Application of a Test Fill at a Layered Clay Site

SCOTT A. ASHFORD AND SANDRA L. MADSEN

Settlement estimates can often dictate structure type selection and construction scheduling in the design of multispan highway structures. At layered clay sites where several different clays exist, the uncertainties associated with estimating settlement rate and magnitude can lead to excess conservatism in the design of these structures. A case history in which a 35-ft-high test fill was used to refine settlement estimates for a 2-mi-long freeway realignment is presented. The soil profile consisted of alternating layers of normally consolidated and overconsolidated clay. How settlement properties of individual soil units were backcalculated using the results of several types of instrumentation is described. The resulting soil model was used to develop preload and surcharge recommendations for the abutments of 13 multispan structures. The test fill eliminated excess conservatism in the settlement estimates for design and allowed the project to continue on its fast-track design and construction schedule.

An important geotechnical consideration for designing multispan highway structures is settlement. Long-term settlement of a structure can result in high maintenance costs, and excessive differential settlement between spans can cause structural distress. Design estimates of settlement rate and magnitude can dictate construction schedule and structure type selection. Often preloading abutments before structure construction is necessary, and the preload may contain a surcharge load in excess of the final embankment load to facilitate settlement.

At layered clay sites, the uncertainties associated with estimating settlement rate and magnitude can lead to overly conservative recommendations for design. Some of these uncertainties can be overcome at layered clay sites through the use of a test fill. For the Great America Parkway and State Route 237 Realignment (GAP) project, a combination of large fills, layered normally consolidated clay, and a fast-track schedule led to the construction of a test fill to refine estimates of settlement rate and magnitude.

The GAP project consists of constructing nearly 2 mi of new six-lane freeway on the margins of San Francisco Bay, just north of San Jose, California. The realigned freeway must cross several creeks, local streets, and a railroad. The proposed construction consists of a series of 13 multispan bridge structures connected by fills up to 35 ft high and 400 ft wide at the base. Fills of this size, involving 1.5 million yd^3 of material, were unprecedented in the area. A site plan and profile are shown in Figure 1.

The remainder of this paper discusses the investigation used to estimate the settlement of the bridge structures due to the large fills and, in particular, how a test fill was used to reduce uncertainties in the settlement estimates and enable the project to proceed economically with design.

CH2M HILL, 6425 Christie Avenue, Suite 500, Emeryville, Calif. 94608.

SOIL CONDITIONS

To a depth of several hundred feet the soil along the alignment consists predominantly of alternating deposits of Pleistocene alluvium and older bay mud, though a desiccated crust of younger bay mud is present. The stress history of the Pleistocene deposits varies greatly because of periodic flooding and desiccation.

Soil conditions along the alignment were studied through an extensive field exploration and laboratory testing program. The field program consisted of nearly 50 soil borings, more than 70 cone and piezo-cone penetrometer tests (CPTs), groundwater monitoring, test pit logging, field vane testing, and geophysical logging. Laboratory soil testing included a large number of index tests, 70 consolidation tests, and a variety of strength tests.

On the basis of this effort, the soil below the alignment was found to consist primarily of heavily overconsolidated clay interbedded with some slightly overconsolidated clay layers, all with moderate plasticity. These slightly overconsolidated layers, which may enter virgin compression upon loading, will be referred to as normally consolidated in this paper. The clay was divided into soil "units" on the basis of the results of consolidation tests and CPT data, as well as appearance, plasticity, strength, and geologic history. The characteristics associated with each soil unit are given in Table 1. The same units were present in layers of different thicknesses across the entire site. Figure 1 shows the soil profile as it varied along the alignment. The normally consolidated clay emphasized in the figure is actually composed of three separate soil units (3D, 4A, and 5A).

The stress history of the normally consolidated clay layers was critical in estimating settlements. In these soil layers, the preconsolidation pressure (P_c) of the soil could not be precisely defined, despite laboratory consolidation testing specifically designed to do so. These tests included reduced load increments near P_c to better define the break from recompression to virgin compression. Casagrande's construction (*1*) was primarily used to estimate P_c from the laboratory data, and Schmertmann's method (*2*) was used to reconstruct field curves.

The difficulty in estimating the precise P_c was primarily due to two factors. First, even for a normally consolidated soil, the preconsolidation pressure is relatively high at depths of

