FIELD CONSOLIDATION OF VARVED CLAY

Report No. 3

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PUTNAM BRIDGE EAST APPROACH

General

Data from the fills for the east approach to the Putnam Bridge provided observations on 78 settlement platforms and 9 piezometer locations as shown in Figures 1 and 2. Each piezometer location contains four piezometers at various depths. Figure 1 shows the settlement platform locations in the fill and berms closest to the Connecticut River. Figure 2 shows the locations of the settlement platforms in the fill and berms around Keeney Cove. As built cross-sections and piezometer data were obtained from Goodkind and O'Dea through the Soils Division of Conn. DOT. Only two of the nine piezometer groups yielded data that could be analyzed for rate of consolidation. The other seven groups showed anomalous behavior after filling which might have been due to several causes. Fortunately the two groups that functioned in a regular manner were widely separated. One group was in the sand-drained area toward the river and the other was in the area around Keeney Cove which was not sand drained. With the data from these two piezometer groups an estimate could be made of the rate of consolidation with and without sand drains.

Initial Settlements

Initial Settlements for all the settlement platforms shown in Figures 1 and 2 are listed in Table 1. The tabulated initial settlements were determined graphically by plotting the observed settlements after the load was in place against the square root of adjusted time ($\sqrt{t'}$) as

Table I

<u>Initial Settlements From Filling of East Approach to Putnam Bridge</u>

Settlement Platform No.	Initial Settlement (ft)	Settlement Platform No.	Initial Settlement (ft)	Settlement Platform No.	Initial Settlement (ft)	Settlement Platform No.	Initial Settlement (ft)	Settlement Platform No.	Initial Settlement (ft)	Settlement Platform No.	Initial Settlement (ft)
1	0.12	15	0.39	29	0.42	42	0.12	61	0.36	76	0.10
2	0.14	16	0.49	30	0.35	43	0.08	62	0.10	77	0.05
3	0.79	17	1.20	31	0.36	44	0.0	63	0.0	78	0.0
4	0.77	18	1.65	32	0.30	45	—	64	0.0	79	0.0
5	0.49	19	1.20	33	0.54			65	0.06	80	0.05
6	0.3	20	0.58	34	0.31			66	0.15	81	0.05
7	0.0	21	0.27	35	0.0	52	0.05	67	0.16	82	0.07
8	0.24	22	0.0	36	0.10	53	0.15	68	0.0	83	0.10
9	0.49	23	0.31	37	0.40	54	0.0	69	0.0	84	0.0
10	1.32	24	0.52	38	0.25	55	0.03	70	0.51		
11	2.30	25	0.78	39	0.0	56	0.07	71	0.53		
12	1.58	26	0.70	40	0.0	57	0.0	72	0.50		
13	0.70	27	0.22	41	0.0	58	0.0	73	0.0		
14	0.21	28	0.05			59	0.0	74	0.0		
						60	0.0	75	0.16		

described in previous reports (1)(2). Many of the fills were built in stages and the fill height for each stage remained constant for several months. In these cases the graphical method was applied once for each filling sequence and the initial settlements from all stages added to obtain the total initial settlement shown in Table 1.

Plots were made to compare the relative magnitudes of initial settlements along the centerline of the fill and at two cross-sections. These plots are shown in Figures 3 and 4. Figure 3 shows the variation in initial settlements perpendicular to the centerline at Stations 82+50 and 84+00. At both cross-sections the greatest initial settlement occurs under the centerline of the fill with the amount of initial settlement diminishing rapidly toward the berms. Figure 4 shows a similar effect along the centerline of the high fill. The initial settlements are greatest under the highest part of the fill and the amount of initial settlement decreases toward the lower parts of the fill. The maximum height of the fill listed on the settlement records and used to determine initial settlements are shown by dashed lines on Figure 4. The solid line represents the height of fill shown on the as-built drawings supplied by Goodkind and O'Dea.

The general pattern of initial settlements for the East Approach to the Putnam Bridge agrees qualitatively with the theoretical analysis for initial settlements under strip loadings published by D'Appolonia et al (3). Initial settlements can be composed of two portions, elastic and plastic. The relative contribution of each depends on the ratio of applied pressure to the ultimate bearing capacity. When the ratio is low the initial settlements result from the elastic deformation of the soil. As the ratio of applied pressure to ultimate bearing capacity increases, the contribution of the plastic strains become significant and at low factors of safety causes the initial settlements to become large. The initial settlements under the high part of the fill show the effects of plastic deformation. The foundation soil is highly stressed. The berms contain the plastic flow and high initial settlement result.

Rate of Consolidation

The piezometer data from the vicinity of settlement platform number 2 showed regular dissipation of excess hydrostatic pore pressure for a period, long enough to analyze for rate of consolidation. This analysis was performed assuming that the piezometers had been properly installed midway between sand drains as required in the specifications. Settlement platform number 2 is in the area that the plans and specifications indicate were serviced by 12" diam. sand drains on 10 foot centers. The rate of consolidation analysis was carried out in the following steps:

1. Approximating the load due to the fill by rectangular loaded areas.
2. Estimating the initial excess pore pressures at the piezometers from the computed stresses using the equation $u_o = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3}$; where $\Delta\sigma_1$, $\Delta\sigma_2$ and $\Delta\sigma_3$ are the principal stresses caused by the fill and u_o is the initial excess pore pressure.
3. Estimating the percent consolidation at the piezometers by comparing the piezometer readings to the initial excess pore pressure.

4. Determining the average percent consolidation for the layer as a whole from the consolidation at the piezometers.

The ratio of observed excess hydrostatic pore pressure to computed initial excess pore pressure on the centerline was used to evaluate a time factor T_h from Figure 7 in a paper by Johnson (4). These time factors were then used to determine average percent consolidation from the graphs published by Schiffman (5). This procedure was used once for each selected reading at each piezometer. An average percent consolidation for the entire clay layer was determined from a weighted average of all the piezometers.

This analysis yielded an apparent coefficient of consolidation in the radial direction $C_{ra} = 0.18 \pm 0.02$ ft^2/day. This value of C_{ra} was checked against the rate at which settlements progressed at adjacent settlement platforms and was found to predict settlements accurately. The coefficient of consolidation showed no decrease for the times after all of the load was in place. No way has yet been found to determine the rate of consolidation during filling.

The settlements of platforms numbers 11 and 33 were predicted using the value of C_{ra} found by this analysis and the computed values agreed with the observed settlements indicating that the value of the coefficient of consolidation in the radial direction is reasonable.

The results of the analyses for settlement platforms numbered 2, 11 and 33 are shown in Table II. As can be seen from Table II the initial settlements determined by the two methods agree. The higher values of C_{ra} were used in the analysis of the settlements from the equation:

$$\rho = \rho_i + U \rho_c \tag{1}$$

Table II

ANALYSIS OF AMOUNT AND RATE OF SETTLEMENT FOR

a.) Areas Serviced by Sand Drains

Settlement Platform No.	Initial Settlement (ft.) $\sqrt{t'}$	$\rho = \rho_i + U \rho_c$	Consolidation Settlement ρ_c (ft.)	Ultimate* Settlement ρ_t (ft.)	Max. Observed Settlement (ft.) (Date)	C_{ra} ft^2/day
2	0.14	0.27	2.58	2.85	2.7 (Oct. 1958)	0.18
11	2.30	1.96	3.44	5.40	5.1 (Dec. 1958)	0.20
33	0.54	0.61	1.98	2.59	2.51 (Oct. 1958)	0.20

b.) Area Not Serviced by Sand Drains

Settlement Platform No.	Initial Settlement (ft.) $\sqrt{t'}$	$\rho = \rho_i + U \rho_c$	Consolidation Settlement ρ_c (ft.)	Ultimate* Settlement ρ_t (ft.)	Max. Observed Settlement (ft.) (Date)	C_{ra} ft^2/day
66	0.51	0.22	1.65	1.87	1.31 (Dec. 1958)	5.4**

*Ultimate settlement is the sum of ρ_i from $\rho = \rho_i + U \rho_c$ and ρ_c.

