2.44 m (8 ft). The next layers are a dense gray sand, a hard gray sandy clay, and then limestone bedrock. All borings and laboratory tests were made by personnel of the Michigan Department of State Highways and Transportation in accordance with procedures described in the department's Field Manual of Soil Engineering (2).

TEST PROCEDURES AND RESULTS

Soil samples were obtained by pressing the sampler hydraulically into the soil or by levering; the sampler was never driven. The sampler was a 76-cm-long (25.8-in) steel tube equipped with a cutting tip that had a 4.44-cm (1.5-in) outside diameter and was fitted with a series of liners that took a soil cylinder of 3.49-cm (1.2-in) diameter and 25.4-cm (8.6-in) length. The

Table 1. Results of soil classification test.

		Sieve		Sample 1		Sample 2		Sample 3	
Test	AASHTO Soil Classification	Size	Opening (mm)	Cumulative Percent Passing	Percent Retained	Cumulative Percent Passing	Percent Retained	Cumulative Percent Passing	Percent Retained
Sieve analysis	Gravel	3/4 in.	19.1						
		1/2 in.	12.7						
		3/8 in.	9.52						
		No. 4	4.76						
		No. 8	2.38						
		No. 10	2	100		100		100	
	Coarse sand	No. 18	1	97		97		98	
		No. 20	0.84	96		96		97	
		No. 35	0.5	94		94		95	
		No. 40	0.425	94	6	93	7	95	5
	Fine sand	No. 50	0.297						
		No. 60	0.25	91		90		92	
		No. 100	0.149						
		No. 140	0.105	84		82		86	
		No. 200	0.075	82	12	80	13	84	11
Hydrometer	Silt		0.05	78		74		79	
			0.005	46	36	43	37	49	35
	Clay		0.001		46		43		49

Note: 1 mm = 0.039 in.

liners were taken to the laboratory where the samples were tested for unconfined compression strength, moisture content, and natural unit weight in accordance with recommended AASHO standards. Gradation analyses, Atterberg limits, and specific gravity tests were also performed on three of the specimens (Table 1). The following table gives the resulting soil constants for three samples.

Soil Constant	Sample 1	2	3
Liquid limit, %	28	29	32
Plasticity index, %	11	13	14
Specific gravity	2.68	2.68	2.67
Shrinkage limit, % by weight	15	15.2	15.4
Shrinkage ratio	1.90	1.9	1.88

A total of 329 specimens of the gray silty clay were obtained from the 42 borings and tested. The results are presented in Table 2 in terms of

$$\bar{x} = 1/N\sum_{i=1}^{N} x_i \qquad (1)$$

Table 2. Statistical properties of gray silty clay.

Statistic	Undrained Shear Strength[a] (kPa)	Unit Weight of Dry Soil[b] (g/cm^3)	Moisture Content[c] (% dry weight)
Median value	36.38	1.679	22
Mean value	38.64	1.668	22.7
Standard deviation	15.79	0.120	4.7

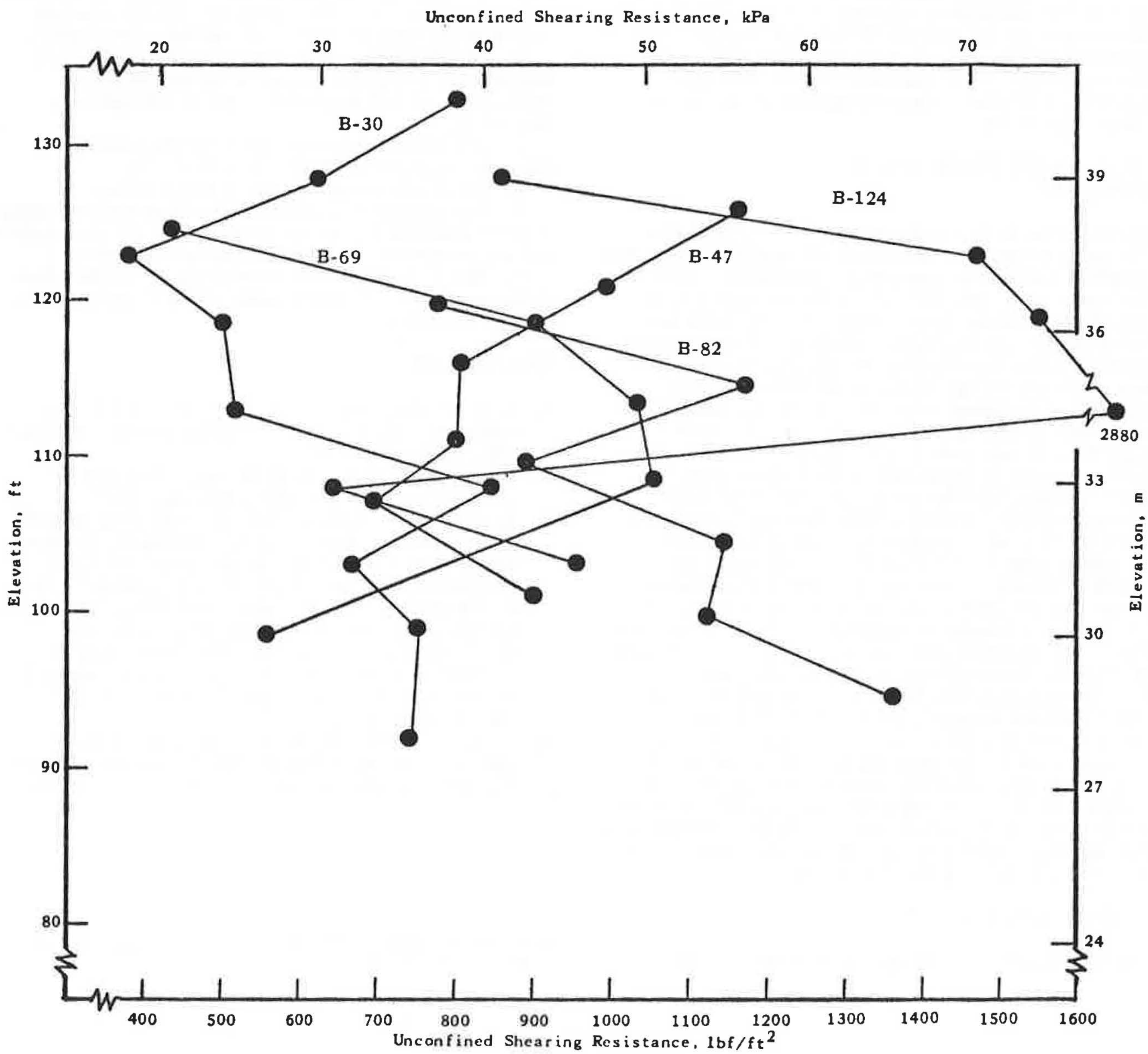
Note: 1 kPa = 20 lb/ft^2; 1 g/cm^3 = 0.036 lb/in^3.

