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Su, Hon-Hsieh; Chang, Jerry C.; and Forsyth, Raymond A.

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SHEAR STRENGTH INCREASE IN A SOFT FOUNDATION SOIL

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Presented at the 52nd Annual Meeting
of the Highway Research Board

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SHEAR STRENGTH INCREASE IN A SOFT FOUNDATION CLAY

Hon-Hsieh Su
Associate Professor of Civil Engineering
California State University, Sacramento

Jerry C. Chang
Associate Materials and Research Engineer

Raymond A. Forsyth
Supervising Materials and Research Engineer

Presented at the 52nd Annual Meeting
of the Highway Research Board
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Abstract: 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.

Key words: Shear strength; Soft clay; Excess pore pressure; Settlement; Sand drain; Highway embankment.

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SHEAR STRENGTH INCREASE IN A SOFT FOUNDATION CLAY

By

H. H. Su[1], J. C. Chang[2], and R. Forsyth[3]
Members, ASCE

INTRODUCTION

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

[1]Associate Professor of Civil Engineering, California State University, Sacramento, California.

[2]Associate Materials and Research Engineer, Materials and Research Department, Division of Highways, State of California.

[3]Assistant Materials and Research Engineer, Materials and Research Department, Division of Highways, State of California.

(a) 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.

(b) 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 is in progress 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 article, an embankment constructed over soft peaty silty clay (locally known as Bay mud) extending from ground surface to a depth of 50 to 70 feet is described. The site is an approach embankment at the west end of the Napa River Bridge, near Vallejo, California, on State Route 37 as shown in the location map, Figure 1. Vertical sand drains were used to accelerate the consolidation. The placing of sand drains, by the Raymond Concrete Pile Company, began in April of 1960. Approximately 2500 sand drains varying in depth from 42 to 72 feet were driven. An 18-inch hollow mandrel with a closed bottom was used. After the mandral 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-foot blanket of filter material was distributed over the working platform. To aid in free drainage of water from the permeable blanket, an 8-inch perforated metal pipe was placed along centerline. The details of the sand drain installation for this project were reported by Weber, W. G. (1966). The embankment fill was 22 feet high after the first stage of construction. The construction was then halted due to a failure in the foundation soil. The second stage construction began approximately 17 months later. The partial plan and typical section of the embankment are shown in Figure 2. The shear strength in the soft clay was determined just prior to construction and for four 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 were described elsewhere by Weber, W. G. (1966) and Smith, Weber and Shirley (1969). Only part of the instrumentation and their measurements will be discussed here. The subsurface profile consists of a relatively homogeneous soft gray peaty silty clay (Bay mud) combined with broken sea shells to a depth of 50 to 70 feet. Some layers of sandy silt and peat were found between 50 to 60 feet. A firm silty clay was encountered underlying the soft Bay mud. The natural water contents average above 90 percent and the in-place shear strength determined by vane borer varies from about 100 pounds per square foot at a depth of ten feet to about 500 pounds per square foot at a depth of 40 feet. The soft Bay mud is slow draining, having a permeability of approximately 10^{-5} feet per hour as determined by an in-situ permeability method described by Weber, W. G. (1968). Without special treatment this soil will support fills approximately six feet in height. The average soil properties are shown 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 at approximately one year (D-224), two years (D-302), and four years (D-504) after the completion of the first stage embankment fill (see Figure 2 for location of the bore holes).

Laboratory shear strength was obtained from laboratory tests on undisturbed samples taken from these boring holes using unconfined compression test (U Test) and unconsolidated-undrained triaxial compression test (UU Test). The samples were obtained by either pushing hydraulically a 2-inch California sampler into the Bay mud or by pushing a Swedish Foil sampler into clay layers. The results of these tests are presented in Figure 3 which shows clearly 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 different depths for measurement of excess pore piezometers, P-84 at elevation -15 feet, and P-85 at elevation -24 feet, are shown in the lower part of Figure 4 (see Figure 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 from 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,

$$\bar{s} = \bar{p} \tan \phi' \quad (1)$$

where $\bar{p}$ is the effective normal stress and ϕ' is the 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 $\phi' = 12$ degrees. Assuming that ϕ' is constant both before and after the 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. The Skempton's pore pressure parameter "A" was evaluated based on the method proposed by Labe, T. W. (1962), using similar samples of Bay Mud the "A" value was found to be 0.8. Based on this "A" value, the initial excess pore pressure, u_i, due to a load imposed by 22 feet 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 using the following equation.

$$U = 1 - \frac{u}{u_i} \quad (2)$$

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 $\bar{p}$ may be computed as follows:

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

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

DISCUSSION AND CONCLUSION

Using the results shown in Tables 2 and 3, the calculated shear strength was plotted against those measured (Figure 5). It is seen that the plotted data closely follow the 45 degree line which represents the common line of shear strength determined by both methods. It appears that the measured shear strength is generally smaller than those calculated. These differences may be attributed to some degree of disturbance of soil samples.

