SHEAR STRENGTH INCREASE IN A SOFT FOUNDATION CLAY

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The increase in shear strength in soft clay (bay mud) due to consolidation under highway embankment loading is presented. Vertical sand drains were used to accelerate the process of consolidation. The gain in shear strength was measured over a 4-year period after construction of the embankment. It is shown that the excess pore water pressure measured in the field can be used to predict the increase in shear strength with reasonable accuracy.

•IN COASTAL REGIONS it is often necessary to construct highway embankments over marsh or swamp deposits. Because the natural shear strength of such deposits is usually too low to provide adequate stability against shear failure under normal construction schedules, the rate of the placement of embankment fill must be controlled to meet the following two requirements simultaneously:

1. The shear stresses induced by the embankment load must be smaller than the available shear strength so that no large-scale shear rupture in the foundation soil will occur, and
2. Consolidation of the foundation soil must result in sufficient gain in shear strength to provide adequate stability under subsequent embankment loading.

Therefore, the in situ natural shear strength of the foundation soil must be evaluated. The amount of gain in shear strength as consolidation progresses must be related to some measurable quantity in the field during construction. The design rate of embankment loading should be adjusted based on actual settlement and pore pressure dissipation during construction. In this paper, an embankment constructed over soft, peaty, silty clay (locally known as bay mud) extending from ground surface to a depth of 50 to 70 ft is described. The site is an approach embankment at the west end of the Napa River Bridge near Vallejo, California, on Cal-37 (Fig. 1).

Vertical sand drains were used to accelerate consolidation. The placing of sand drains began in April 1960. Approximately 2,500 sand drains varying in depth from 42 to 72 ft were driven. An 18-in. hollow mandrel with a closed bottom was used. After the mandrel had been driven and the sand placed inside, compressed air was applied at the top, forcing the sand out of the hinged bottom as the steel mandrel was withdrawn. When the drains were completed, a 2- to 3-ft blanket of filter material was distributed over the working platform. Free drainage of water from the permeable blanket was aided by placement of an 8-in. perforated metal pipe along the centerline. The details of the sand drain installation for this project were reported by Weber (4). The embankment fill was 22 ft high after the first stage of construction. The construction was then halted because of a failure in the foundation soil. Second-stage construction began approximately 17 months later. The partial plan and typical cross section of the embankment are shown in Figure 2. The shear strength in the soft clay was determined just prior to construction and for 4 years after construction of the embankment.

FIELD AND LABORATORY TEST DATA

The project was instrumented primarily for a study of the effect of sand drains in soft foundation soils. Detailed instrumentation and measured data have been described

Figure 1. Project site.

