PERFORMANCE OF A FILL FOUNDATION AT WEST POINT, NEW YORK

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After a landslide of about $6\frac{1}{2}$ acres occurred at West Point, New York, investigations indicated that the slide site had been a fill area. Borings were taken at another fill area south of the slide to determine its stability. Visual classifications and natural water content determinations were made on all samples, and representative samples were tested for Atterberg limits, shear strength, and percentage of consolidation. Stability analysis of the south fill area indicated a factor of safety of about 1.0. The intended use of the south fill area, athletic fields for the U. S. Military Academy, was approved provided that piezometers and reference points were regularly observed to ensure that no significant movements were occurring.

•ON AUGUST 25, 1961, a land area of about $6\frac{1}{2}$ acres along the west bank of the Hudson River at the U. S. Military Academy subsided suddenly into the river (Fig. 1). The area had been the site of extensive fill operations in the 14 months prior to the slide. The subsidence, involving the movement of about 2,000,000 yd^3 of earth and resulting backwash in the form of a wave about 8 ft high, caused considerable property loss. Investigations and analyses subsequent to the slide indicated that the fill operations had reduced the factor of safety of the riverward edge of the fill to a critical value. Consequently, it was concluded that initial failure of the riverward edge of the fill was the main cause of the slide.

The slide focused attention on a similar fill area located downstream of the slide and designated as the south fill area (Fig. 2). Foundation investigations in the south fill area included a single undisturbed boring (BU-2), four split-spoon borings, and placement of four Casagrande type of piezometers. The locations of the borings are shown in Figure 2; the piezometers were installed at various depths adjacent to boring BU-2. On the basis of laboratory shear tests on undisturbed samples, stability analyses were performed, which indicated that the south fill area had a narrow margin of safety but that its stability would increase slowly with time. Inasmuch as the area was to be used for athletic fields, it was decided to monitor the foundation movement and pore pressures to corroborate the increase in safety with time. A description of the soil conditions, the results of laboratory tests, instrumentation, and analysis of data are presented later.

SOIL AND GROUNDWATER CONDITIONS

A generalized soil profile through the south fill area is shown in Figure 3. The material overlying the bedrock consists principally of estuarine deposits (an organic silt or clay) with a layer of peat underlying the recent fill. The small pocket of sand and gravel that underlies the layer of peat along the valley wall is considered to be of glacial origin. Some of the fill was placed in 1941. Additional fill, amounting to approximately 120,000 yd^3, was placed in 1960. Fill placement produced the large mud waves shown in Figure 2.

A profile through the south fill area indicating boring logs and piezometric data is shown in Figure 4. Natural water contents of the estuarine deposits varied between about 40 and 70 percent, generally decreasing with depth. Split-spoon resistances varied between about 0 and 3 blows per foot.

Figure 1. Location of slide area.

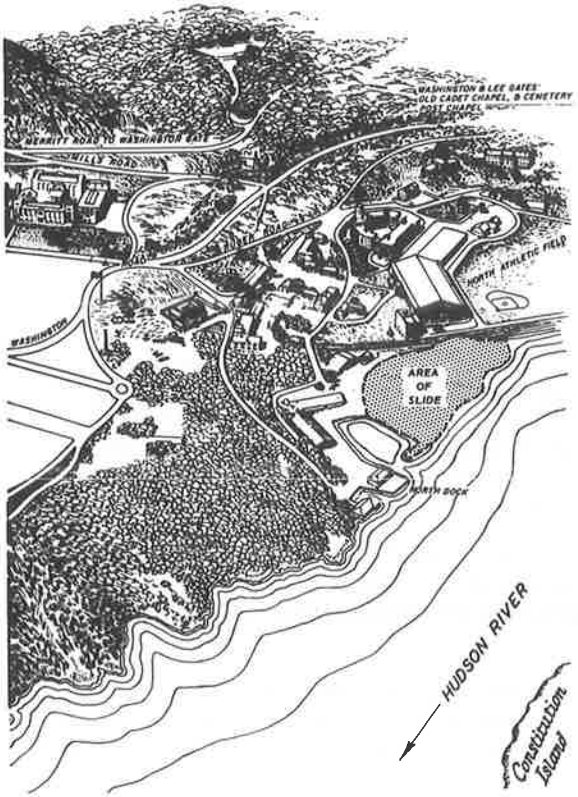


Figure 2. Aerial view of south fill area showing boring locations.

Figure 3. Generalized soil profile through south fill area.