Figure 5 also indicates that the calculated shear strengths based on excess pore pressure generally show a better agreement than those based on settlement data. Since 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 upon embankment loading, the settlements due to plastic flow, and the settlements due 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 foundation clay are often designed on 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. Since 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 of completely separating 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 strenth, it should be complemented by field pore pressure measurements.

CONCLUSIONS

The following conclusion 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 over the use of time-settlement data.

(3) The assumption that the ratio $\bar{s}/\bar{p}$ is a constant in a normally consolidated clay appears reasonable in the prediction of shear strength increase.

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The contents of this paper reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

APPENDIX

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Weber, W. G. (1966), "Experimental Sand Drain Fill at Napa River," paper presented at 45th Annual Highway Research Board Meeting, Washington, D. C., Jan. 1966.

Weber, W. G. (1968), "In Situ Permeabilities for Determining Rates of Consolidation," California Division of Highways, Materials and Research Department, Presented at the 47th Annual Meeting of the Highway Research Board, January 1968.

Table 1

Summary of Properties of the

Compressible Subsoil at Napa River

Properties Classification	Depth in Feet: 0 - 6	6 - 26	26 - 46
	Top soil silt-clay w/extensive organic matter and seashells	Very soft silty clay w/varying amount of peat	Soft to firm silty clay w/trace of seashells and peat
Natural density (pcf)	85 - 90	90 - 95	95 - 105
Natural water content (%)	95 - 120	80 - 95	68 - 76
Liquid Limit (%)	90 - 110	70 - 95	60 - 75
Plasticity Index (%)	45 - 60	35 - 50	22 - 42
% finer than #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}	
Coeff. of consolidation (sq.ft/day)		0.05-0.022	

Table 2

Calculation of Shear Strength Increase from Field Pore Pressure Measurements, Napa River Project.

$$\bar{s}/\bar{p} = 0.21, \ \bar{p} = 2750U + \bar{p}_i$$

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

Elapsed Time in Days	Excess Pore Pressure in ksc	U in %	Boring Number	Effective Vertical Stress, $\bar{p}$, in psf			Calculated Shear Strength in psf			Measured Shear Strength in psf		
				O.G.	20'	40'	O.G.	20'	40'	O.G.	20'	40'
0	1.10	0	D-11,12 and 87	0	860	1860	0	180	390	0	220	380
150	.95	14	D-213	385	1245	2245	80	260	470	80	300	500
480	.65	41	D-224	1130	1990	2990	240	420	630	160	380	550
750	.50	55	D-302	1520	2380	3380	320	500	710	240	450	650
1440	.30*	73	D-504	2650**	3510	4510	560	740	950	450	870	1100

Note: * The excess pore pressure due to the first stage of embankment load only.
**Assuming 50% dissipation from second stage of loading.