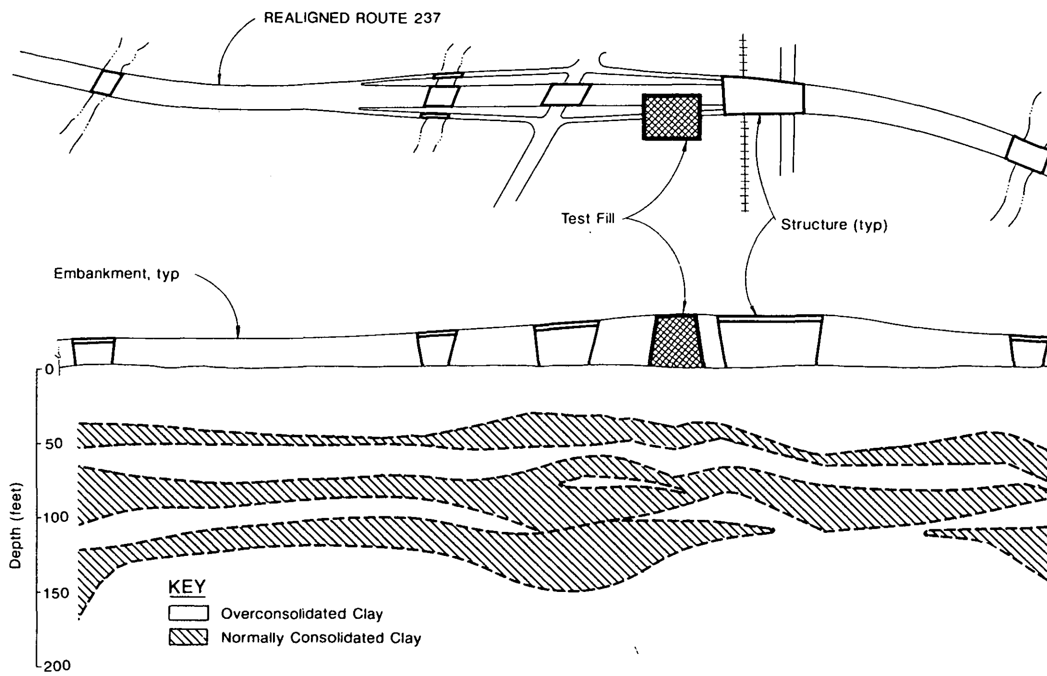

FIGURE 1 Site plan and profile and generalized subsurface profile.

TABLE 1 Soil Unit Properties

Soil Unit	Saturated Unit Weight (psf)	Natural Water Content (%)	Liquid Limit	Plasticity Index	Void Ratio	C_c	Consistency
2	118	34	49	26	0.93	0.30	Very Stiff
3A	120	33	34	21	0.90	0.38	Firm to Stiff
3B	120	32	42	19	0.88	0.26	Firm to Stiff
3C	130	22	42	23	0.60	0.26	Very Stiff
3D	123	29	40	18	0.80	0.30	Firm to Stiff
3E	123	29	45	20	0.70	0.26	Stiff to Very Stiff
4A	125	27	41	17	0.75	0.22	Firm to Stiff
4B	127	26	41	19	0.71	0.26	Stiff to Hard
5A	123	28	32	15	0.77	0.26	Stiff to Very Stiff
5B	126	26	52	24	0.72	0.33	Very Stiff to Hard
6	130	22	39	17	0.60	0.25	Hard
7	131	21	42	20	0.59	0.28	Hard
8	130	22	NT[a]	NT	0.60	0.25	Hard

[a] "NT" indicates not tested.

50 to 150 ft. Small variations on a typical plot of void ratio (e) and log of pressure (log P) result in differences of a few thousand pounds per square foot in P_c. Second, even with great care, the sample disturbance and stress relief associated with sampling from those depths result in somewhat rounded e–log P curves.

These two factors, in combination with the layered soil profile, lead to wide data scatter for the estimate of P_c. The precise value of P_c is important because if varied slightly, settlement estimates changed drastically. Reductions of less than 15 percent in P_c, which approaches the accuracy of the computational method, more than doubled the estimated settlement because the soil entered virgin compression.

Just as important, or even more so for construction scheduling, the rate of settlement was also uncertain. Initial estimates of settlement period ranged from 1 to 5 years for the 2 to 4 ft of estimated settlement. This range in estimated settlement rate was large because it was difficult to accurately determine the factors that control it. The settlement rate was determined using the consolidation characteristics of the soil, the state of stress, and the location of drainage layers. None of these factors could be established with enough accuracy to reduce the estimated range of settlement rate.

The uncertainties in determining P_c and in establishing settlement rates led to initially conservative settlement estimates for design. However, the long preload period and magnitude of long-term settlement associated with these initial estimates made potential project costs increase substantially.

TEST FILL

To economically design and construct the project, the estimates of settlement rate and magnitude needed to be refined by reducing the uncertainties. It did not appear that these estimates could be improved using the results of additional laboratory tests. Therefore, a 350-ft by 300-ft by 35-ft-high test fill was built to confirm and refine the estimates of settlement rate and magnitude. This approach allowed the project to proceed on a reasonable schedule using the engineer's best estimate of settlement on the basis of the available laboratory and field data, while accepting the possibility of redesign if these estimates were not confirmed by the test fill. Without the test fill, much more conservative settlement estimates would have been used in the design because of the uncertainty in the consolidation parameters exhibited by the laboratory tests.

Ideally, the stress distribution beneath the test fill should be identical to distribution beneath a typical proposed freeway section. The size of the test fill was, however, limited by budget and space constraints. Therefore, the test fill was designed so that the stress distribution beneath its center would be similar to the stress distribution beneath the proposed structure abutments, as shown in Figure 2.