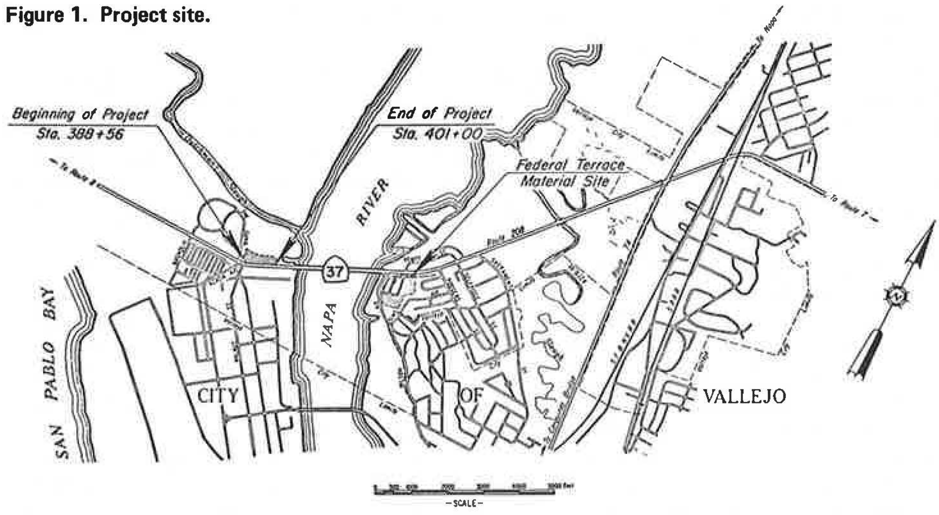
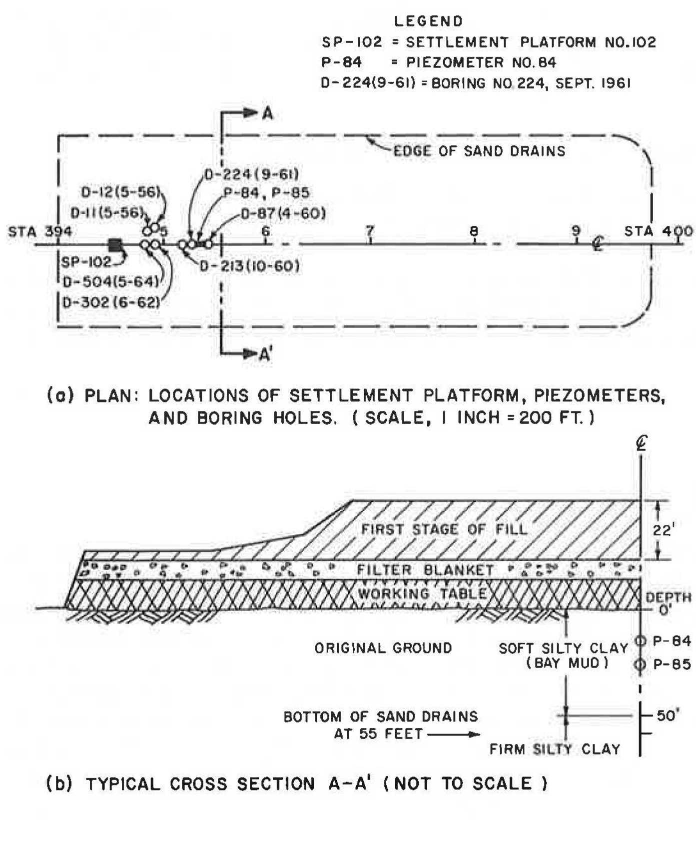


Figure 2. Partial plan and cross section of test site.



by Weber (4) and Smith, Weber, and Shirley (3). Only part of the instrumentation and their measurements will be discussed here.

The subsurface profile consists of a relatively homogeneous, gray bay mud combined with broken seashells to a depth of 50 to 70 ft. Some layers of sandy silt and peat were found between 50 to 60 ft. A firm silty clay underlies the soft bay mud. The natural water contents average more than 90 percent, and the in-place shear strength determined by vane borer varies from about 100 psf at a depth of 10 ft to about 500 psf at a depth of 40 ft. The soft bay mud is slow draining and has a permeability of approximately 10^{-5} ft per hour as determined by an in situ permeability method described by Weber (5). Without special treatment, this soil will support fills approximately 6 ft high. The average soil properties are given in Table 1.

As shown in Figure 2, borings were made for determination of shear strength in foundation clay prior to the construction of the embankment (D-11, D-12, and D-87) and immediately after the completion of the first-stage embankment (D-231). Additional borings were made approximately 1 year (D-224), 2 years (D-302), and 4 years (D-504) after the completion of the first-stage embankment fill.

Laboratory shear strength was obtained from laboratory tests on undisturbed samples taken from these boring holes by using the unconfined compression test (U-test) and the unconsolidated-undrained triaxial compression test (UU-test). The samples were obtained by either pushing a 2-in. California sampler hydraulically into the bay mud or pushing a Swedish foil sampler into clay layers. The results of these tests are shown in Figure 3, which clearly displays the increase in shear strength with time due to consolidation. Settlement platforms were installed at various parts of the embankment. The time-settlement relationship at settlement platform SP-102 is shown in Figure 4. Piezometers were also installed in the foundation clay at various depths for measurement of excess pore piezometers: P-84 at elevation -15 ft and P-85 at elevation -24 ft are shown in the lower part of Figure 4 (see Fig. 2 for the locations of these instrumentations).