Table 3

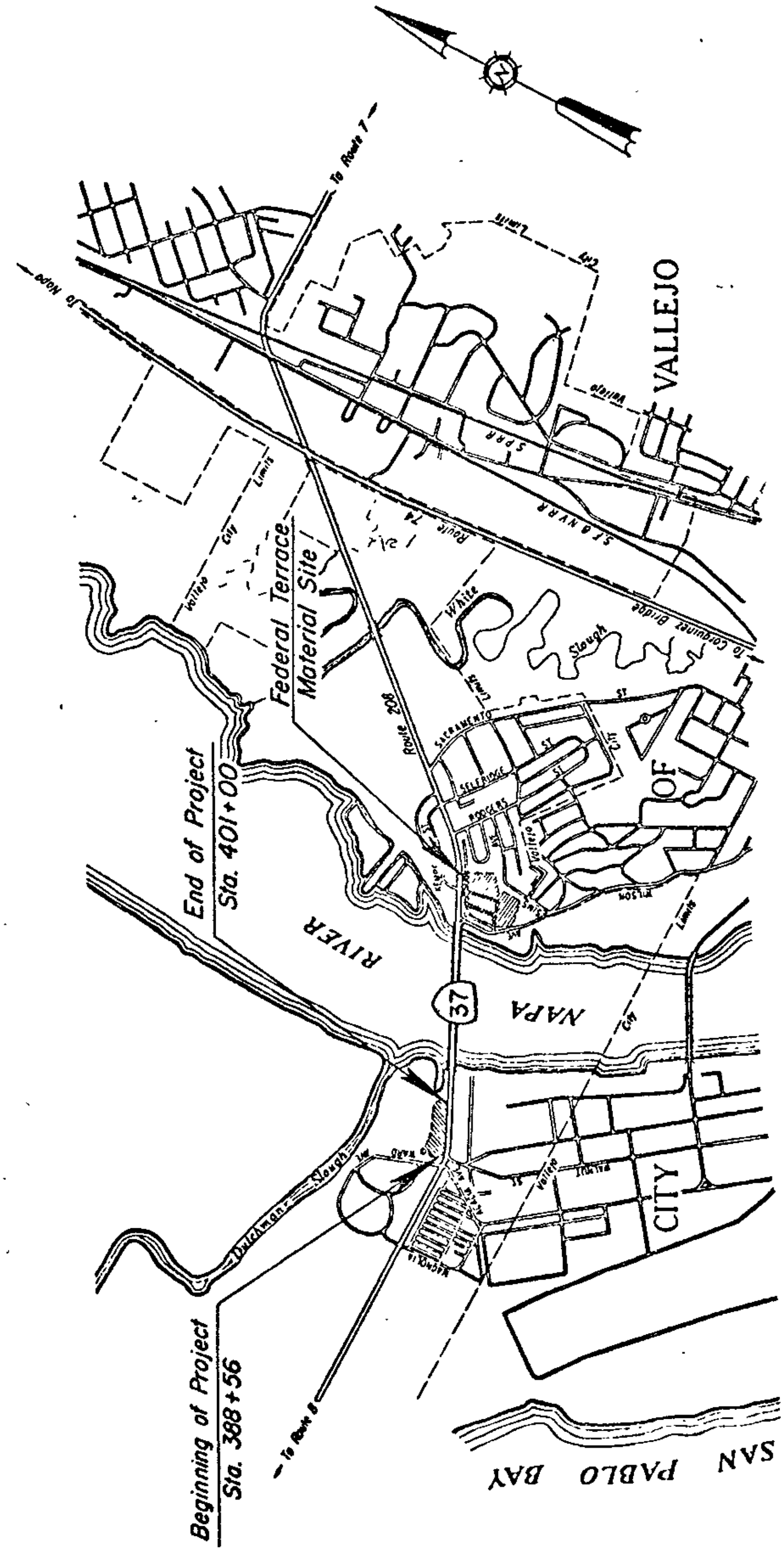
Calculation of Shear Strength Increase from Field Settlement Measurements, Napa River Project.