**This is the apparent coefficient of consolidation, C_{va}.

The area filled near Keeney Cove contains no sand drains. In this area one set of piezometers, in the vicinity of settlement platform number 66, showed regular dissipation of excess hydrostatic pore pressures. The fill was again approximated by rectangular loaded areas and stresses computed. One difference between this area and the one serviced by sand drains is that the initial excess over hydrostatic pore pressures were computed by the relation $u_o = B\ [\Delta\sigma_3 + A\ (\Delta\sigma_1 - \Delta\sigma_3)]$; where A and B are the pore pressure parameters as defined by Skempton (7) and $\Delta\sigma_1$, $\Delta\sigma_3$ and u_o are as defined before. The parameter B was assumed equal to 1.0 and comparison of computed values of pore pressure with piezometer data indicated a reasonable value for A to be 0.88. These values were used in the analysis. Using different methods of computing initial excess pore pressures under high and moderate fills may seem at first glance to be questionable but a similar approach has been previously used with field data (6).

The analysis of data from settlement platform number 66 yielded a value of C_{va} equal to 5.4 ft^2/day. This value compares to the apparent coefficient of consolidation found for the fills at the Bissell Bridge and reported previously (2). Results for this settlement platform are summarized in Table II.

The Apparent Effect of Sand Drains

The analysis from the observations of the settlement behavior of the East Approach to the Putnam Bridge cannot be conclusive because only two sites yielded results. From this limited data (one sand-drained piezometer group and one non-sand-drained piezometer group) some preliminary computations can be made to check the effect of sand drains. The probable progress of consolidation at the East Approach are plotted in Figure 5. All the curves shown in Figure 5 were computed assuming a construction time of three months. The lowest curve which indicates the fastest rate of consolidation was computed on the assumption of sand drains one foot in diameter, 10 feet on center servicing a soil with a constant coefficient of consolidation in the radial direction of $C_{ra} = 0.2\ ft^2/day$. The middle curve was computed for a double drained clay layer 100 ft thick having an apparent coefficient of consolidation $C_{va} = 5.0\ ft^2/day$ and containing no sand drains. The highest curve representing the lowest rate of consolidation is for a non-sand drained area having a double drained clay 100 feet thick and $C_{va} = 1.0\ ft^2/day$. The highest curve is included for comparison and does not reflect conditions at the Putnam Bridge.

The two lower curves represent conditions in the East Approach to the Putnam Bridge. As can be seen from Figure 5, the sand drained areas had about 96% consolidation at the end of one year; the non-sand drained areas, an average consolidation of 82% after the same length of time.

The sand-drained area was probably well into the virgin compression range during its process of consolidation. The maximum fill over the Keeney Cove region was much lower and the apparent coefficient of consolidation determined from the hydrostatic excess pore pressure dissipation is in the range of C_{va} found at Bissell Bridge. No predictions of the rate of consolidation under a high fill without sand drains can be made from the data presently available.

The Coefficient of Volume Compressibility

The consolidation settlements for three additional settlement platform in the vicinity of the high fill were computed. These computations were made using the technique of Schiffman (5). The results are shown in Table III. The coefficients of volume compressibility were computed from the equation:

$$\rho = M_v \, \Delta\sigma \, h \qquad (2)$$

where ρ is the settlement, M_v is the coefficient of volume compressibility h is the thickness of the clay layer and $\Delta\sigma$ is the increase in vertical stress at mid-depth of the clay.

The coefficient of compressibility was determined two ways: on the basis of consolidation settlement only and on the basis of total settlement. The results are shown in columns 6 and 7 of Table III. The values are greater than the coefficients of compressibility at the East Hartford Expressway on the Bissell Bridge.

PUTNAM BRIDGE WEST APPROACH

Original Approach

No information could be extracted from the data for the original West Approach to the Putnam Bridge. The piezometers in the vicinity of settlement platforms had a high mortality rate. Many piezometer locations contained only one piezometer, which is not sufficient for making an analysis.

The settlement observations on the platforms were not reported with sufficient frequency to allow square root of adjusted time plots. No

Table III

Settlement Summary

East Approach Putnam Bridge

Settle. PL No.	ρ_i (ft)	ρ_c (ft)	ρ_t* (ft)	Boring No.	M_v ft^2/Ton Consolidation Settlement Only	M_v ft^2/Ton Total Settlement	Ht. of Fill
River Side of Fill							
2	0.27	2.58	2.85	E-8	2.5	2.8	24'
11	1.96	3.44	5.40	E-9	2.4	3.6	60'
33	0.61	1.98	2.59	E-7	2.0	2.6	21'
18	1.77	2.53	4.30	E-13	1.9	3.2	57'
6	0.59	2.52	3.11	E-9	2.4	3.0	25'
4	0.60	4.24	4.84	E-8	2.8	3.2	43'
Keeney Cove							
66	0.22	1.65	1.87	B-1	1.1	1.2	31'

* $\rho_t = \rho_i + \rho_c$

information was obtained, therefore, on initial settlements.

Additional Fills Under Projects 159-90 and 118-68

The data from the Soils Division of Conn. DOT from projects 159-90 and 118-68 were analyzed for apparent coefficient of consolidation, and initial and consolidation settlements. Since the filling of this area occurred in two stages separated by approximately six to seven months, the analysis treated the data in two parts.

1. Initial Settlements

The observed settlements were analyzed by the square root of adjusted time. The results are shown in Table IV. The first phase of filling, which was about 10 to 12 feet at most settlement platforms, caused little initial settlement. The second phase of filling which took the fill to 40 feet in some cases caused larger initial settlements.

2. Rate of Consolidation

Apparent coefficients of consolidation were determined for piezometer groups 1, 2 and 3 on project 118-68. The data for the other piezometer groups could not be analyzed. The analysis was carried out in two parts, one for each stage of filling. The stresses computed using ICES-SEPOL and the loads indicated by the cross-sections yielded hydrostatic pore pressures that were much smaller than the pore pressures indicated by the piezometer readings. The initial hydrostatic excess pore pressure isochrone was therefore estimated from the piezometer response to filling. The reason for the difficulty with the pore pressures is not completely understood

Table IV

Initial Settlements for Fills Placed
For Projects 159-90 and 118-68

Project No.	Settle. PL No.	ρ_i (ft)) 1st Stage	ρ_i (ft) 2nd Stage	ρ_i (ft) Total	Remarks
118-68	X-1	--	--	--	Unknown Loading
	X-2	--	--	--	Unknown Loading
	X-3	0	0.07	0.07	
	X-4	0	0	0	
	X-5	0	0.10	0.10	
	X-6	0	0.05	0.05	
	X-7	0	0	0	
	X-8	0	0.13	0.13	
	X-9	0.23	0.23	0.46	
	X-10	0	0.13	0.13	
	X-11	--	--	--	Data Incomplete
159-90	X-1	--	--	--	Data Incomplete
	X-2	0	0	0	
	X-3	--	--	--	Unknown Loading

but might be due to an error in platform location with respect to the fill. The recorded fill height did not always agree with the cross-sections. Another difficulty arose due to the variation of the thickness of the clay layer in the vicinity of the piezometer groups. The closest "complete" borings were too far from the piezometer groups and the bottom of the clay had to be estimated from the curve generated by piezometer readings. As a result, the values extracted from this data show some inconsistencies.