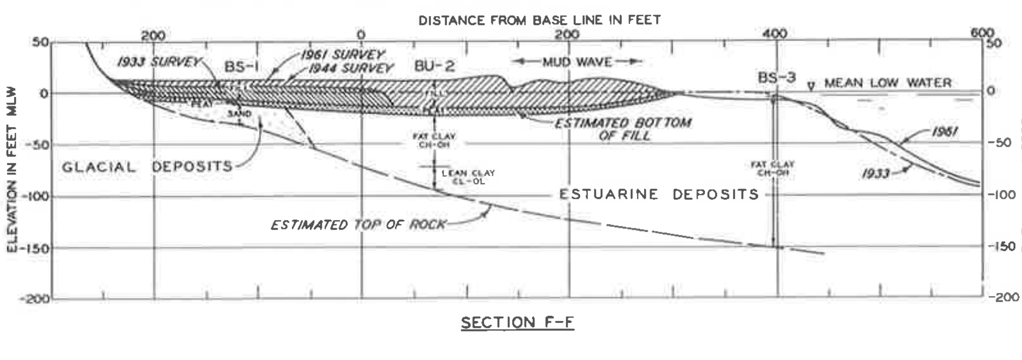
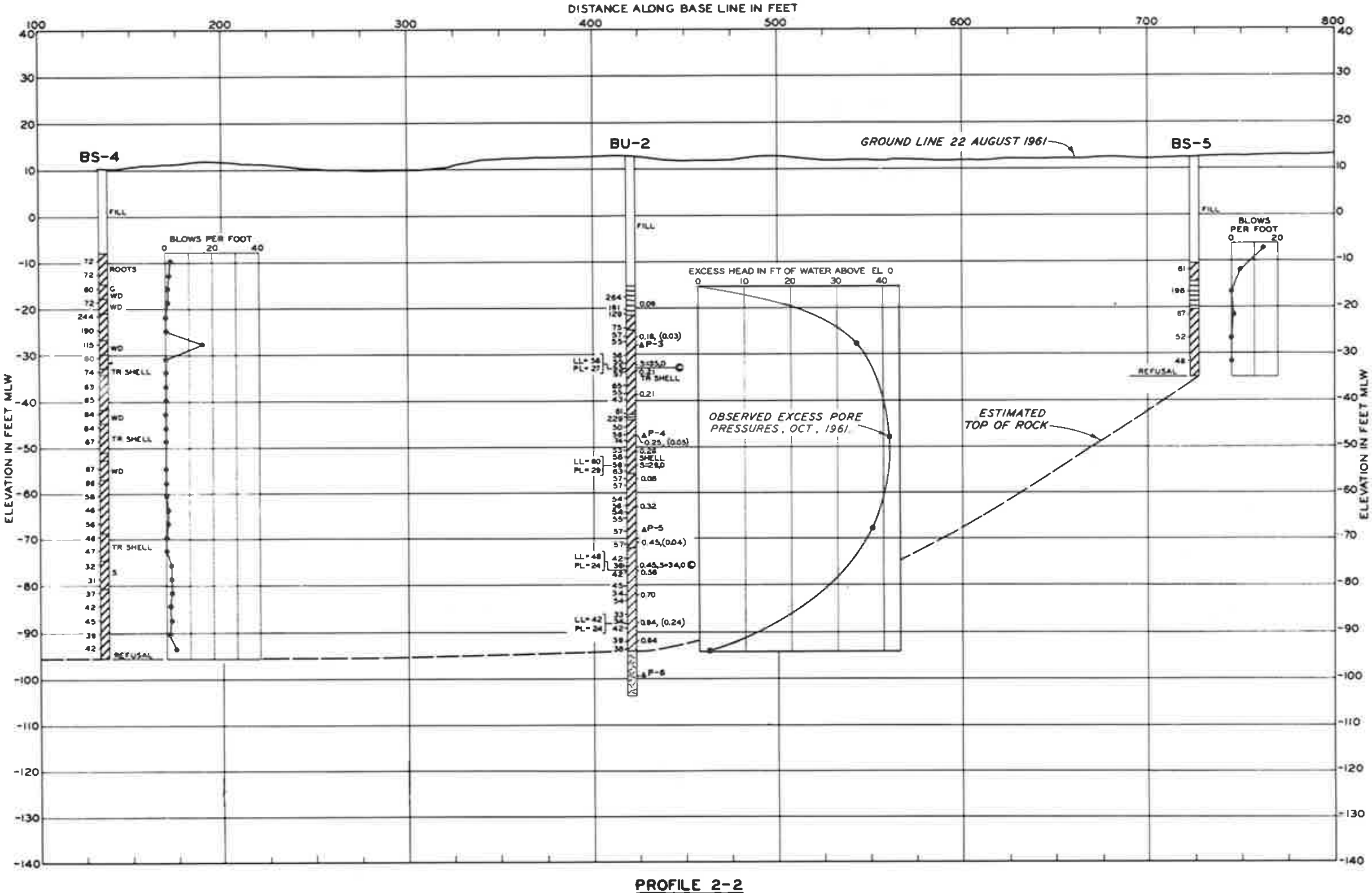


Figure 4. Profile through south fill area showing boring logs and piezometer data.