ANALYSIS OF SHEAR STRENGTH INCREASE

The analysis of increase in shear strength in soft clay material involves the evaluation of its natural strength and the increase in shear strength during the consolidation process.

The bay mud at the project site is generally normally consolidated. This is verified by the shearing strength-elevation relationship shown in Figure 3. Therefore, the value of c' in the Mohr-Coulomb equation is zero, and the shear strength may be written as

$$\bar{s} = \bar{p} \tan \phi' \tag{1}$$

where

$\bar{p}$ = effective normal stress and
ϕ' = angle of internal friction in terms of effective stress.

From the shear strength data tested on samples taken from D-11, D-12, and D-87, the value of $\bar{s}/\bar{p}$ is found to be equal to 0.21, or $\phi' = 12$ deg. Assuming that ϕ' is constant both before and after embankment loading, the increase in shear strength in foundation clay would be directly proportional to the increase in $\bar{p}$. In this article the increases in $\bar{p}$ were calculated based on field measurements of excess pore pressure and settlements. Skempton's pore pressure parameter A was evaluated based on the method proposed by Lambe (3). Similar samples of bay mud were used to find the A value of 0.8. Based on this value, the initial excess pore pressure u_i due to a load imposed by 22 ft of embankment fill (unit weight of 125 pcf) is equal to 1.10 kg/cm^2. The degree of consolidation based on the measured excess pore pressure may be calculated with the following equation.

$$U = 1 - \frac{u}{u_i} \tag{2}$$

Table 1. Summary of soil properties.

	Depth (ft)		
Property	0 to 6[a]	6 to 26[b]	26 to 46[c]
Natural density, pcf	85-90	90-95	95-105
Natural water content, percent	95-120	80-95	68-76
Liquid limit, percent	90-110	70-95	60-75
Plasticity index, percent	45-60	35-50	22-42
Percentage finer than No. 200		55-60	35-50
Specific gravity		2.70	
Shear strength from CU test (TSF)	0.05-0.07	0.09-0.14	0.30-0.45
Void ratio		2.5-3.5	
Compression index		0.75-0.9	
Permeability, ft/hr		3.6×10^{-5}	
Coefficient of consolidation, ft^2/day		0.05-0.22	

[a]Top soil, silt-clay with extensive organic matter and seashells.
[b]Very soft silty clay with varying amounts of peat.
[c]Soft to firm silty clay with trace of seashells and peat.

Figure 3. Determinations of shearing strength.

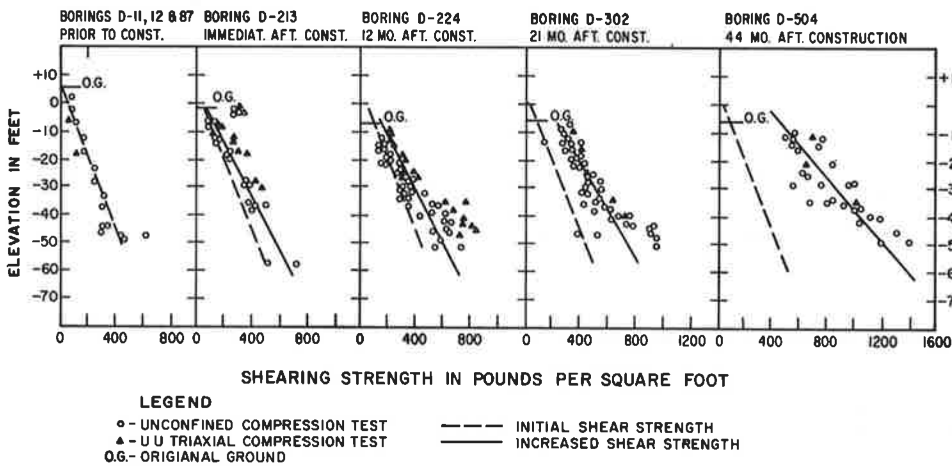


Figure 4. Time-settlement relationship and pore pressures.