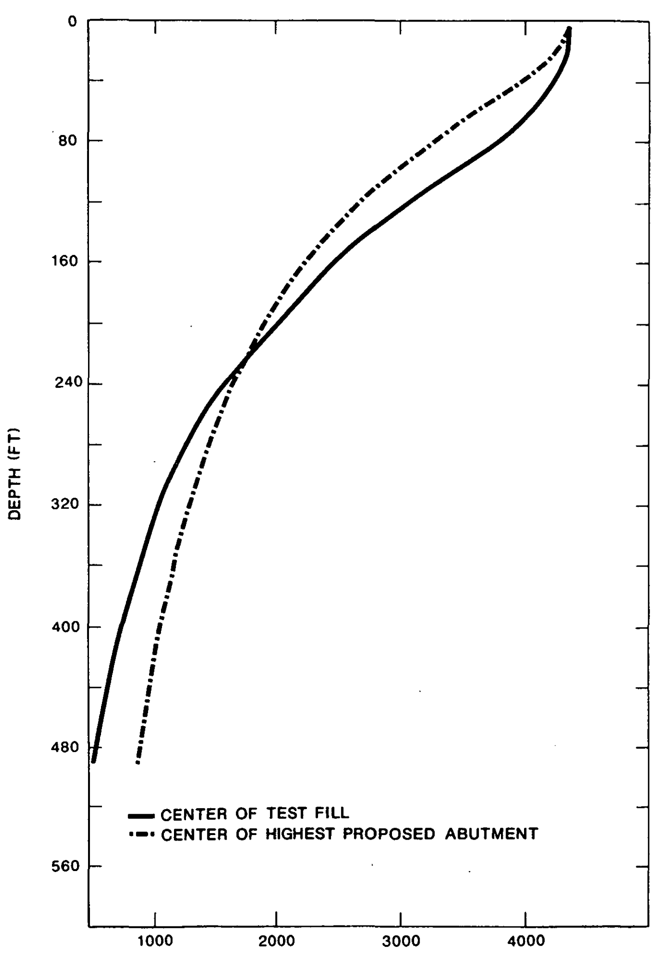

FIGURE 2 Comparison of stress distribution.

INSTRUMENTATION

The purpose of the instrumentation was to yield enough data to backcalculate settlement properties for each of the different soil units. Four types of instrumentation were installed: surface settlement risers, Sondex settlement systems, extensometers, and pore-pressure transducers. The configuration and instrumentation of the test fill are shown in Figure 3. Instrumentation for the test fill was installed just before construction of the test fill.

Ten surface settlement risers were installed to measure original ground surface settlement with time. Each settlement riser was constructed of a ¾-in. ductile iron pipe attached to a wooden platform resting on a bed of sand at the original ground surface. A 1½-in. outer casing was used to protect the riser from downdrag of the surrounding fill (*3*). The risers were extended up in 5-ft segments as the test fill was placed. Original ground surface settlement was monitored by horizontal and vertical survey of the top of the riser. The horizontal survey was used to correct for nonverticality.

Three Sondex settlement systems (*4*) were installed to a depth of 140 ft to monitor the variation of settlement within each soil unit over time. The Sondex system consists of a sensing probe and a series of metal rings fixed at regular 5-ft intervals to flexible corrugated polyethylene pipe. The corrugated pipe was grouted into a borehole using a cement-bentonite grout with a stiffness similar to that of the surrounding soil. As soil settled within the upper 140 ft, the flexible pipe compressed in proportion to the settlement, thus changing the location of the metal rings. The depths of the metal rings were determined by lowering the sensing probe through the pipe. By comparing the depths of the rings at later times with the initial readings, settlement with depth and time was determined.

Two extensometers were installed to monitor settlement below 180 ft over time (*4*). Each extensometer consists of a ¼-in. stainless steel reference rod, a protective ABS plastic pipe casing, and a hydraulic anchor. The reference rods extended from the ground surface to a depth of approximately 180 ft and were supported at the bottom of the borehole by the hydraulic anchor. The protective pipe, which fit closely around the reference rod, was grouted into the borehole using a cement-bentonite grout. The protective pipe prevented soil downdrag on the rod by sliding down with the soil; three slip couplings attached to the pipe each provided approximately 12 in. of movement. Settlement below 180 ft was determined by surveying the top of the reference rod.

Three sets of five pneumatic pore-pressure transducers were installed at depths from 30 to 180 ft. They were installed to monitor changes in pore water pressure over time in an effort to better define the end of primary consolidation. A mandrel was used to push the transducers into the soil at the bottom of a borehole. The borehole above the transducer was filled with a cement-bentonite grout. Pore pressures were recorded pneumatically from the ground surface.