$\bar{s}/\bar{p} = 0.21$

$\bar{p} = 2750\ U + \bar{p}_i$

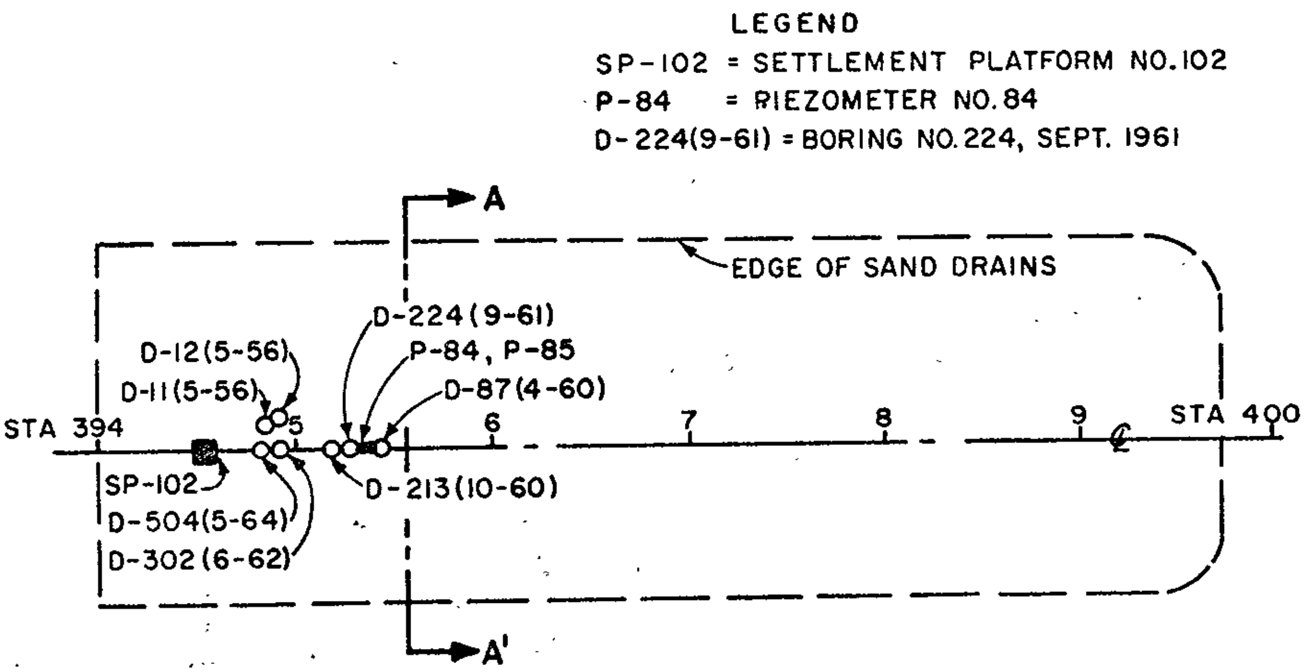
ultimate settlement = 16 ft.

Elapsed Time in Days	Settlement in Feet	U in %	Boring Number	Effective Vertical Stress, $\bar{p}$, in psf			Calculated Shear Strength in psf			Measured Shear Strength in psf		
				O.G.	20'	40'	O.G.	20'	40'	O.G.	20'	40'
0	0	0	D-11,12 and 87	0	860	1860	0	180	390	0	220	380
150	4.4	28	D-213	760	1620	2620	160	340	550	80	300	500
480	8.3	51	D-224	1400	2260	3260	290	480	690	160	380	550
750	9.8	60	D-302	1650	2510	3510	320	530	740	240	450	650
1440	11.2	69	D-504	1900	2760	3760	400	580	790	450	870	1100