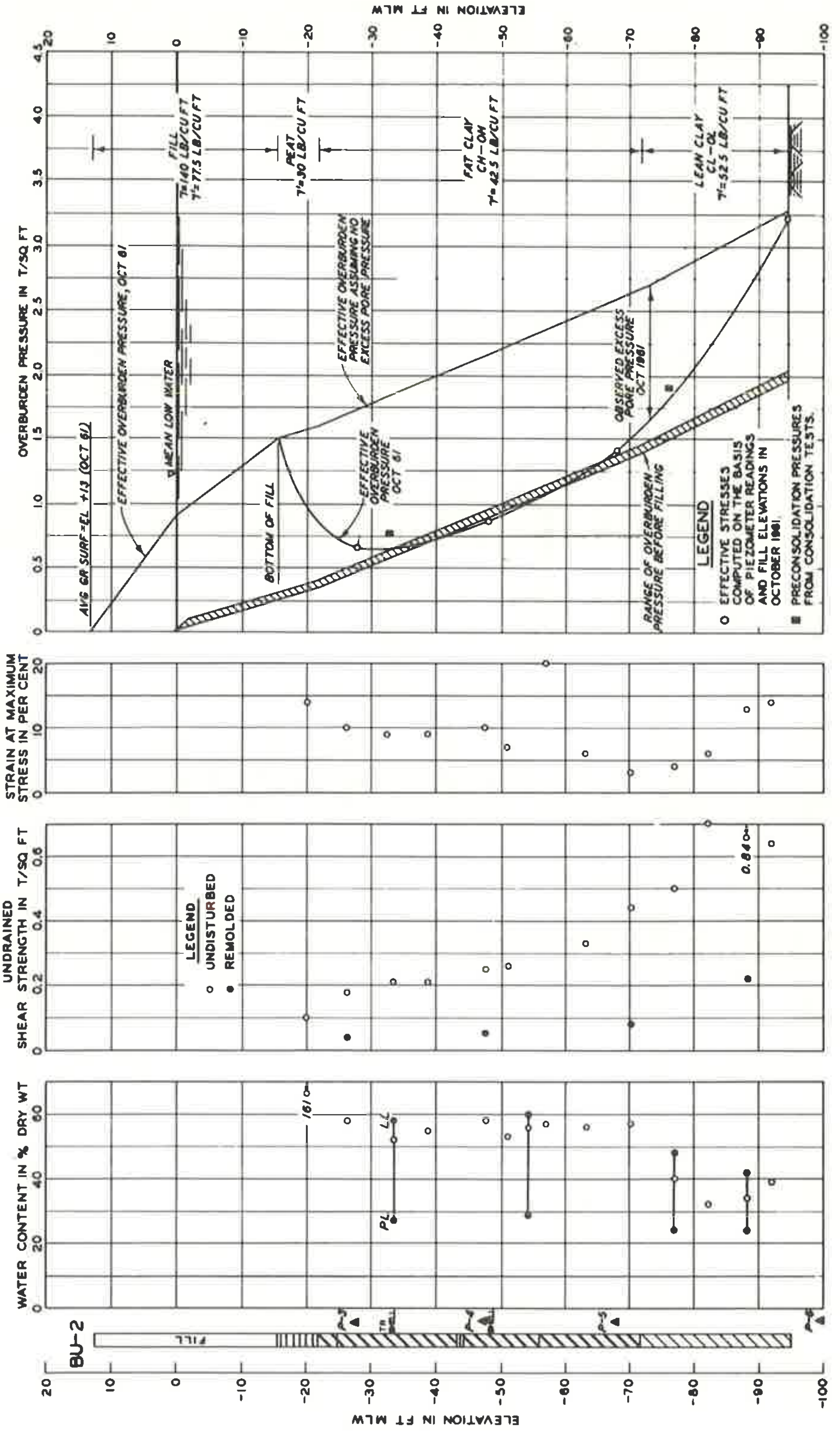


Figure 5. Soil properties versus depth.

All of the piezometers except P-6, which was installed in the fissured rock, indicated excess hydrostatic pressures. The piezometer readings varied only slightly with time after equilibrium was reached. The maximum excess hydrostatic pressure was found at piezometer P-4 near the midpoint of the stratum. At this point, the excess hydrostatic pressure was equal to 41 ft of water, approximately equivalent to the net weight of fill applied in 1960. It was concluded that practically no consolidation had occurred at the midpoint of the stratum because of the weight of the fill.