As shown in Figure 3, most of the instruments were located in one of three locations, which are referred to as instrument clusters. The first cluster was in the center of the test fill, the second at the middle of the west side, and the third at the southwest corner. Instruments were installed in clusters to

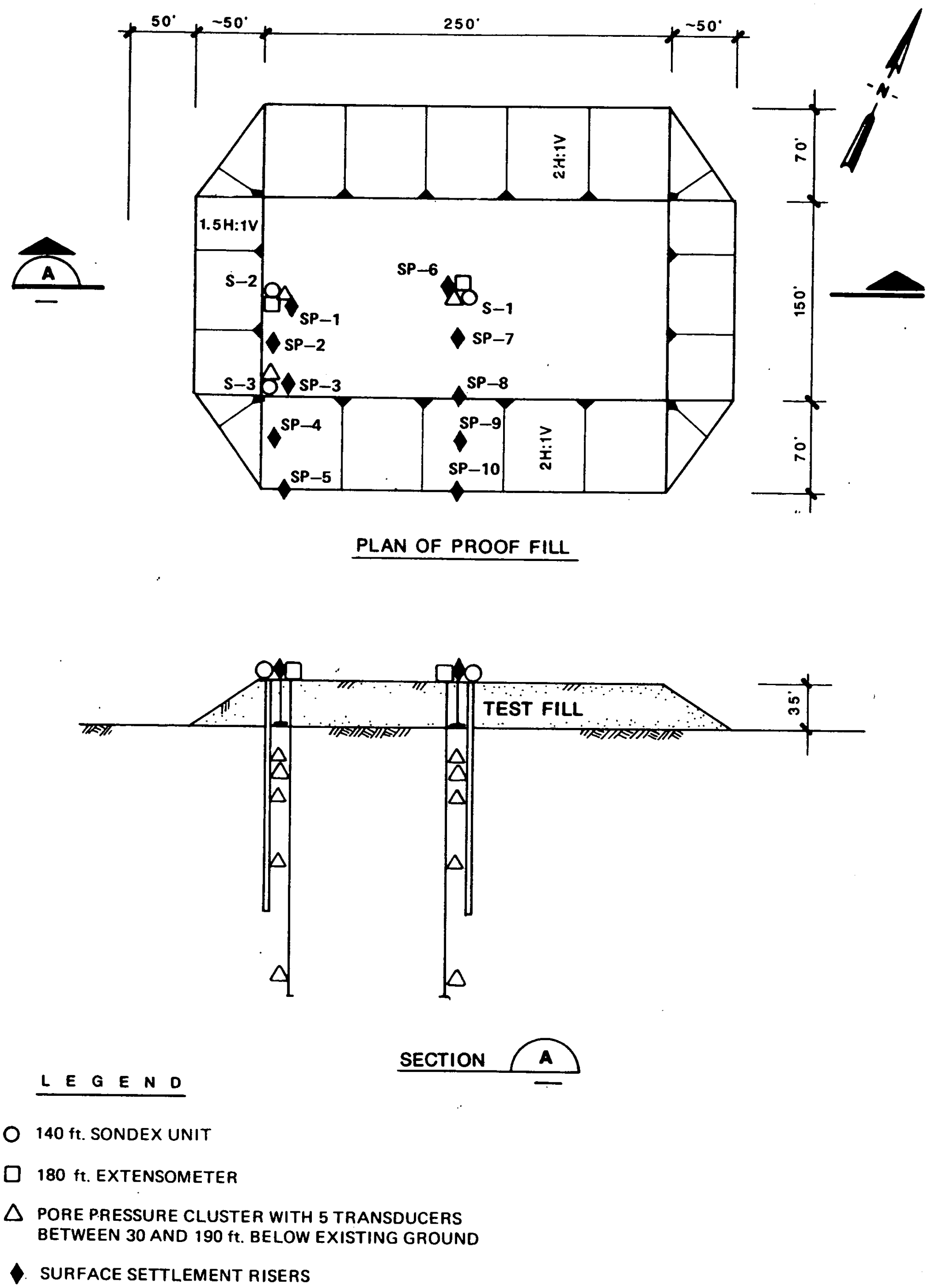

FIGURE 3 Plan and section of test fill.

collect different types of measurements within similar soil conditions and stress regimes.

SUMMARY OF MONITORED SETTLEMENT

A baseline set of instrumentation readings was obtained before construction began. Monitoring was performed weekly during the 2-month construction period and then gradually decreased in frequency. Monitoring continued for 12 months. The surface settlement riser data are shown in Figures 4 and 5. A summary of the Sondex data is shown in Figures 6, 7, and 8.

The surface risers and the Sondex were the most useful in monitoring the consolidation of the test fill. After 7 months, measurements from both sets of instruments began to level off at approximately 1.5 feet. The total settlement measured by the Sondex was 1 to 3 in. less than that measured by the

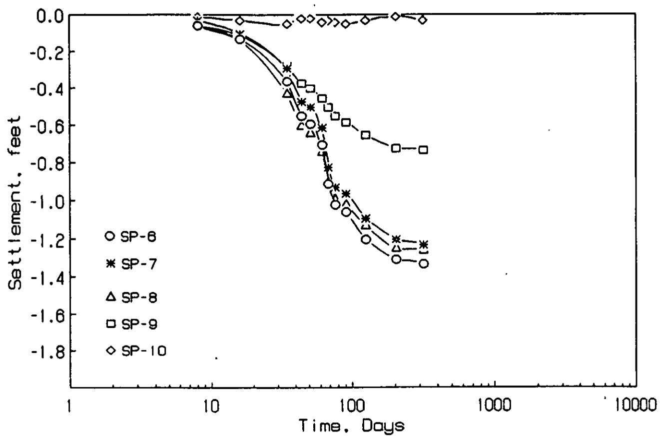

FIGURE 4 Surface settlement over time—settlement risers, center, test fill.

