# Performance of a Prefabricated Vertical Drain Installation Beneath an Embankment

Z. KYFOR, J. MASI, AND R. GEMME

**An embankment was constructed over foundation soils consisting of marls overlying soft clayey silts. Analyses of the foundation conditions indicated that the embankment could not be constructed safely within the required time frame of 9 mo without foundation treatment. In addition, the embankment would be subjected to a large amount of settlement. It was decided to use prefabricated vertical drains spaced at 4.0, 5.5, and 7.5 ft on centers in order to accelerate consolidation of the soft foundation soils. This would accomplish a twofold purpose of transitioning differential settlement behind a pile-supported abutment and developing the necessary foundation strength required to support the embankment. The embankment was constructed in two stages with waiting periods. Performance of the drains was monitored with piezometer and settlement recording devices. This instrumentation indicated that the prefabricated vertical drains performed as expected and are a viable alternative to conventional vertical sand drains. This paper presents an evaluation of the performance and effectiveness of the prefabricated vertical drain system.**

When highway embankments are constructed over weak foundation soils, stability and settlement problems can be expected. If time is of no importance, the embankments can be constructed slowly without special foundation treatment. However, in most cases required scheduling can add considerable cost to a project. Foundation treatments implemented to reduce construction time have proven to be cost effective.

Vertical drains are commonly used to accelerate consolidation of fine-grained foundation soils. The time to reach a given degree of consolidation is related to the permeability of the foundation soils and the square of the length of the maximum drainage path. As the length of the drainage path increases, the time required to reach a given degree of consolidation increases. Although soil permeabilities cannot be changed, the drainage path can be shortened by the use of vertical drains.

This paper examines the treatment design and performance of an embankment foundation that was part of a major highway project that entailed the widening, relocation, and reconstruction of an existing interchange. The project is located in the city of Syracuse at the northeastern end of Onondaga Lake in New York State (Figure 1). The embankment is the north approach to a 2,000-ft-long pile-supported structure. The maximum height of the embankment at the bridge abutment is 35 ft.

A maximum 9-mo time period was imposed on construction of the approach embankment in order to complete this project within the scheduling constraints. Analyses of the embankment loading indicated that settlement and stability would be a problem. To meet the 9-mo construction time schedule, a prefabricated vertical drain system was used in conjunction with stage construction.

New York State Department of Transportation, Soil Mechanics Bureau, Building 7, State Campus, 1220 Washington Avenue, Albany, N.Y. 12232.

## BACKGROUND

### Geology of the Syracuse Vicinity

During the last glacial era, the Syracuse area was covered by a glacial lake. This glacial lake basin began filling with lacustrine silts and clays by underwater sedimentation which has never been subjected to more than its own weight. As the glacier began receding some 10,000 years ago, some of the low-lying areas were occupied by smaller lakes, the present Onondaga Lake being one of them. During the glacial retreat, the surrounding areas of Onondaga Lake began filling with marl, which is a mixture of calcium carbonate, shells, silt, and clay. After the marl was laid down, the formation of peat began. The swampy environment, which exists to this day, favored the preservation of the organic material. In addition to the natural causes of deposition the foundation conditions are further complicated by man-made deposits. Landfill operations in the Syracuse area have dumped garbage and miscellaneous fill in varying quantities throughout many parts of the area.

### General Foundation Conditions

Ten drill holes and one undisturbed sample drill hole were dug in this area. This subsurface investigation generally revealed a surface layer of miscellaneous fill (brick, cinders, sand, etc.) ranging in thickness from 5 to 10 ft overlying 25 to 30 ft of marl. This marl is underlain by 30 ft of soft clayey silt over compact sands and silts extending to bedrock. The embankment profile and foundation stratigraphy are shown in Figure 2. Moisture contents obtained from the drill hole samples were plotted versus elevations and are shown in Figure 3.

## ENGINEERING PROPERTIES OF FOUNDATION SOILS

### Sampling

Hydraulically driven Shelby tube samples were obtained for the purpose of conducting tests to determine the engineering properties of the soft foundation soils. Laboratory testing