LABORATORY INVESTIGATIONS

Laboratory tests consisted of visual classification and natural water content determinations of all samples and grain-size analyses, Atterberg limits tests, and shear strength and consolidation tests on representative samples. Natural water content, plasticity characteristics, and unconfined compression test results for boring BU-2 are plotted versus depth in Figure 5. Liquid limit values indicated the estuarine deposits to be fat clays, except for the lowermost 22 ft of material, which consisted of lean clays. Oven drying tended to reduce the Atterberg limits by about 25 percent, indicating the presence of organic matter. An activity ratio of 0.94 was calculated, indicating that the soils can be classified as "normal" clays.

The undrained shear strength s_u varied from about 0.2 to 0.8 ton/ft^2 and increased with depth (Fig. 5). The strain at maximum stress generally varied between 3 and 14 percent. Effective overburden pressures based on piezometer readings were used to compute the s_u/p_o ratio as about 0.29. The sensitivity ratio based on unconfined compression tests on remolded samples averaged about 4. The drained direct shear tests indicated an effective friction angle between 30 and 35 deg.

The estimated preconsolidation pressures from the two consolidation tests are compared in Figure 5 with the computed effective overburden pressures based on piezometer readings, and the two pressures are in reasonable agreement. The close agreement indicated that foundation soils had not previously received loads greater than the existing fill. The results of the consolidation tests are shown in Figure 6. The compression indexes were 0.51 and 0.37 for samples 6 and 21 respectively. The coefficients of consolidation for pressures in the range of 1 to 2 tons/ft^2 were about 35×10^{-4} and 90×10^{-4} cm^2/sec for samples 6 and 21 respectively.

ANALYSES

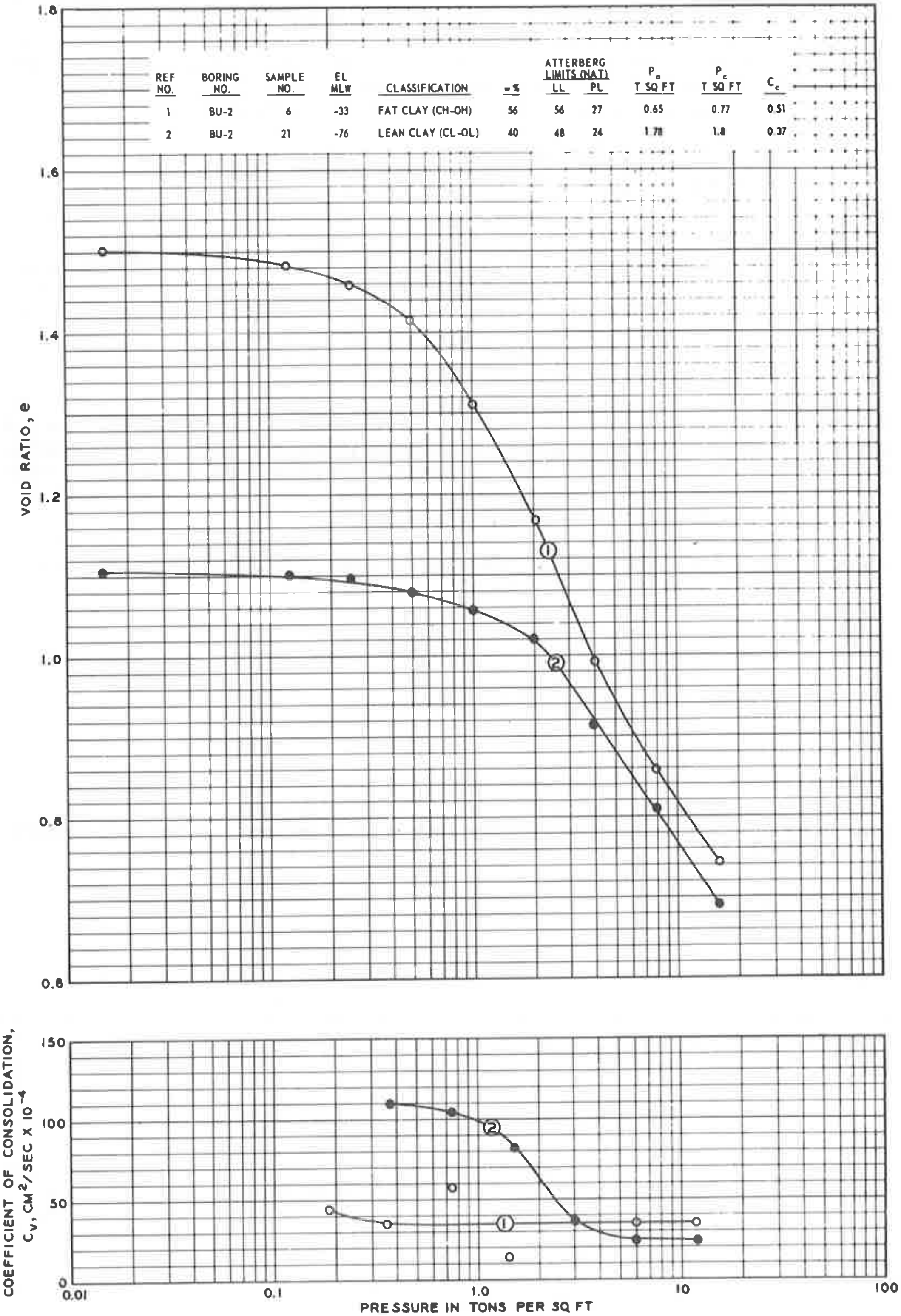
Based on the laboratory consolidation test data and assuming that drainage was taking place at the bottom surface of the fill and also at the rock surface, we computed the percentage of consolidation as of October 1961 to be 34 percent. It was also possible to check the degree of consolidation by using the pore pressure distribution curves. Based on these curves, the percentage of consolidation is equal to the area corresponding to the effective stress increase as of October 1961 divided by the area corresponding to the ultimate increment of effective stress increase. The percentage of consolidation was computed by the latter method to be 22 percent, which was considered to represent more truly the actual degree of consolidation. Stability analysis of the south fill area indicated a factor of safety of close to 1.0 with respect to failure of the riverward edge of the fill. Therefore, further placement of fill was prohibited. It was also concluded that the area could be safely used for athletic fields provided that piezometers and reference points were observed at frequent intervals to ensure that no significant movements of pore pressures were occurring.