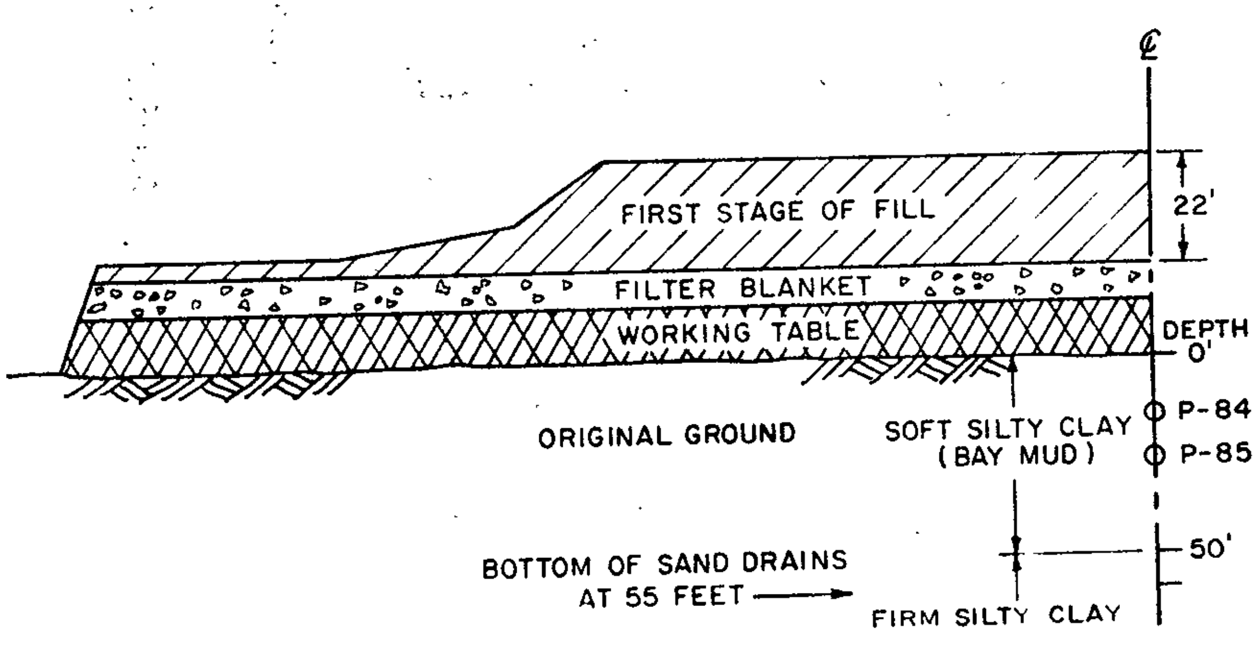


PROJECT SITE LOCATION MAP

Figure 1

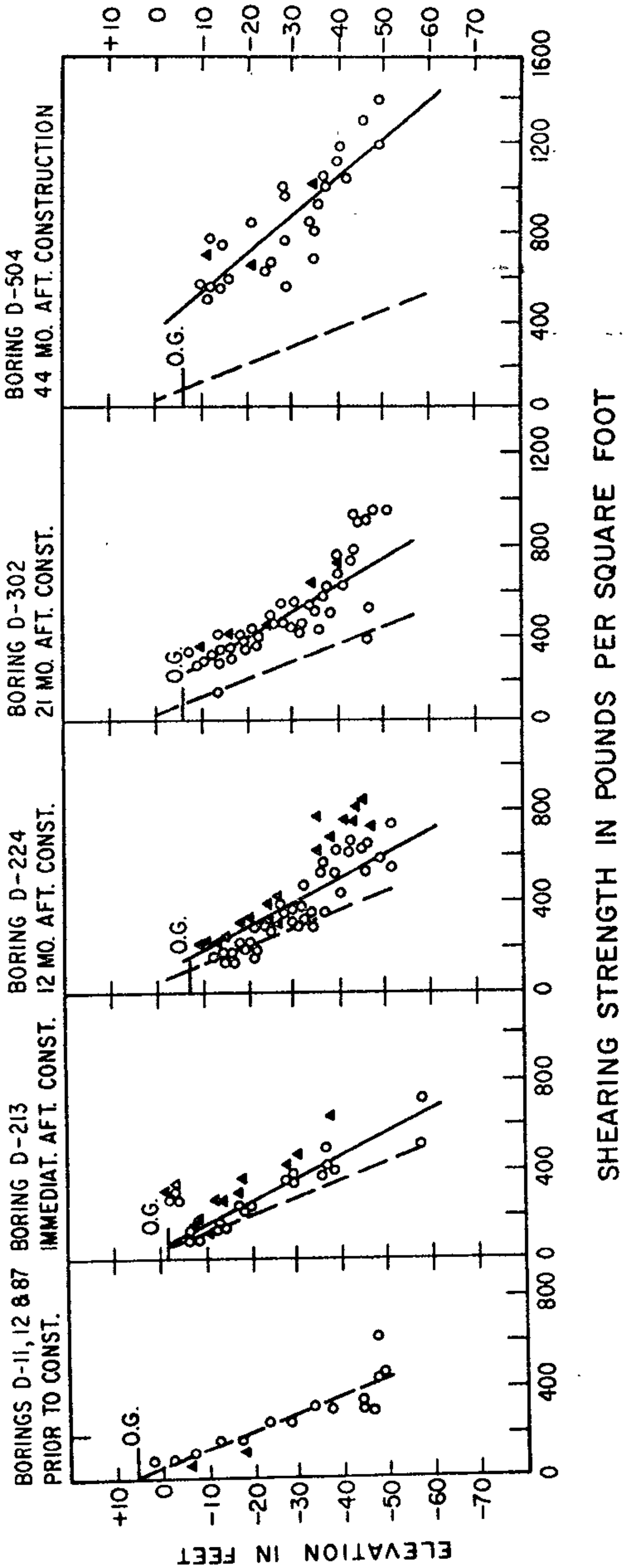


(a) PLAN: LOCATIONS OF SETTLEMENT PLATFORM, PIEZOMETERS, AND BORING HOLES. (SCALE, 1 INCH = 200 FT.)