The apparent coefficient of consolidation was determined in the usual way (1). The results are shown in Table V. The ratio of thickness of clay to width of fill is small in this area which results in the low values of C_{va} shown in Table V.

The method of analyzing C_{va}, because it involves differences in percent consolidation, is insensitive to small errors in initial hydrostatic excess pore pressure and clay thickness. The values of initial settlement determined from the Equation 1 appear to be sensitive to these errors because it includes the average percent consolidation. This may be one of the reasons that the initial settlements in the vicinity of piezometer groups 2 and 3 appear negative. The total settlements as shown in Table V appear to be approximately correct indicating that Eq. 1 may compensate between the two types of settlement.

Table V

Analyzed Data from Piezometer R adings Project 118-68

Piez. Gp.	Bore Hole No.	Filling Stage	C_{va} ft^2/day	ρ_c (ft)	ρ_i (ft)	Indicated ρ_t (ft)
1	S-19	1	0.58	0.49	0.02	
		2	0.56	0.62	0.60	
				1.11	0.62	1.73
2	S-19	1	1.46	0.09	0.0	
		2	0.82	0.80	-0.25	
				0.89	-0.25	0.64
3	S-14	1	0.61	0.27	-0.04	
		2	0.42	0.93	0.0	
				1.20	-0.04	1.16

ADDITIONAL ANALYSES ON EAST HARTFORD EXPRESSWAY

General

Although the data from each construction site is being examined as completely as possible before the report is written, certain aspects require additional analysis. As additional information and methods are found the appropriate data will be reexamined.

Computed Consolidation Settlements

Several fills on the project concerning the East Hartford Expressway were not of a plane strain configuration. As a result the three dimensional character of the loading had to be taken into account when computing the consolidation settlement. The loading imposed by the fill was approximated by a large number of rectangular loaded areas. The number of loaded areas used to approximate each fill was about 50. The stress distributions were computed using the STRESS portion of ICES-SEPOL.

The consolidation settlements were computed using Equation 2 and assuming $M_v = 0.07\%\ ft^2/T$.

The results of these computations are shown in Table VI. As can be seen from the results listed in Table VI, the observed consolidation settlements appear to be slightly higher on the average than the computed settlements. The average modulus of volume compressibility based on the results shown in Table VI is $M_v = 0.85\%\ ft^2/T$. Also shown in Table VI are the estimated settlements that appear on the drawings for the settlement platforms indicated. There is not much difference between the estimated settlements from the drawings and the computed settlements from ICES-SEPOL indicating that the hand calculations of consolidation settlement are

Table VI

Comparison of Computed and Observed Consolidation Settlements from the East Hartford Expressway

Settlement Platform No.	Computed Consolidation Settlement (ft.) $\left[\bar{M}_v = 0.7\%\ ft^2/Ton\right]$	Observed Consolidation Settlement (ft.)	Ratio Obs/Comp.	Ht. of Fill (max.)	Est. Settlement From Drawings (ft.)
2	0.73	0.55	0.75	24	0.69
3	1.61	2.78	1.72	38	1.37
4	0.75	1.11	1.41	20	0.55
5	0.39	0.33	00.85	15	--
6	0.48	0.72	1.50	15	--
7	0.65	0.69	1.06	38	--
8	1.29	1.52	1.18	36	1.13
12	0.58	0.52	0.90	38	0.61
13	0.41	0.68	1.66	30	--

adequate. The largest differences occur for settlement platforms 2, 3 and 4. This is an area of very complicated embankment configuration.

Checks were run using the STRESS portion of ICES-SEPOL to determine the region of influence that most strongly affects the computed stresses. The results showed that vertical stresses below a point on a layer 100 ft thick are not significantly affected by loads more than 200 feet away.

Undrained Young's Modulus (E_u) Indicated by Field Initial Settlements

Several of the fills from the East Hartford Expressway closely approximated the plane stress case which is the theoretical model used by D'Appolonia et al (3). The initial settlements determined from the field data usually consists of an elastic and a plastic portion. By using the charts in the above-mentioned paper, the undrained Young's modulus (E_u) can be backfigured by hand.

To carry out the computations the undrained strength of the varved clay was assumed equal to 890 $\#/ft^2$ which is an average value taken from available reports. The percent of the initial settlement due to elastic shear strains was estimated from the charts and the E_u determined from the settlement portion of ICES-SEPOL. The average value of E_u for the fills of the East Hartford Expressway was found to be 190 T/ft^2 (3.8×10^5 $\#/ft^2$).

This value of E_u is only a first approximation. A more refined analysis would include anisotropic strength properties of the varved clay which must be determined by additional testing. The coefficient of lateral pressure at rest is another property of the varved clay that is not yet known but will be required for a more refined analysis.

LABORATORY TESTING OF VARVED CLAYS

Triaxial Tests

One of the objectives of the project is to correlate laboratory test data with field behavior. A series of tests were run on shelby tube samples from the vicinity of settlement platform 7 East Hartford Expressway to see if the undrained stress-strain curve from triaxial testing yielded an undrained modulus close to that backfigured from the field. Most of the triaxial tests were run consolidated undrained; two were run unconsolidated undrained. The test results are shown in Table VII. None of the samples tested showed an undrained Young's modulus close to the value backfigured from the field values of initial settlement. As can be seen from Table VII most of the laboratory values are about one order of magnitude lower than values indicated from the field data.

Predicting initial settlements from laboratory undrained stress and strain curves appears to be a common problem (3)(6)(7). The laboratory modulus is generally too low. The discrepancy is attributed to disturbances of the soil by common tube sampling techniques. Raymond (6) has investigated various sampling tubes and testing sequences to obtain the best method of evaluating E_u for Canadian clays. He correlated his test results with the behavior of a test fill. The varved clay samples at depths of 46, 47, and 50 feet were tested according to one of Raymond's methods. His technique does not appear to work on varved clay from the Connecticut River Valley.

Table VII

Summary of Undrained Young's Modulus from Triaxial Tests

Depth of Sample ft.	Chamber Pressure T/ft^2	Final Water Content (Percent)	Undrained Young's Modulus E_u T/ft^2	Remarks
41	2.9	38	21	CU
42	2.9	37	21	CU
43	2.9	41	4	CU
44	2.9	37	21	CU
46	1.4	--	7	CU
46	2.9	--	7	UU
46	4.3	37	7	UU
47	1.4	--	14	CU
47	2.9	--	24	CU
47	4.3	38	32	CU
48	2.9	37	11	CU
50	1.4	--	20	CU
50	2.9	--	43	CU
50	4.3	32	86	CU

CU - Consolidated undrained test
UU - Unconsolidated undrained

Permeability Tests

Six permeability tests were run on samples of varved clay from the same location (settlement platform 7). The samples were reconsolidated to their overburden stress before testing. The permeability was measured in the horizontal direction. The average permeability measured was 1.7×10^{-7} ft/min. This value of permeability does not appear high enough to allow the rate of consolidation observed from field data. The high values of C_{va} observed in the field may be due to drainage from the clay through the silt varves.