ENGINEERING MEASUREMENT DATA

Instruments included the piezometers and reference points installed in 1961 and also four slope indicator casings. The south fill area was subsequently used for tennis courts, which afforded a means for tracing the development of surface cracks. The crack pattern, as of 1967, reflected the greater settlement of the thicker portions of the estuarine deposit toward the river.

Summary time plots through 1969 of lateral movement and settlement are shown in Figure 7. These data indicate that surface settlements are continuing, but at a decreas-

Figure 6. Consolidation test data.



REF NO.	BORING NO.	SAMPLE NO.	EL MLW	CLASSIFICATION	w %	ATTERBERG LIMITS (NAT) LL	PL	P_o T SQ FT	P_c T SQ FT	C_c
1	BU-2	6	-33	FAT CLAY (CH-OH)	56	56	27	0.65	0.77	0.51
2	BU-2	21	-76	LEAN CLAY (CL-OL)	40	48	24	1.78	1.8	0.37

Figure 7. Lateral movement and settlement data.

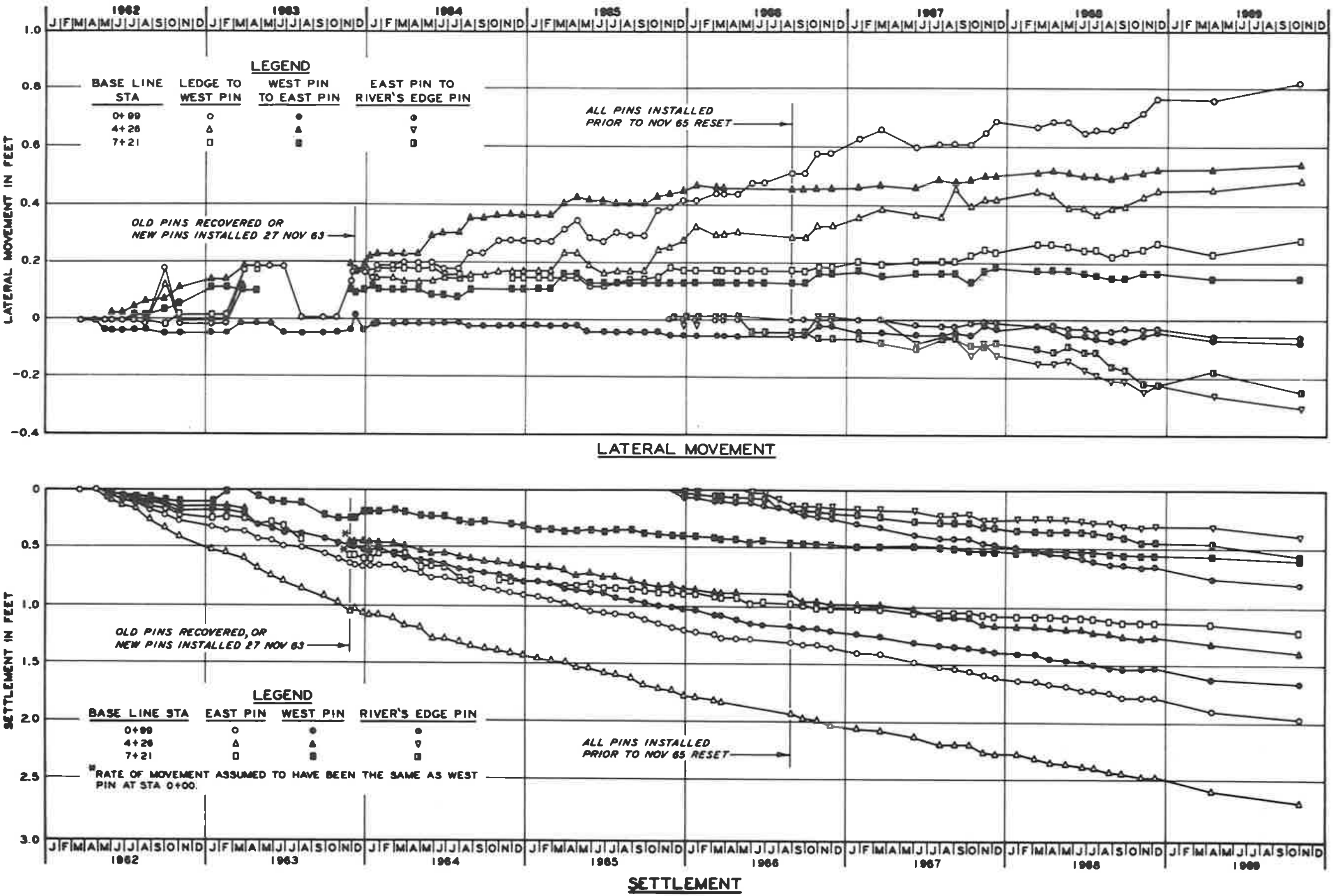


Figure 8. Distribution of foundation pore pressures.

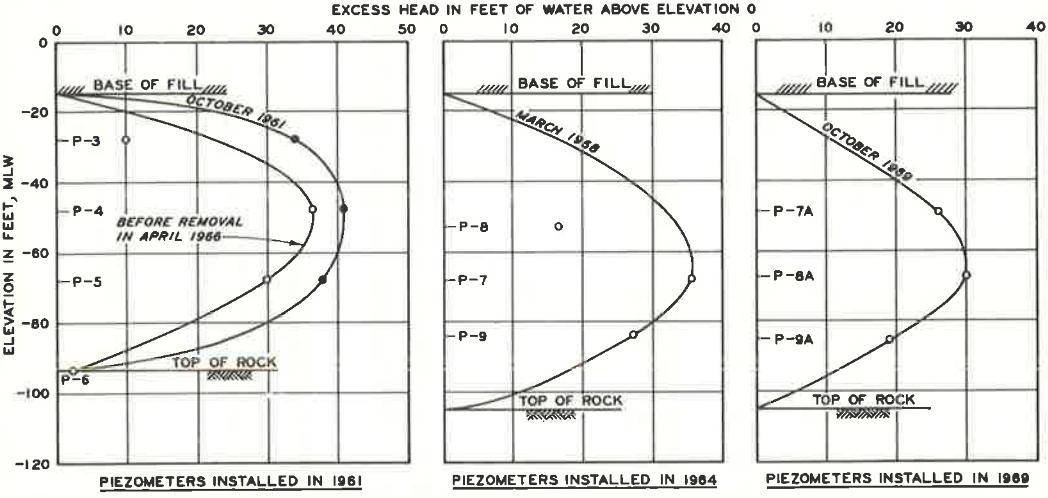


Figure 9. Pore pressure and settlement extrapolations.