(b) TYPICAL CROSS SECTION A-A' (NOT TO SCALE)

FIG. 2 PLAN AND CROSS SECTION OF TEST SITE
NAPA RIVER PROJECT



LEGEND

o - UNCONFINED COMPRESSION TEST
▲ - U U TRIAXIAL COMPRESSION TEST
O.G.- ORIGIANAL GROUND

– – – INITIAL SHEAR STRENGTH
——— INCREASED SHEAR STRENGTH

FIG. 3 DETERMINATIONS OF SHEARING STRENGTH
NAPA RIVER PROJECT

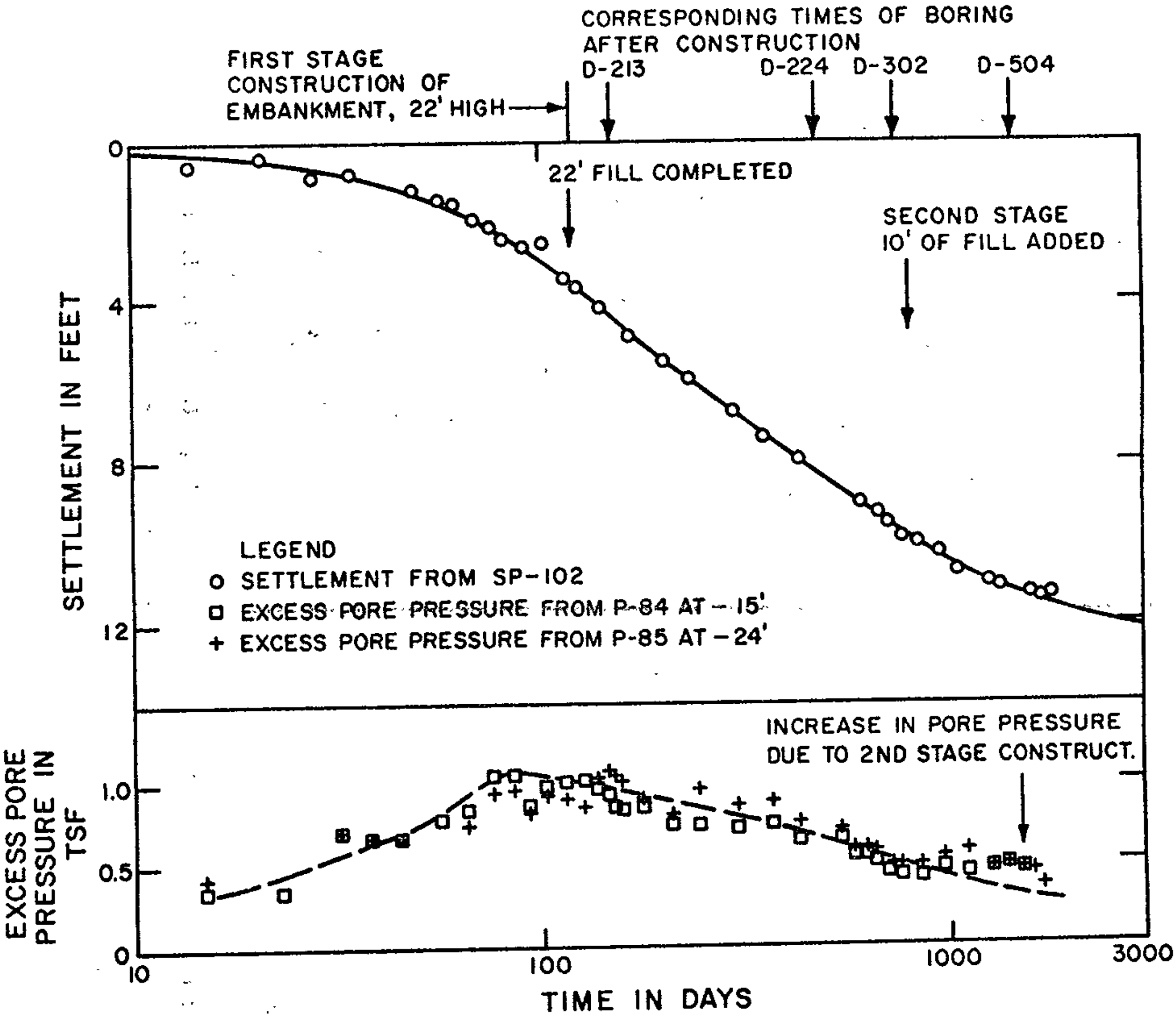


FIG. 4 FIELD MEASUREMENT DATA OF SETTLEMENTS
AND EXCESS PORE PRESSURES
NAPA RIVER PROJECT

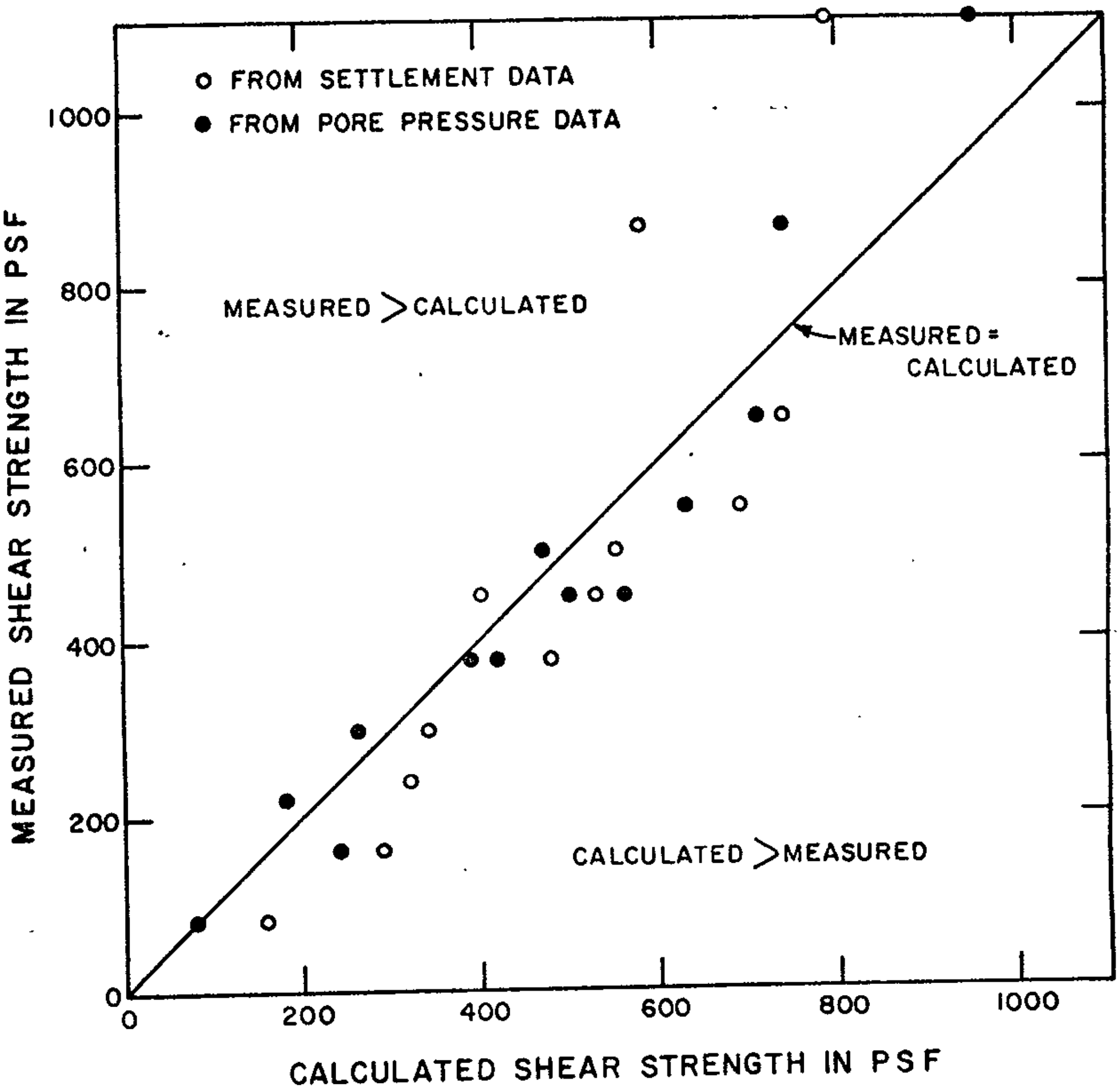


FIG. 5 COMPARISON OF MEASURED AND CALCULATED SHEAR STRENGTH (FROM TABLE 1 AND 2) NAPA RIVER PROJECT