PROPOSED TEST FILL

Limitations of Data Presently Available

Although the analysis of the field data presently available has yielded consistent results, several questions will remain unanswered when this project is completed. In the past, field observations were made primarily to control construction. While the available data have yielded much information, test fill with well planned instrumentation is needed to close some of the gaps in our present knowledge. The regularity of the results to date indicate that information from a test fill will allow better design procedures to be developed.

The following are among the soil behavior parameters that require further information from field sites:

1. The amount and rate of secondary compression.
2. The most probable value of the undrained Young's modulus (E_u) to use in design.

3. The rate of consolidation that the varved clay experiences under high fills (60 ft).

Configuration of the Test Fill

One possible configuration of the test fill is shown in Figures 6 and 7. The fill is long enough so that the behavior in the vicinity of the center of the fill approaches the plane strain case. Many theoretical solutions for the plane strain case are available so that the pertinent stress-strain parameters can be easily defined. Piezometers and settlement platforms should be set at various depths and locations to acquire necessary information. Settlement platforms shown in Figure 6 are at the original ground surface or on the top of the clay layer. Other arrangements might be worked out with settlement posts anchored at various depths in the clay. The piezometers shown in Figure 6 are arranged to monitor the dissipation of hydrostatic excess pore pressure in the horizontal direction.

Information Desired from a Test Fill

1. Secondary Compression

Secondary compression is an important consideration because excessive post-construction settlement can foster pavement deterioration and increase maintenance costs. In the vicinity of abutments, settlements due to secondary compression adversely affect the riding qualities of the roadway especially where the bridge deck meets the pavement supported by fill. Recent profiles run on the East Approach of the Putnam Bridge by Carmen De Vito indicate that the settlements of the fills are continuing at about one in. per year.

Little information on secondary compression can be extracted from data presently available. In the past, settlement observations were seldom made after the highway was completed. Secondary compression continues after pore pressures have dissipated. The Casagrande-type piezometers have a limited life and seldom function long enough to determine when the hydrostatic excess pore pressures become negligible. The problem of the shortness of useful piezometer life is common. In recent years other types of piezometers such as the vibrating wire and air activated piezometers have been developed which function for longer times after installation. Using these types of piezometers and settlement observations over a longer period of time, the amount and rate of secondary compression can be evaluated. Better understanding of secondary compression will lead to better control of settlements and reduced pavement maintenance costs.

2. The Undrained Young's Modulus E_u

Only in the last few years have investigators treated the portion of the total settlement in clays that occurs without volume change from shear strains and contained plastic flow. This component of settlement known as initial settlement is evident in the field data already analyzed. The difficulty in handling initial settlements in design is the evaluation of the undrained Young's modulus, E_u.

Under the present project, values of E_u were backfigured from initial settlements extracted from the field data and compared to values of the same parameter determined from laboratory triaxial tests. The average value of E_u from the field data is about one order of magnitude greater than from the laboratory samples. Similar discrepancies have been found at other locations on other clay deposits (8)(9). The difficulty appears to be soil disturbance from sampling. The best way to elucidate this parameter is to make observations on a fill built at a controlled rate (6).

If the values of E_u determined from laboratory tests were used in design, the predicted amounts of initial settlements will be too large. An accurate prediction of initial settlements will allow all settlements to be more closely estimated. Large initial settlements indicate the formation of a plastic zone in the foundation clay and are probably related to secondary compressions experienced at later times. Understanding initial settlements will help to estimate post-construction settlements and lead to reduced maintenance costs. The present value of E_u from the field data shows some scatter which may be partially due to uncertainties about the filling rate. A test fill with a controlled filling rate is needed for a better estimate. New field testing methods might prove helpful in evaluating E_u. The menard pressure meter is capable of measuring E_u in a bore hole. The initial settlement of the test fill will be compared with estimates based on E_u's from the data analyzed on this project, triaxial tests and pressure meter tests to determine the best design parameter.

3. The Rate of Consolidation under High Fills

An interesting, although somewhat expected, effect of loading geometry on the rate of consolidation was indicated by analysis of the available field data. The coefficient of consolidation determined from laboratory tests in the oedometer was about 0.2 ft^2/day in the recompression range. The coefficient of consolidation indicated by the field rate of consolidation was found to be as great at 5.0 ft^2/day; an increase of 25. The laboratory sample is constrained so that the dissipation of excess pore pressure takes place in the vertical direction only. In the field the dissipation of hydrostatic excess pore pressures can occur in more than one direction and an increase in rate of consolidation results.

Under high fills (60') the varved clay is stressed into the virgin compression range. Laboratory rates of consolidation in this range decrease by nearly one order of magnitude. Data for comparing the laboratory rate to field rates under this magnitude of stress are not available. The decrease observed in the laboratory is attributed to a change in structure and a decrease in permeability. Whether these two changes occur in a similar manner in the field is unknown. Only one site (Putnam Bridge East) contained fills up to 60 feet high. At this site sand drains spaced 10 feet center to center were used under the high part of the fill. The piezometer data from this site was sketchy since most of the piezometers malfunctioned. Sand drains are expensive to install, but help to accelerate consolidation in some circumstances. To better elucidate if and when sand drains are needed in varved clay, a value of the coefficient of consolidation under high fills without sand drains must be determined in the field. This rate will then be compared to the apparent rate with sand drains to evaluate the post-construction settlements. The elimination of sand drains from under one high fill would probably balance the cost of the entire test fill.

Recently Developed Analytical Techniques

In the past few years settlement analysis has developed by leaps. Perhaps one of the biggest advances is the use of finite element models to predict hydrostatic excess pore pressures and settlements. These programs are available from M.I.T. and can be modified to meet our needs with a test fill. Good agreement was found between predicted and observed values in the field sites where these methods were used (3)(6)(8). At the time that the field data from the varved clays of the Connecticut River Valley were gathered, the development of these techniques could not be foreseen. Their development allows better planning of test fills because they provide greater insight into the process of settlement. In light of these recent developments, ICES-SEPOL, which we are presently using, has definite limitations in predicting stresses and strains accurately under all conditions.

Some sites have a layer of sand over the varved clay. The settlement platforms have been placed at the natural ground level so that in some cases the initial settlement is partially caused by the immediate compression of the sand. In a test fill instruments for measuring settlements should be anchored at various depths in the clay layer to check the contributions of various strata below the surface to settlement. From these settlements a better understanding of how the stresses are distributed will also be determined. The stress distribution will be checked from piezometer readings on line as shown in Figures 6 and 7. The final configuration and instrumentation of the fill will be worked out with the Soils Division of Conn. DOT.