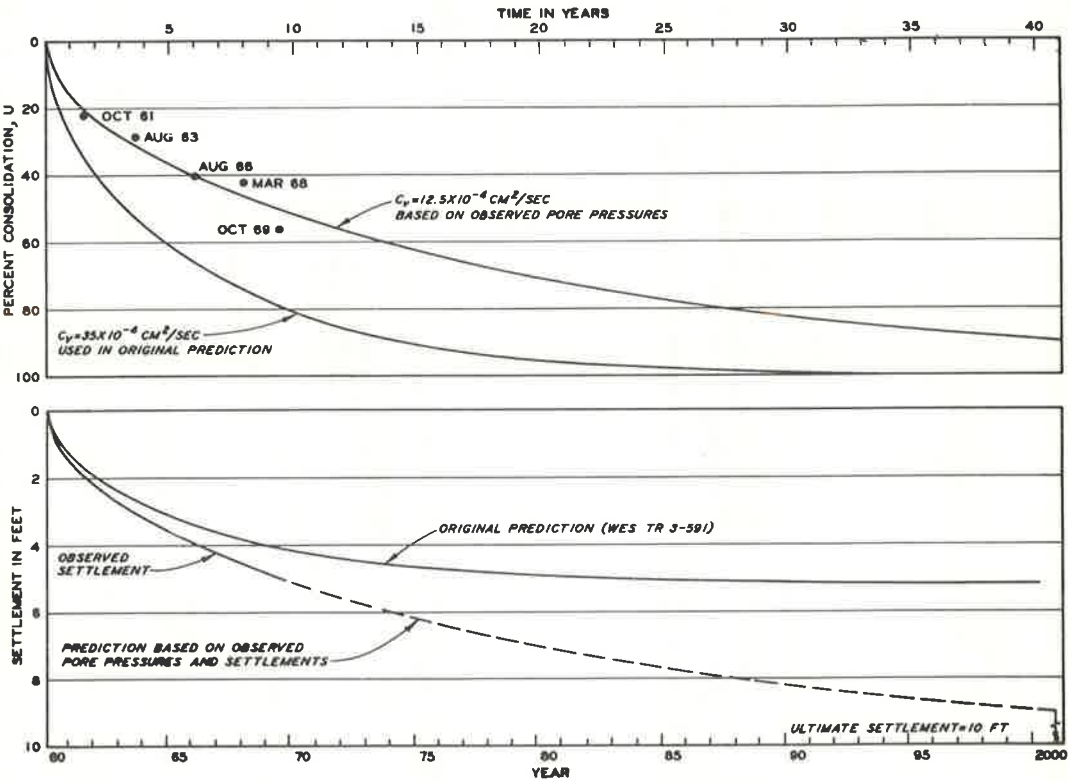


Figure 10. Summary of movement data.

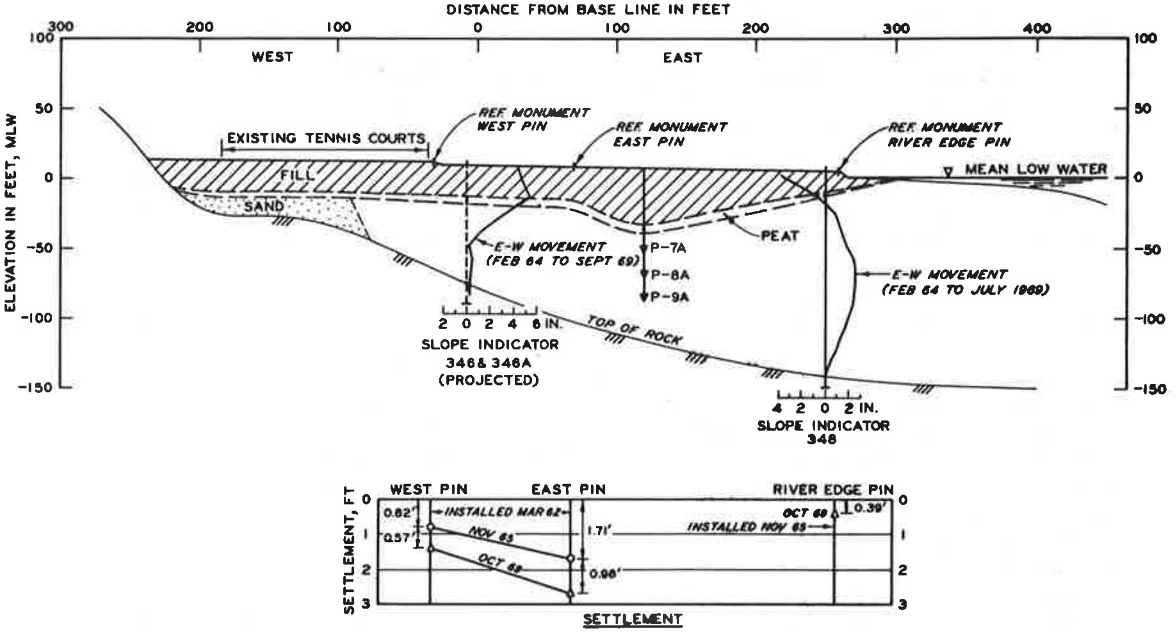


Figure 11. Movement versus time at slope indicator 346.