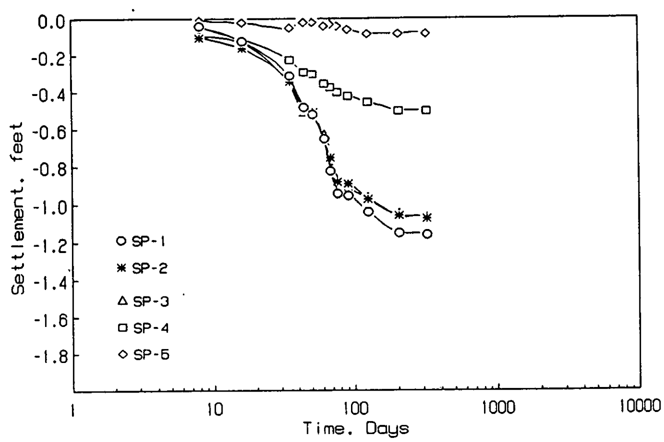

FIGURE 5 Surface settlement over time—settlement risers, west end, test fill.

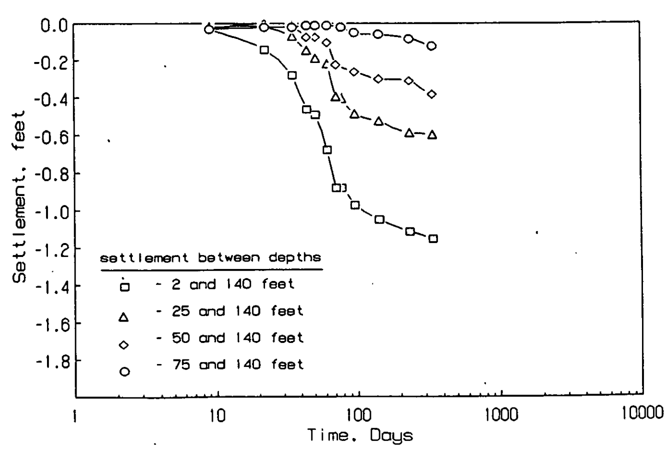

FIGURE 6 Settlement over time—Sondex S1, center, test fill.

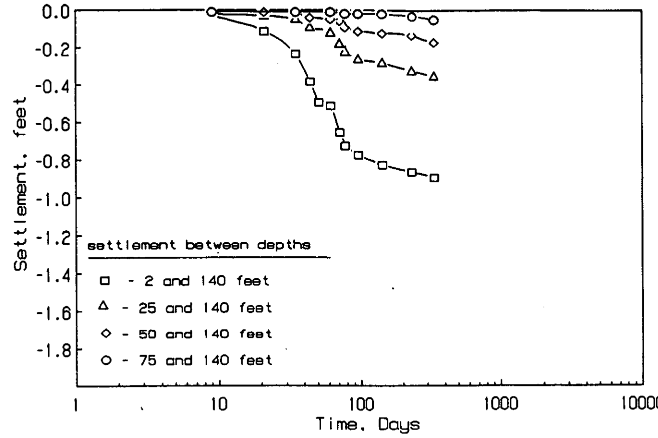

FIGURE 7 Settlement over time—Sondex S2, west side, test fill.

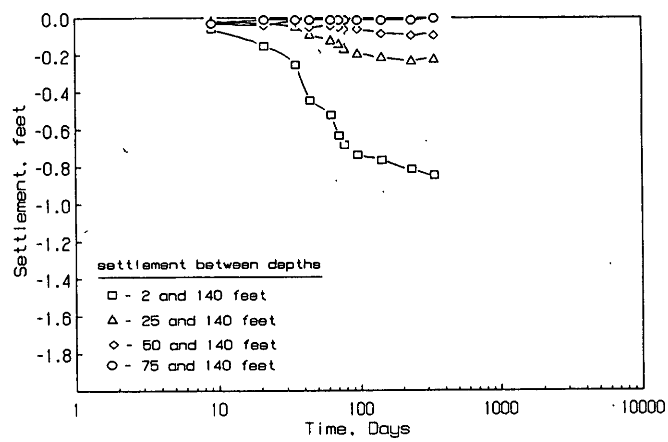

FIGURE 8 Settlement over time—Sondex S3, southwest corner, test fill.

surface risers, indicating that some settlement did occur below 140 ft.

Though no instruments were completely lost to damage, two systems, the extensometer and the pore pressure transducer, did not yield useful results. The extensometer data indicated that unreasonably high settlements occurred below a depth of 180 ft (nearly 1 ft). This is probably due to incomplete anchoring at the base of the reference rod. Difficulties were experienced in activating the hydraulic anchors during installation. Incomplete anchoring may have allowed the rod-anchor assembly to creep downward under the weight of the system. Other causes for the unreasonably high settlement readings may have been transfer of downdrag forces to the reference rod due to inoperative slip couples on the protective casing or bending of the extensometer system in the borehole because the cement-bentonite grout may have been too weak.