[a] Coefficient of variation is 0.408.

[b] Coefficient of variation is 0.072.

[c] Coefficient of variation is 0.207.

Figure 2. Strength profile of gray silty clay layer.



$$S = \left[1/(N-1) \sum_{i=1}^{N} (x_i - \bar{x})^2 \right]^{1/2} \quad (2)$$

$$V = S/\bar{x} \quad (3)$$

where

$\bar{x}$ = sample mean value,
S = sample standard deviation,
V = sample coefficient of variation,
N = number of specimens, and
x_i = ith value of the random variable.

The median value is x_i, for which 50 percent of the values are larger (or smaller).

Examination of the values in Table 2 reveals that the coefficient of variation for the undrained shear strength of this glacial silty clay is 0.408. This agrees with previously published data on similar glacial silty clay strata from the same geological region, which recommend the value of 0.40 (6).

Linear regression analyses in which elevation was used as the independent variable were performed to determine whether vertical or horizontal spatial correlations existed. The largest correlation coefficient found was 0.20, which indicated that no such useful correlation existed. The strength profile shown in Figure 2 is typical.

PROBABILITY DISTRIBUTION FUNCTIONS

Undrained shear strength and moisture content were plotted on normal and lognormal probability paper. For moisture content the lognormal distribution is clearly the best model. The results for the undrained shear strength are not as clear because the lognormal distribution best fits the higher strength tail but the lower strength tail is best fitted by the normal density function. The chi-square goodness-of-fit test (1) was applied to the strength data for both the normal and the lognormal distribution models for cases in which (a) all data were grouped into 20 equally likely intervals, (b) the 10 percent of the data of the highest strength were "lumped," and (c) the 2.5 percent of the data having the highest strength were truncated. For each of the three cases considered, a pair of chi-square statistics were obtained and the best fit was determined by the value most likely to occur in a random process described by the chi-square distribution. On this basis the lognormal statistic is closer to the mean than is the Gaussian statistic in every case except that in which the lower 97.5 percent of the data are used. This is consistent with the work of Wu and Kraft (5). The lognormal is clearly the more probable fit of the two if all the data are used, and it would be the one selected if the choice were based only on statistical considerations. However, because the Gaussian distribution fits the lower strength tail very well and this tail is more critical to prediction of failure probabilities than the higher strength one, the Gaussian statistic would be a better empirical selection.

BORINGS AND SPECIMENS

In exploring a site of the size used in this research, it is typical practice to begin with five borings arranged in an X pattern. This allows the construction of two intersecting soil profiles that indicate the gross characteristics of the soil stratigraphy. Extensive analysis of the data show that, if the five borings contain a minimum of 30 specimens, that number is sufficient to calculate a value of strength for use in design. It was found that the strength corresponding to the lower bound of the 99 percent confidence interval on the mean is conservative, but not unduly so, and it is consistent with that produced by the ad hoc procedures currently used by experienced geotechnical engineers. The analysis also shows that it is not conservative to use the average specimen strength for the design value. Using the minimum of the individual boring averages, however, is too conservative.

CONCLUSIONS

For a gray, water-worked, glacial silty clay of low plasticity,

1. On the basis of the most probable chi-square statistic, the lognormal distribution function models undrained shear strength, moisture content, and unit weight better than the normal (Gaussian) distribution;
2. On the basis of engineering needs, the normal distribution is a better model for undrained shear strength because it predicts the low strengths more accurately;
3. The coefficient of variation for the undrained shear strength, for 329 specimens is 0.408;
4. There are no useful spatial correlations;
5. Five borings arranged in an X pattern and containing at least a total of 30 specimens are sufficient to calculate a design value for the undrained shear strength; and
6. The best design undrained shear strength is that corresponding to the lower bound of the 99 percent confidence interval.

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