The fill shown in Figure 6 is only 40 feet high and probably will not give all the information necessary for rate of consolidation under high fills. To build a fill 60 feet high requires the addition of berms for stability. An alternate means of determining this information is by electro-osmosis in the field. The application of a direct current to a clay simulates the consolidation conditions created by a fill (10). The rate of settlement under electro-osmotic treatment proceeds according to the hydraulic properties of the soil. One advantage of using electro-osmosis is that a fill 60 feet high could be simulated without berms since the stresses induced by electricity cannot lead to shear failure.

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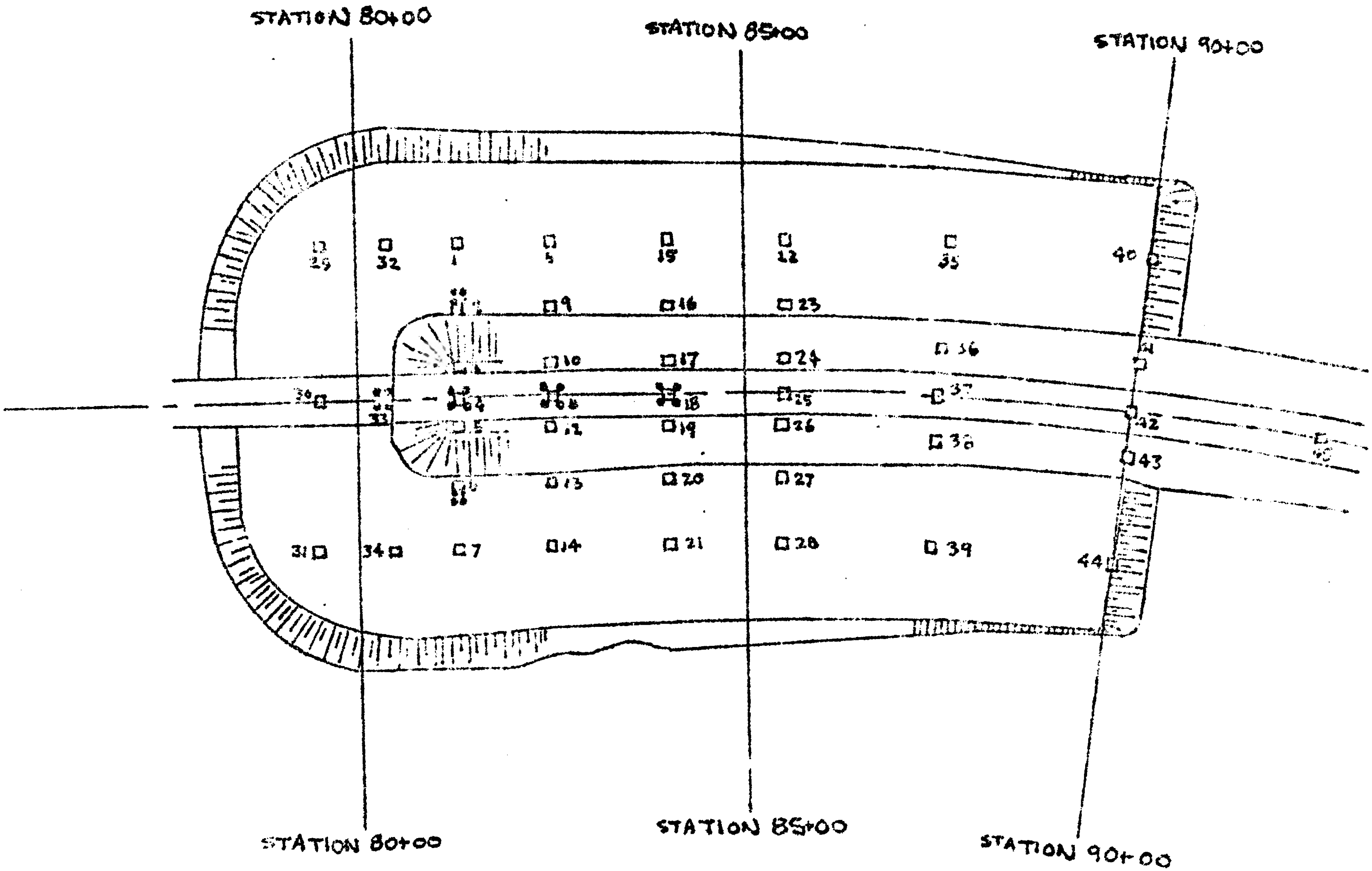
LEGEND FOR DIAGRAMS