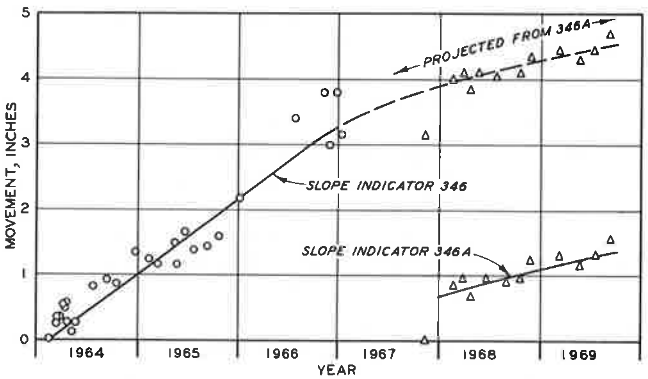
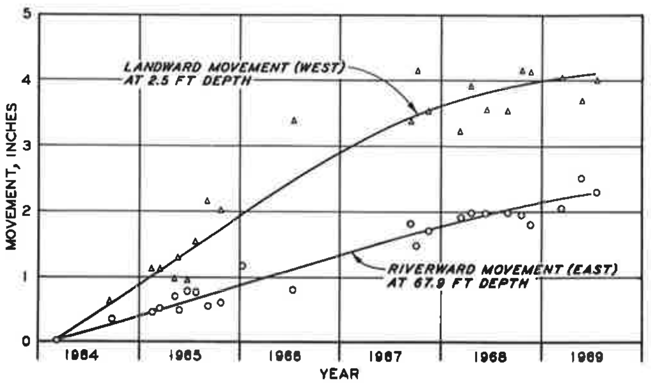


Figure 12. Movement versus time at slope indicator 348.



ing rate. The lateral distance between reference points indicates continuing extension and in some instances compression. However, no unusual movements or indications of distress are apparent.

The distribution of foundation pore pressures at selected times is shown in Figure 8. The distributions are in accordance with consolidation theory for a stratum drained at the top and bottom. Pore pressure distributions are based on observations made before obvious malfunctioning of the piezometers, which were used to compute the overall degree of consolidation within the stratum; a time plot of the degree of consolidation is shown in Figure 9. Based on this curve, it appears that about 50 percent consolidation occurred as of 1969. The coefficient of consolidation was computed to be 12.5×10^{-4} cm/sec, which is substantially lower than the value of 35×10^{-4} cm/sec estimated from limited consolidation test data and used for the original prediction of time rate of settlement. Also shown in Figure 9 is the observed settlement and the extrapolated settlements calculated on the basis of changes in the observed degree of consolidation. Calculations indicate that the ultimate settlement of the fill in the vicinity of the piezometers will be on the order of 10 ft and that it may take as long as 30 years to reach 90 percent consolidation. These predictions apply only at the piezometer installation location where the thickness of the fill is a maximum. In the vicinity of the tennis courts where the clay stratum is much thinner and less fill has been placed, settlement should be much less and should occur at a much faster rate than shown in Figure 9.

Although the average degree of consolidation for the stratum in the vicinity of the piezometers in October 1969 was 56 percent, the excess head at the center of the stratum (Fig. 8) has decreased from 41 ft to only 30 ft, resulting in a degree of consolidation at this depth of 27 percent. Nevertheless, the increase in effective pressure that has occurred at the center of the stratum since fill placement in 1960 amounts to 0.34 ton/ft^2 based on the s_u/p_o ratio 0.29. It appears that the strength has increased by about 0.10 ton/ft^2 (about one-third of its original strength) at the center of the stratum with much greater strength increases near the top and bottom. Although only a portion of the critical failure surface would be situated beneath the fill area, it was concluded that strength increases that have significantly increased the stability of the south fill area with respect to sliding have taken place.

Foundation movements as indicated by the slope indicator installations are shown in Figure 10 together with observed settlement profiles. The movement data indicate a dish-shaped settlement profile with the top of the landward slope indicator (346 and 346A) moving riverward and the top of the riverward slope indicator (348) moving landward. The vector movement at the tops of these slope indicators is corroborated by observations of horizontal movement of the surface reference monuments. The riverward movements at the top of slope indicators 346 and 346A are plotted versus time in Figure 11, which shows that the rate of riverward movement is decreasing with time. Similar data for slope indicator 348 are shown in Figure 12. The landward movement at the top of slope indicator 348 appears to be decreasing with time similar to the movement in an opposite direction indicated by slope indicator 346A. In summary, the slope indicator data demonstrated that horizontal movements are continuing at a decreasing rate with no indication of potential distress.