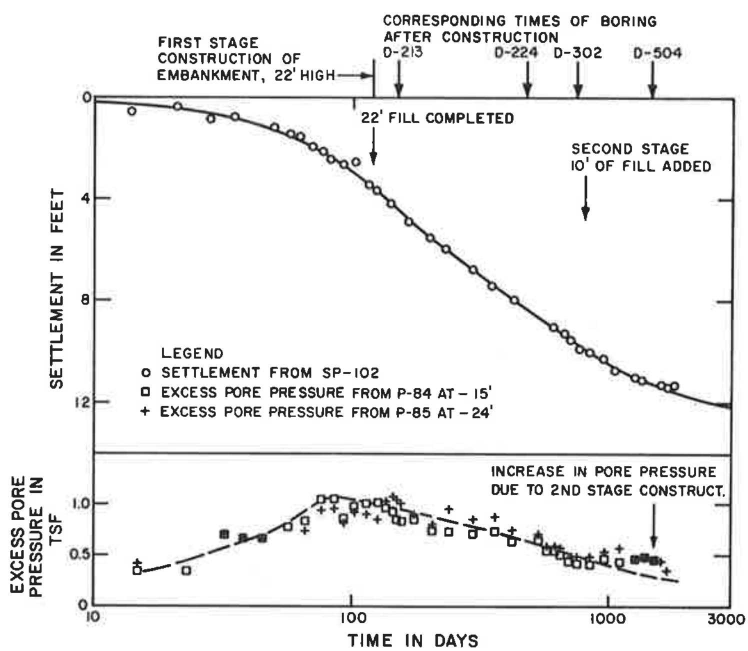


Table 2. Calculation of shear strength increase from field pore pressure measurements: $\bar{s}/\bar{p} = 0.21$, $\bar{p} = 2{,}750U + \bar{p}_i$, $U = 1 - (u/u_i)$.

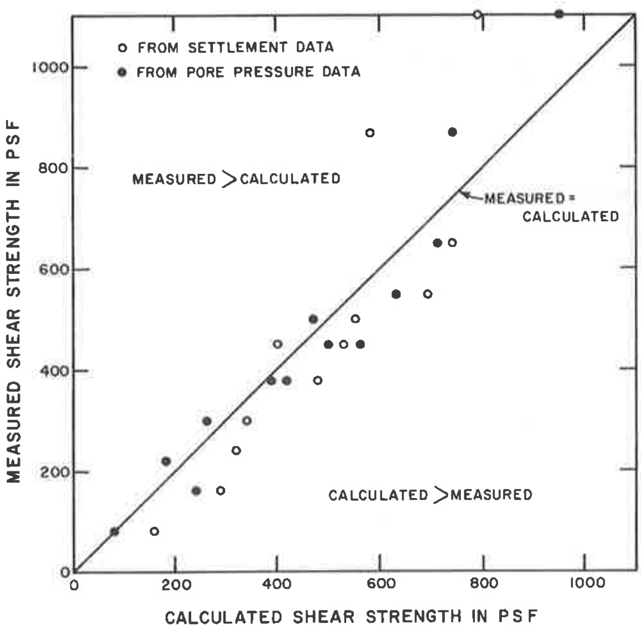
Elapsed Time (days)	Excess Pore Pressure (ksf)	U (percent)	Boring Number	Effective Vertical Stress (psf)			Calculated Shear Strength (psf)			Measured Shear Strength (psf)		
				Ground	20 Ft	40 Ft	Ground	20 Ft	40 Ft	Ground	20 Ft	40 Ft
0	1.10	0	D-11, 12, and 87	0	860	1,860	0	180	390	0	220	380
150	0.95	14	D-213	385	1,245	2,245	80	260	470	80	300	500
480	0.65	41	D-224	1,130	1,990	2,990	240	420	630	160	380	550
750	0.50	55	D-302	1,520	2,380	3,380	320	500	710	240	450	650
1,440	0.30[a]	73	D-504	2,650[b]	3,510	4,510	560	740	950	450	870	1,100

[a]The excess pore pressure due to the first stage of embankment load only. [b]Assuming 50 percent dissipation from second-stage loading.