STATION 80+00
STATION 85+00
STATION 90+00
STATION 80+00
STATION 85+00
STATION 90+00

FIGURE 1

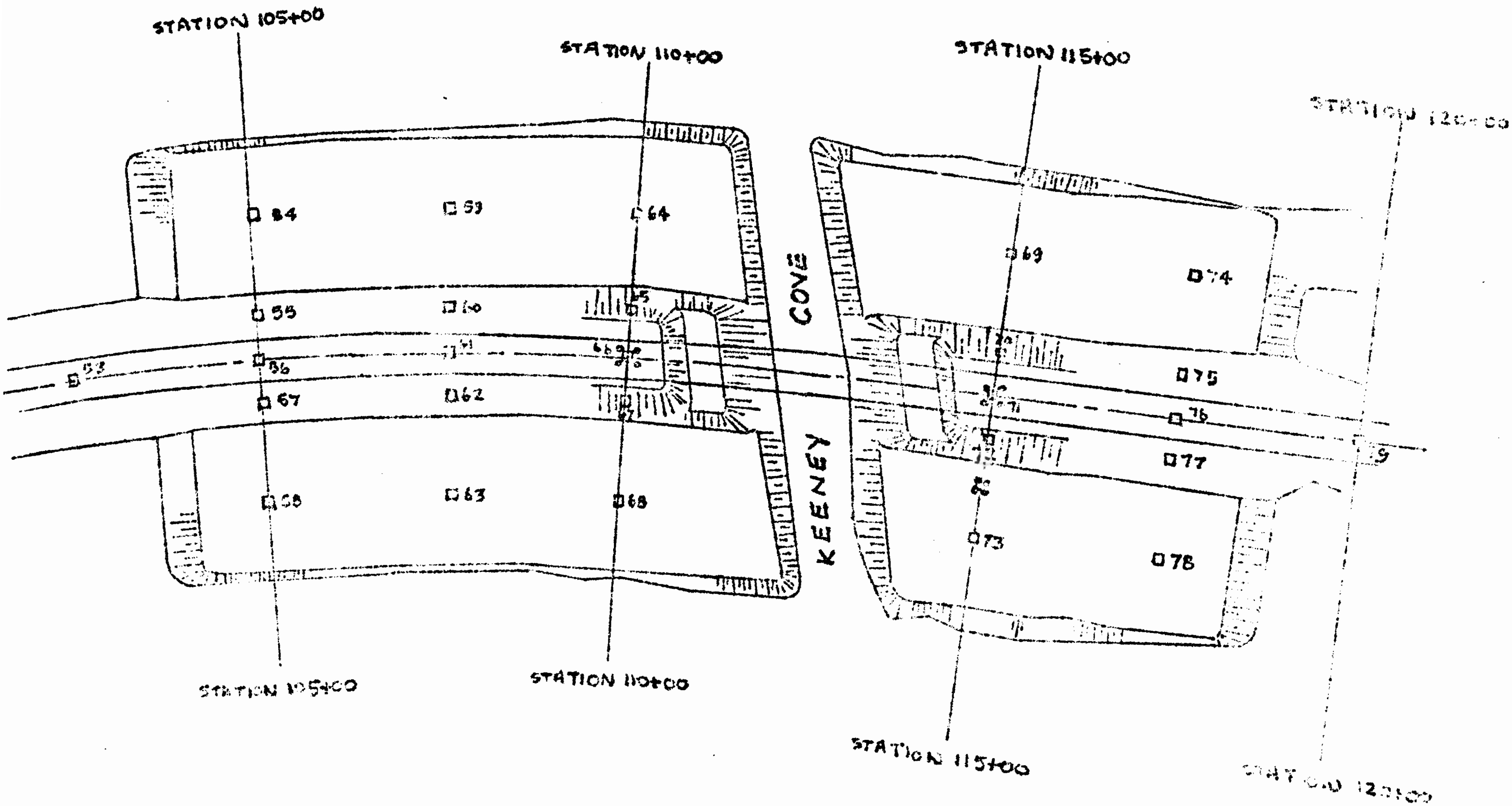
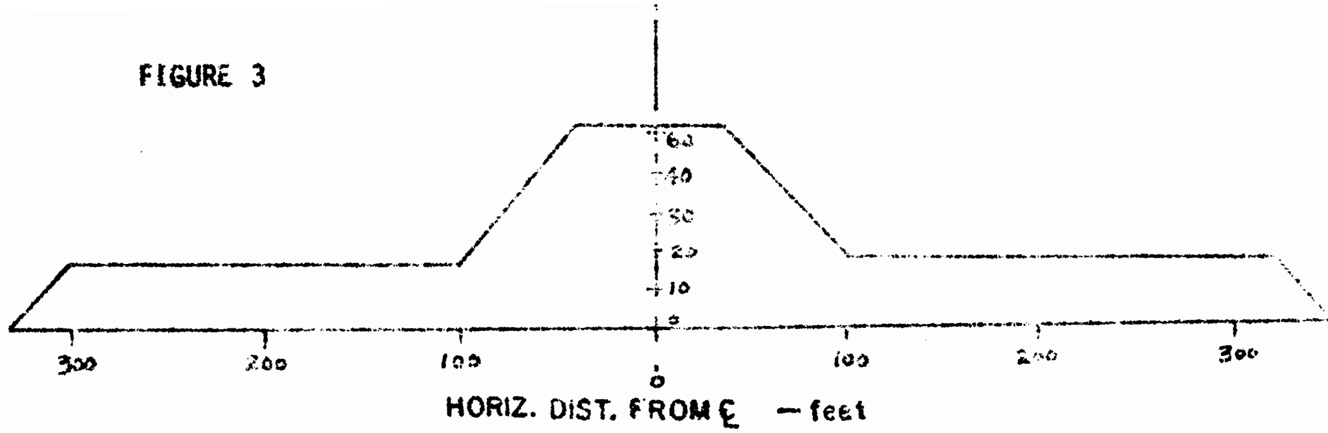
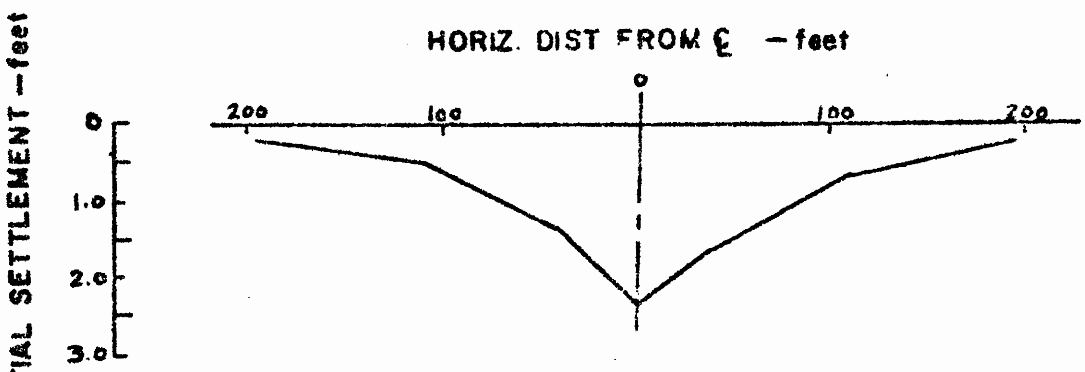

STATION 105+00
STATION 110+00
STATION 115+00
STATION 120+00
COVE
KEENEY

FIGURE 2

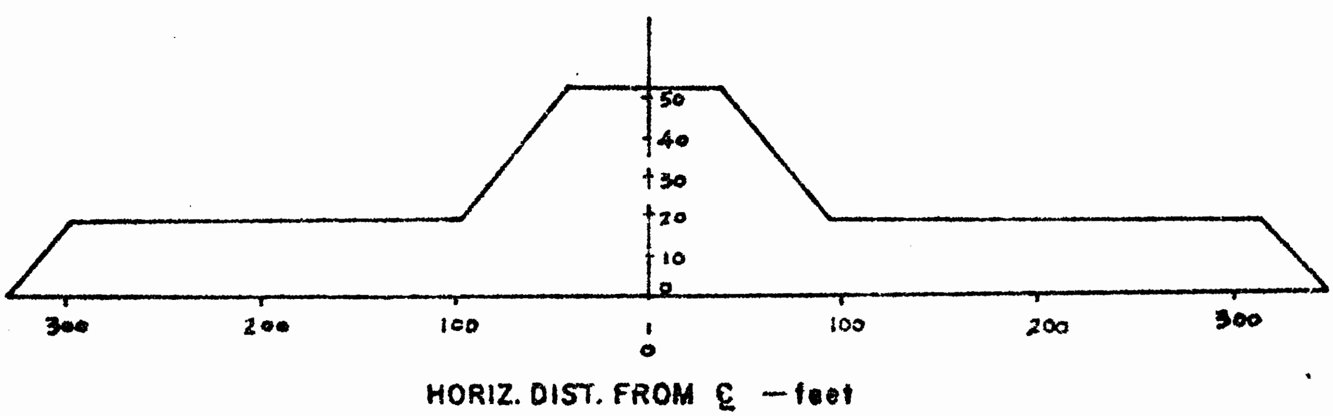
FIGURE 3



CROSS SECTION



STATION 82+50



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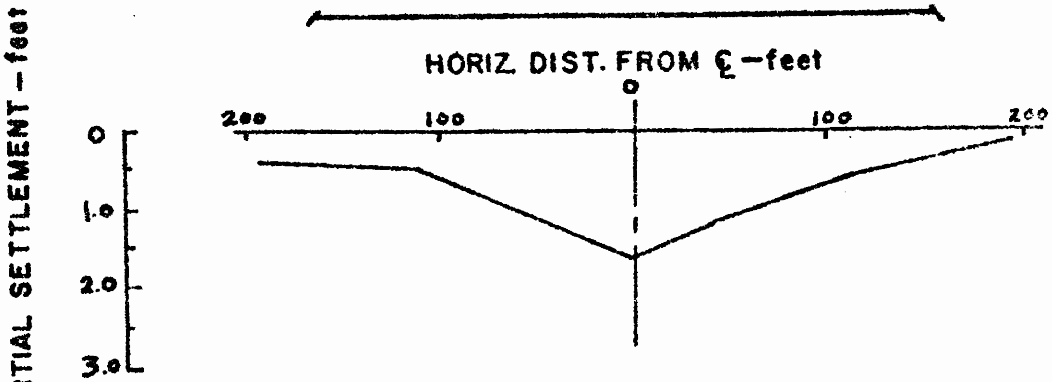