Although the pore pressure transducers (PPTs) appeared to function properly, they showed very little change with time, indicating either that buildup in pore water pressure was very small or that any buildup of the pore water pressure dissipated rather quickly. The absence of substantial excess pore water pressures may indicate that the PPTs were either installed in overconsolidated clay, were close to a drainage layer, or that voids or cracks formed around the PPTs during their installation that allowed rapid drainage.

DEVELOPMENT OF SOIL MODEL

The original settlement estimates at each abutment location on the GAP project were based on field and laboratory tests across the entire site for each soil unit. The purpose of the test fill data analysis was to adjust the soil properties so that the settlement estimates along the alignment could be refined. Because the Sondex instruments measured settlement with depth, the data from those instruments were used to adjust the soil properties. By comparing the depth of specific soil units with the depths of particular Sondex rings, the settlement within each unit was determined. The data from the surface settlement risers were used to check the accuracy of the Sondex data and as an indication of settlement occurring below 140 ft.

The measured settlement was treated as consolidation settlement. Whereas immediate settlement undoubtedly contributed to some settlement of the Sondex rings, it could not be isolated within the settlement measurements. However, it was estimated to be less than 15 percent of the total settlement. Therefore, the revised consolidation parameters of the clay reflect a combination of immediate and consolidation settlement.

Determination of Soil Properties Governing Settlement Rates

The rate of settlement within any layer was determined by analyzing settlement over time data obtained from the Sondex measurements. Specifically, the analysis involved determination of the coefficient of consolidated (C_v) for each soil layer. Equations presented by Terzaghi and Peck were used as the basis for analysis (*5*).

Method of Analysis

The analyses of settlement rate involved determination of the coefficient of consolidation (C_v) for each soil unit. C_v is defined as the following:

$$C_v = \frac{D^2 T}{t} \tag{1}$$

where

D = drainage path,
t = time from start of consolidation, and
T = a dimensionless time factor.

The time factor, T, was related to the percent consolidation (U) using published curves (*5*). By graphically determining the time to 50 percent consolidation (t_{50}) for each soil unit from the Sondex data and using the time factor (T) for 50 percent consolidation (two-way drainage), the coefficient of consolidation was computed using the following equation:

$$C_v = \frac{0.2D^2}{t_{50}} \tag{2}$$

For estimation of t_{50} values, the start of consolidation (t_0) was assumed to be at the midpoint of the fill construction, and the strain in each layer was assumed to decrease with depth. Values of t_{50} were calculated for all soil layers except for those layers that had not finished consolidating or that had not settled enough to create a meaningful settlement curve.

For each soil unit at each cluster location, it was assumed that the drainage path was the average distance to a sand layer or seam based on the soil boring and CPT logs. Drainage layers were not evident within three of the soil units at the test fill. The soil unit thicknesses and drainage path lengths used in the final analysis for each cluster location are presented in Table 2.

Results

C_v values were assigned to particular soil units by comparing average C_v values calculated from the Sondex data with the expected range of C_v from laboratory tests and with published correlations of C_v and liquid limit (*6,7*). In soil layers where a drainage path was not evident, values of C_v could not be calculated. Instead, a normalized consolidation coefficient, C_v/D^2, was determined. The value of this coefficient was used at other bridge abutment locations without being changed to account for different drainage conditions. This value may overestimate the settlement period when used at other locations where drainage paths are evident, but it appeared to give reasonable results. Table 3 compares the original soil properties with those determined from the test fill data.

Values of C_v and C_v/D^2 reflect a combination of virgin compression and recompression. It was not practical to separate the two parameters using the test fill data. However, since the geometry, induced stresses, and drainage modes of the test fill are similar to those expected under the abutments, the combined parameter values should be applicable across the site.

Determination of Soil Properties Governing Settlement Magnitude

The magnitude of settlement in overconsolidated and normally consolidated clay was determined using Sondex and settlement riser data. The equations used in the analysis are based on those presented by Terzaghi and Peck (*5*).

Methods of Analysis

The purpose of the analysis was to refine settlement estimates by using field measurements to backcalculate parameters that govern consolidation. Settlement of an overconsolidated soil stressed into virgin compression was governed by seven parameters: soil thickness (H), void ratio (e), compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c),

TABLE 2 Soil Unit Thickness and Drainage Path at Cluster Locations

Center			West Side			Southwest Corner		
Soil Type	Thickness (ft)	Drainage Path (ft)	Soil Type	Thickness (ft)	Drainage Path (ft)	Soil Type	Thickness (ft)	Drainage Path (ft)
2	6.0	6	2	6.5	6.5	2	6.5	6.5
3A	4.0	4	3A	4.0	4	3A	4.0	4
SAND	3.0	1	3B	5.0	2.5	3B	5.0	10
3B	6.0	2.5	SAND	4.0	1	3B	6.0	10
3B	7.0	6	3B	5.0	5	3C	20.0	ND
3C	14.0	ND[a]	3C	17.0	ND	3D	12.0	ND
3D	13.0	ND	3D	5.0	ND	3E	20.0	ND
3E	18.0	ND	3E	23.0	ND	SAND	5.0	1
SAND	6.0	1	SAND	6.0	1	4A	16.0	8
4A	12.0	8	4A	25.0	15	SAND	10.0	1
4B	16.0	16	4B	5.0	5	5A	22.0	10
5A	19.0	15	5A	20.0	16	5B	75.0	20
5B	79.0	20	5B	76.0	15	SAND	13.0	1
SAND	13.0	1	SAND	13.0	1	5B	52.0	30
5B	52.0	20	5B	52.0	30	6	10.0	10
6	10.0	10	6	10.0	10	SAND	25	1
SAND	25.0	1	SAND	25.0	1	6	35	15
6	30.0	15	6	35.0	15	7	145.0	30
7	145.0	30	7	145.0	30	8	20	50
8	20.0	50	8	20.0	50			

[a] "ND" indicates drainage path was not determined for soil layer.