Table 3. Calculation of shear strength increase from field pore pressure measurements: $\bar{s}/\bar{p} = 0.21$, $\bar{p} = 2{,}750U + \bar{p}_i$, ultimate settlement = 16 ft.

Elapsed Time (days)	Settlement (ft)	U (percent)	Boring Number	Effective Vertical Stress (psf)			Calculated Shear Strength (psf)			Measured Shear Strength (psf)		
				Ground	20 Ft	40 Ft	Ground	20 Ft	40 Ft	Ground	20 Ft	40 Ft
0	0	0	D-11, 12, and 87	0	860	1,860	0	180	390	0	220	380
150	4.4	28	D-213	760	1,620	2,620	160	340	550	80	300	500
480	8.3	51	D-224	1,400	2,260	3,260	290	480	690	160	380	550
750	9.8	60	D-302	1,650	2,510	3,510	320	530	740	240	450	650
1,440	11.2	69	D-504	1,900	2,760	3,760	400	580	790	450	870	1,100

Figure 5. Comparison of measured and calculated shear strength.



where u is the excess pore pressure and U is the degree of consolidation. Assuming that the effective normal stress $\bar{p}$ in Eq. 1 is equal to the effective overburden pressure, the value of $\bar{p}$, in psf, may be computed as follows:

$$\bar{p} = (22 \times 125 \times U) + \bar{p}_i \tag{3}$$

where $\bar{p}_i$ is the initial effective overburden pressure prior to embankment loading. By using these equations, the expected increase in shear strength at the original ground surface, at 20 ft, and at 40 ft was calculated and is given in Table 2 at a corresponding time when the borings were made. The measured shear strengths (Table 2) were taken from Figure 3, in which the corresponding depths and time are shown. Similar calculations for determining increase in shear strength by using time-settlement data to determine the percentage of consolidation were performed and are given in Table 3. The ultimate settlement in this calculation was assumed to be 16 ft. The effect of the second-stage embankment loading was ignored.

DISCUSSION OF RESULTS

The results given in Tables 2 and 3 were used to plot the calculated shear strength against the measured values (Fig. 5). It is seen that the plotted data closely follow the 45-deg line, which represents the common line of shear strength determined by both methods. It appears that measured shear strength is generally smaller than the calculated values. These differences may be attributed to some degree of disturbance of soil samples.

Figure 5 also shows that the calculated shear strengths based on excess pore pressure generally show a better agreement than those based on settlement data. Because the dissipation of excess pore pressure is considered as a direct measure of the transfer of load from pore water to soil skeleton, the dissipation of pore pressure data should give a more accurate estimate of the increase in effective stress. On the other hand, the time-settlement data usually include the settlements due to elastic deformation on embankment loading, to plastic flow, and to secondary compression. The settlements due to elastic deformation are immediate settlements without dissipation of excess pore pressure and without change in void ratio.

Highway embankments founded on soft clay are often designed with a marginal factor of safety so that some amount of plastic flow in the foundation soil may occur. The vertical settlement of ground surface due to elastic and plastic deformations may not contribute to any increase in shear strength. Because the secondary compression takes place at little or no dissipation in excess pore pressure, it is not known whether the volume change due to this secondary compression will result in any strength gain in foundation clay. A method to completely separate these components of settlements is not yet available; therefore, the estimate of shear strength in foundation clay should preferably be based on field pore pressure rather than settlement data. When time-settlement data are used to calculate the increase in shear strength, they should be complemented by field pore pressure measurements.

CONCLUSIONS

The following conclusions can be drawn from this study:

1. The increase in shear strength in soft foundation clay may be calculated with reasonable accuracy if field excess pore pressure and time-settlement data are available;
2. The excess pore pressure data generally yield a better estimate of strength gain and should be preferred to the use of time-settlement data; and
3. The assumption that $\bar{s}/\bar{p}$ is a constant in a normally consolidated clay appears reasonable in the prediction of shear strength increase.

ACKNOWLEDGMENTS

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