STATION 84+00

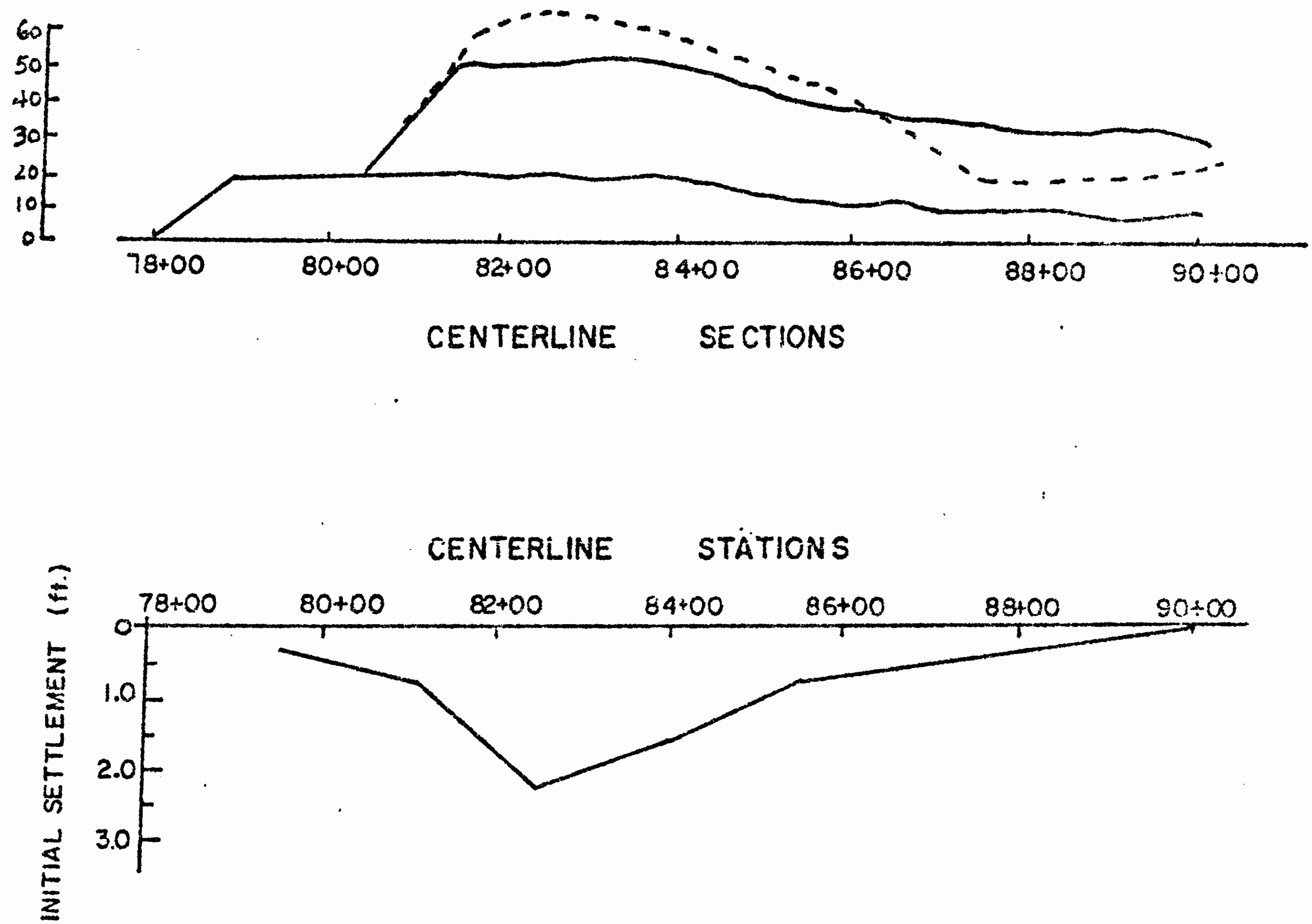
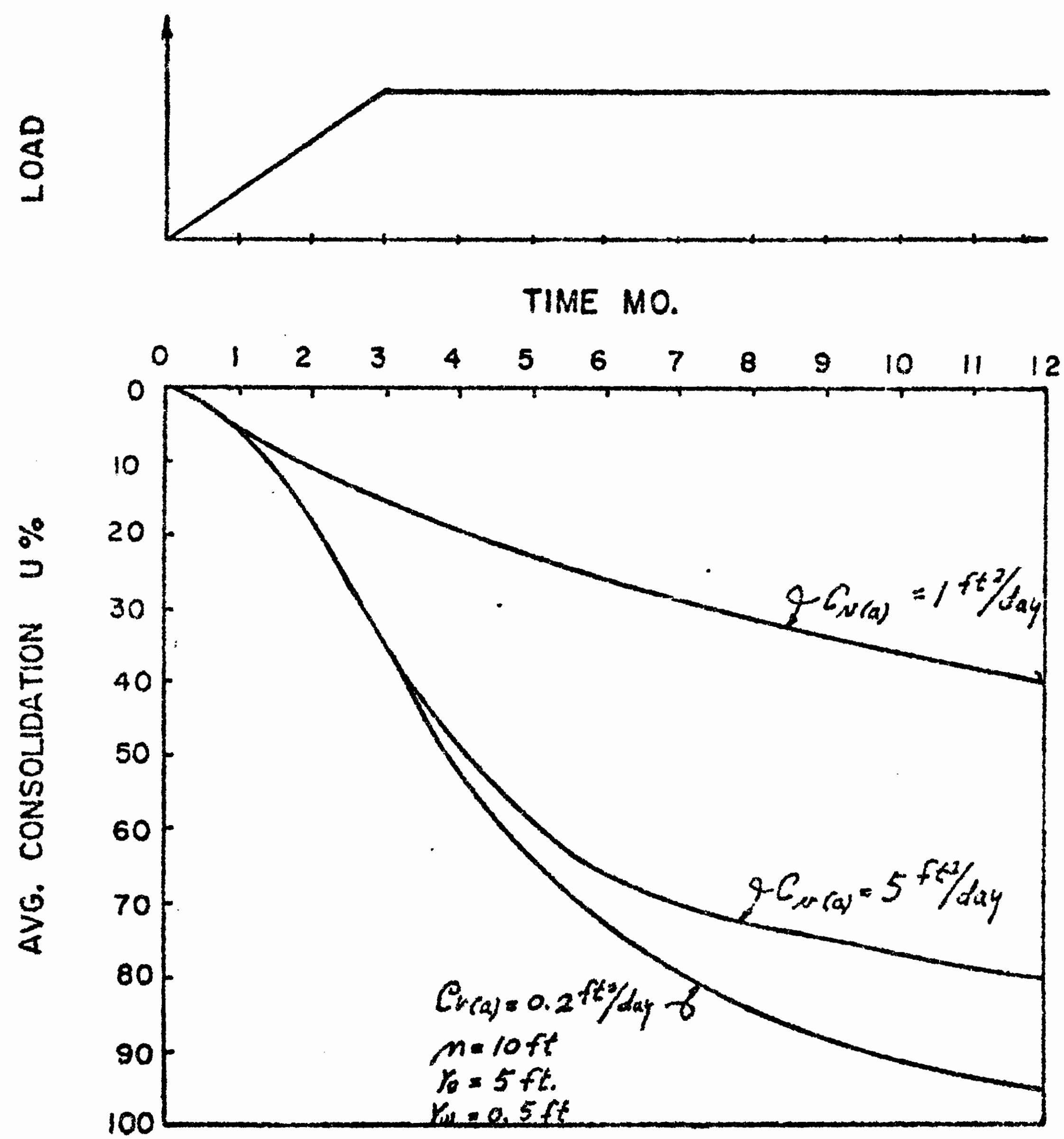