TABLE 3 Comparison of Original and Revised Properties

Soil Unit	C_v (ft²/yr) Original[a]	C_v Revised	C_v/D^2 (1/yr)	C_r Original	C_r Revised	P_c (psf) Original	P_c Revised
2	3-26	80		0.025	0.048	4460	NR[c]
3A	28-61	76		0.034	0.036	6000	NR
3B	11-150	80		0.021	0.046	3500	3500
3C	NA[b]	NA	1.8	0.015	0.017	10000	NR
3D	NA	NA	1.8	0.028	0.030	4600	5900
3E	NA	NA	1.8	0.025	0.043	8320	NR
4A	8-200	37		0.025	0.022	8200	8200
4B	7-54	60		0.026	0.029	14000	NR
5A[d]	27-77	37		0.020	0.017	8250	10700
5B	7-260	50		0.024	0.026	20000	NR
6	310	50		0.025	0.012	32000	NR
7	9	50		0.028	0.017	35000	NR
8	none	50		0.025	0.025	40000	NR

[a] "-" indicates range of values from laboratory test.
[b] NA indicates not applicable.
[c] NR indicates not revised.
[d] Unit 5A did not undergo virgin compression.

initial overburden pressure (P_i), and final overburden pressure (P_f):

$$\text{Settlement} = H\left(\frac{C_r}{1+e}\log\frac{P_c}{P_i} + \frac{C_c}{1+e}\log\frac{P_f}{P_c}\right) \quad (3)$$

To improve the accuracy of the settlement estimates, the most reliable parameters for calculating settlement were identified and then considered constants. The compression index (C_c) and e were assumed to be determined accurately through laboratory tests. The final overburden pressure (P_f) and the initial overburden pressure (P_i) were easily computed. The remaining parameters, C_r and P_c, appear to be the least reliable and, therefore, were backcalculated using the test fill data.

Parameter Revision of Overconsolidated Soil For those soils that did not go into virgin compression, the consolidation equation was reduced to the following:

$$\text{Settlement} = H\left(\frac{C_r}{1+e}\log\frac{P_f}{P_i}\right) \quad (4)$$

The recompression index, C_r, was easily backcalculated at each instrument cluster for each soil unit using Equation 4. The settlement was determined from the Sondex data, the void ratio was determined in the laboratory, and the other parameters in the equation were known. Since these soils did not enter virgin compression, calculation of P_c was not possible (or necessary) from the data.

In some of the deep overconsolidated soil layers, the Sondex did not register settlement. However, since the Sondex may not accurately measure small displacements, settlement was assumed to occur in those layers on the basis of the surface risers. C_r for the deep overconsolidated layers, including the layers that lie below the Sondex base, was determined using laboratory data in addition to experience gained from the other units. This was thought to be conservative, since values of C_r determined in the laboratory were found to be higher than values calculated using field measurements.

Parameter Revision for Normally Consolidated Soil To determine the consolidation parameters of soil that went into virgin compression, C_r was assumed to be both more predictable and less critical to settlement estimates than P_c. The ratio of the recompression index (determined as described previously) to the compression index (measured in the laboratory) in the overconsolidated shallow soils varied from $\frac{1}{10}$ to $\frac{1}{7}$. On the basis of those data, C_r was assumed in the normally consolidated soil to be $\frac{1}{10}\,C_c$. Using this assumption, values of P_c for each soil unit could be backcalculated from the settlement measurements at each cluster since all other parameters in Equation 3 were known.

Final Revision The accuracy of the backcalculated C_r and P_c was checked by running consolidation analyses for each of the three cluster locations, each having a different stress regime. If the average absolute difference between the measured and calculated settlement within any layer for the three clusters was more than 25 percent, C_r or P_c was altered and the settlement recalculated until the average absolute difference was within 25 percent. Since cumulative differences within clusters tended to be compensating, absolute values were used in developing the model in an effort to more closely match the in situ soil properties.

Secondary Compression The duration of monitoring since the completion of primary consolidation was too short to make any reasonable judgment on the magnitude of secondary compression. In the absence of field data, estimates of secondary compression were based on laboratory test results and published values (*6,7*).

Results

A summary of the original and revised properties is presented in Table 3. For the overconsolidated soil units, the value of C_r was typically increased from the values originally estimated from laboratory data, though no trends were apparent. The preconsolidation pressure for the normally consolidated soil units was increased from zero to 30 percent over the values developed from laboratory data. This confirmed that all of the normally consolidated soil units were slightly overconsolidated.