60
50
40
30
20
10
0
78+00
80+00
82+00
84+00
86+00
88+00
90+00
CENTERLINE SECTIONS
CENTERLINE STATIONS
78+00
80+00
82+00
84+00
86+00
88+00
90+00
0
1.0
2.0
3.0
INITIAL SETTLEMENT (ft.)

FIGURE 4



LOAD
TIME MO.
0
1
2
3
4
5
6
7
8
9
10
11
12
AVG. CONSOLIDATION U %
0
10
20
30
40
50
60
70
80
90
100
Cv(a) = 1 ft²/day
Cv(a) = 5 ft²/day
Cv(a) = 0.2 ft²/day
m = 10 ft
r0 = 5 ft.
rw = 0.5 ft

FIGURE 5

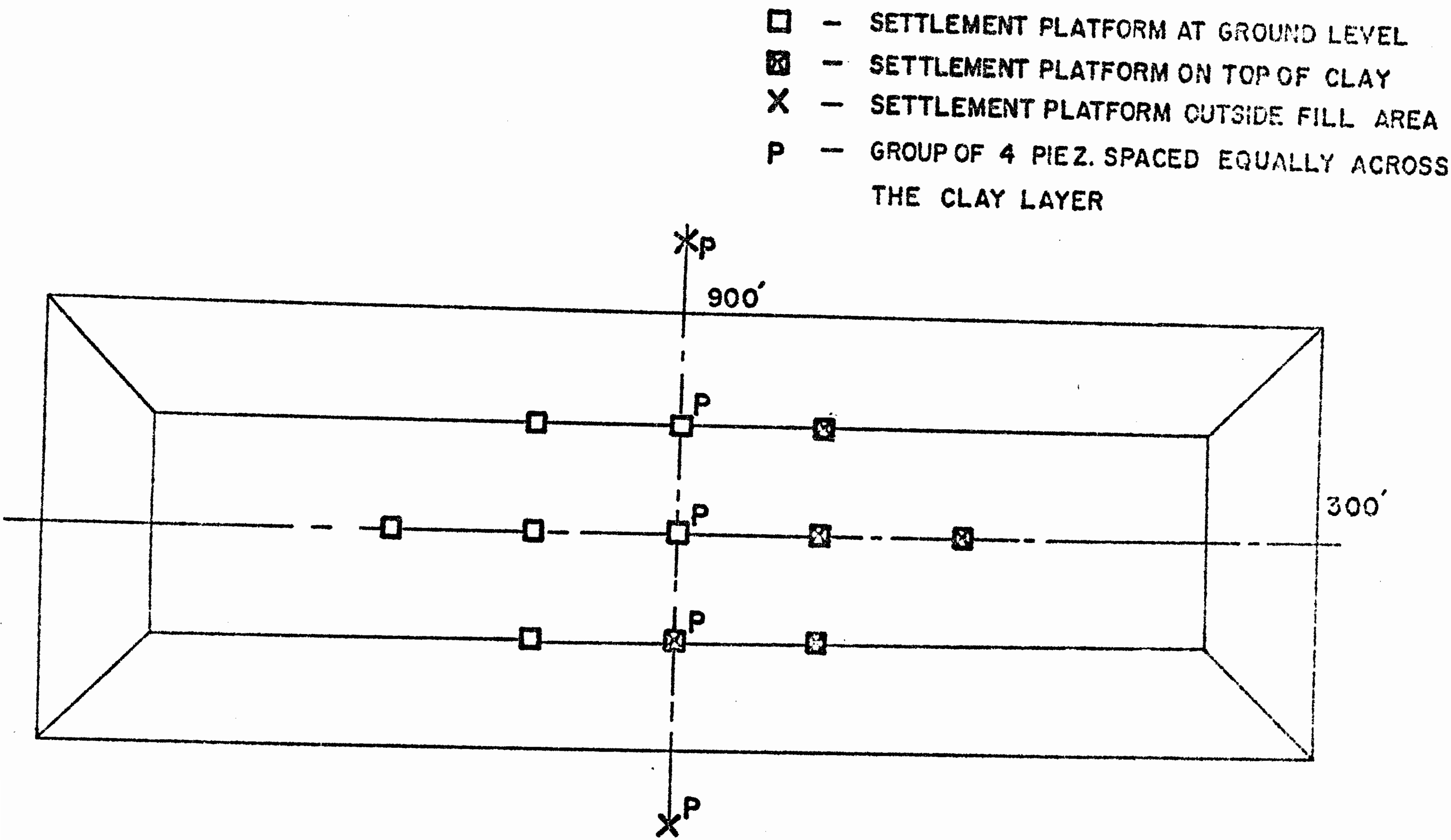

□ — SETTLEMENT PLATFORM AT GROUND LEVEL
⊠ — SETTLEMENT PLATFORM ON TOP OF CLAY
X — SETTLEMENT PLATFORM OUTSIDE FILL AREA
P — GROUP OF 4 PIEZ. SPACED EQUALLY ACROSS THE CLAY LAYER
900′
300′
P
P
P
P
P

FIGURE 6

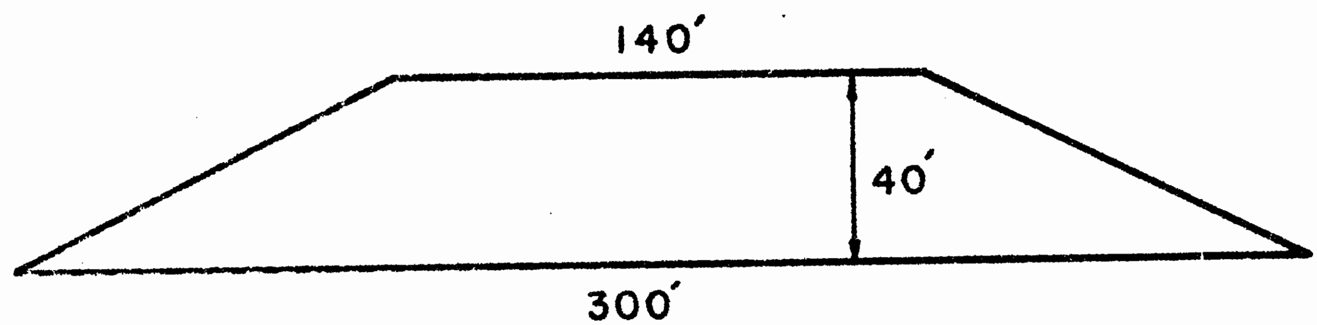


FIGURE 7