Comparison of Settlement Estimates

A comparison of the original and revised settlement estimates at the center of the test fill, along with the measured settlement, is presented in Figure 9. Figure 9 indicates that the modification of settlement magnitude was not nearly as great as the reduction in the settlement period. The estimated magnitude was reduced by approximately 20 percent, whereas the settlement period was reduced several-fold. The figure shows that the revised settlement estimate is greater than that measured. This is the result of developing the revised properties on the basis of the measured settlement at the three instrument clusters. To get all three estimates within the acceptable error, the revised estimates at the other two clusters are slightly less than those measured.

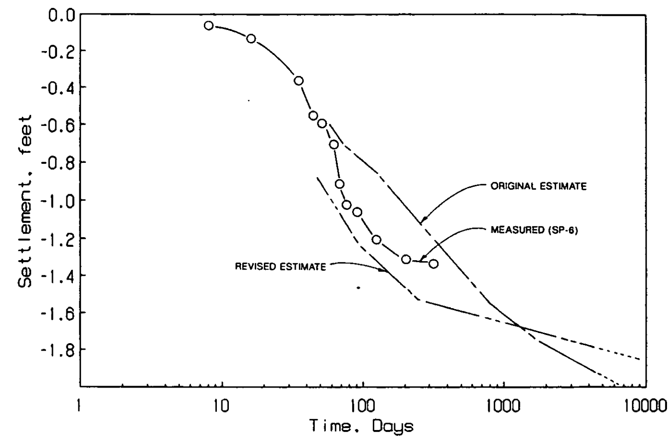

FIGURE 9 Estimated compared with measured surface settlement—center, test fill.

The original settlement estimate shown in Figure 9 includes a correction first proposed by Skempton and Bjerrum (*8*) that allows for lateral deformations. This correction, based on the pore pressure coefficient "A" (A-parameter), is primarily applied to overconsolidated soil layers and reduced the calculated consolidation settlement by 25 percent. If not for this initial correction, the revisions based on the test fill would have had an even more pronounced effect on settlement magnitude.

APPLICATION OF THE SOIL MODEL

Once the soil model was developed from the information gathered at the test fill, it was used to estimate the settlement of the bridge structures for the entire alignment. On the basis of the calculated rates and magnitudes at each structure location, procedures to reduce the effects of settlement were developed. At all sites, a combination of preloading with surcharge loads was determined to be sufficient to economically construct the project in a reasonable time. Recommended preload periods varied from 6 to 15 months, and surcharges varied from 5 to 15 ft of fill. Before the test fill, preload periods were estimated at between 1 and 5 years. These long preload periods may have forced much of the alignment to a viaduct structure to keep the project on schedule. In addition, since the owner agreed to continue the design using less conservative settlement estimates during test fill monitoring, little redesign was required. Without the test fill, the structure design would have contained excess conservatism, or a substantial redesign effort would have been required following completion of the preload period.

CONCLUSIONS

1. Use of test fill can be an economical way to eliminate excess conservatism in settlement estimates where soil properties are uncertain. With adequate instrumentation soil properties for individual soil layers can be backcalculated.

2. In layered soil profiles, it is critical to monitor settlement with depth if properties are to be backcalculated. With only surface data, it would not have been possible to differentiate behavior between units. This would have prevented extrapolation of the information to other sites along the alignment, where different stress distributions and unit thicknesses were present.

3. Multiple types of instrumentation installed in different stress regimes proved very useful in developing the soil model. The different instruments complemented each other and provided independent checks. The three instrument clusters with different stress regimes provided three independent checks for the soil model.

4. The A-parameter correction factor suggested by Skempton and Bjerrum brought the initially estimated consolidation closer to that measured. It seems appropriate to consider its use on projects in which no test fill is used.

REFERENCES

1. A. Casagrande. The Determination of the Pre-Consolidated Load and Its Practical Significance. *Proc., First International Conference on Soil Mechanics*, Vol. 3, Cambridge, Mass., 1936, pp. 60–64.
2. J. H. Schmertmann. Estimating the True Consolidation Behavior of Clay from Laboratory Test Results. *Proceedings of the American Society of Civil Engineers*, Vol. 79, Separate 311, 1953.
3. Method of Installation and Use of Embankment Settlement Devices, California Test 112. *Standard Test Methods*, Vol. 1, California Department of Transportation, 1978.
4. J. Dunnicliff. *Geotechnical Instrumentation for Monitoring Field Performance*. John Wiley and Sons, Inc., New York, 1988.
5. K. Terzaghi and R. B. Peck. *Soil Mechanics in Engineering Practice* (2nd ed.). John Wiley and Sons, Inc., New York, 1967.
6. J. M. Duncan and A. L. Buchignani. *An Engineering Manual for Settlement Studies*. Geotechnical Engineering Report, University of California, Berkeley, 1976.
7. *Design Manual 7.01, Soil Mechanics*. Naval Facilities Engineering Command, U.S. Department of the Navy, 1986.
8. A. W. Skempton and L. Bjerrum. A Contribution to the Settlement Analysis of Foundations on Clay. *Geotechnique*, Vol. 7, No. 4, 1957, pp. 168–178.

Publication of this paper sponsored by Committee on Transportation Earthworks.