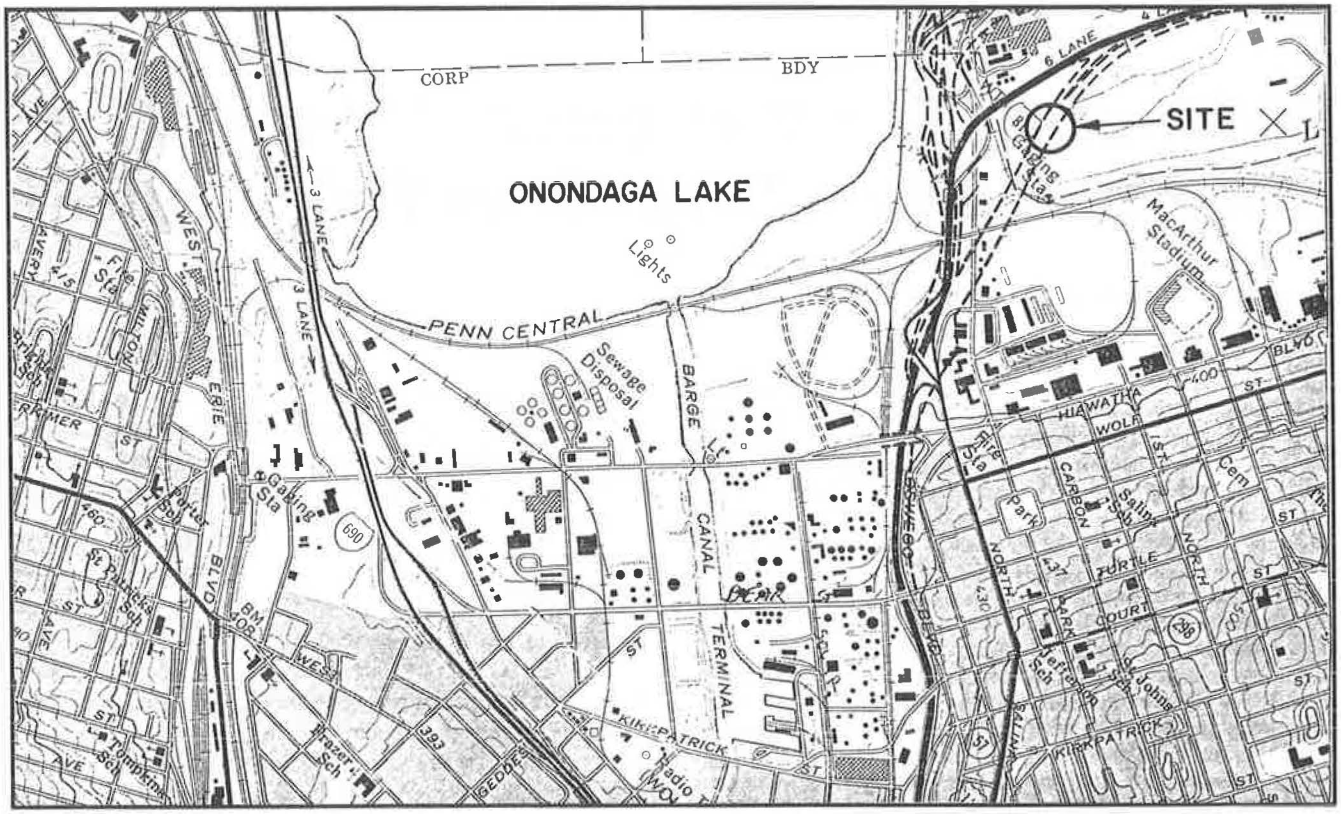


FIGURE 1 Project location plan—Syracuse, New York.

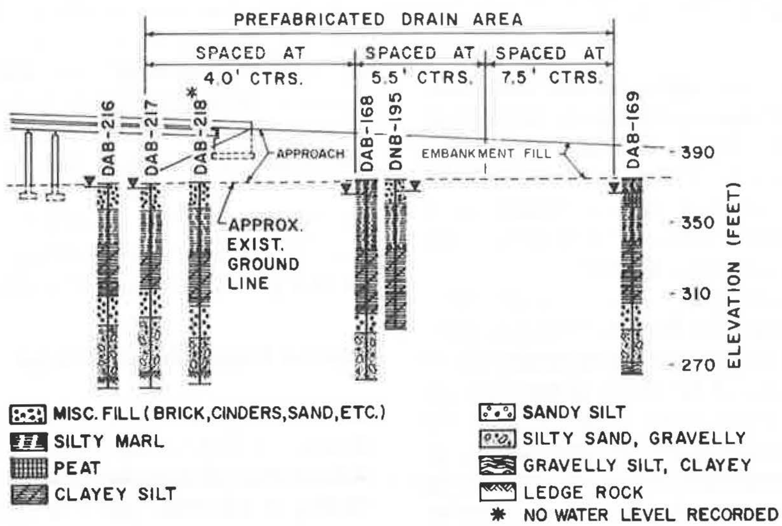


FIGURE 2 Soil profile.

consisted of classification tests, oedometer tests, and triaxial compression strength tests.

Considerable difficulty was encountered in trimming the Shelby tube samples for consolidation and strength testing. Many of the marls were so granular that it was difficult to test them. The marl samples that lent themselves to being tested were those of a fine-grained nature. Some of the clayey silt samples would start to flow upon extrusion from the Shelby tubes due to a very low clay content. The clayey silt samples that lent themselves to being tested were those with a higher percentage of clay. Hydrometer analysis performed on the clayey silt samples indicated that 15 to 20 percent passed the 0.002-mm size.

## Classification Characteristics

Classification tests were performed to aid in the identification of the various strata. These tests consisted of Atterberg limits, specific gravity, wet density, and natural moisture content. A summary of the results is presented in Table 1. According to the Unified Soil Classification System the soft clayey silts would

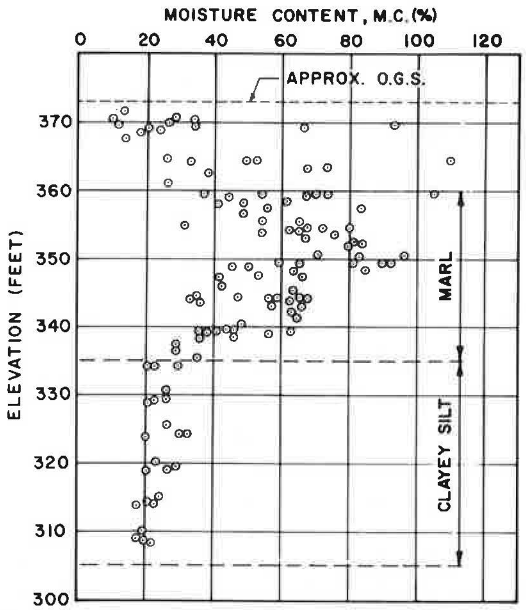


**FIGURE 3 Elevation versus moisture content (prefabricated drain area).**

be classified as CL-ML (inorganic clayey silts of slight to low plasticity).

## Consolidation Characteristics

The consolidation characteristics of the foundation soils determined from oedometer tests indicated that large settlements could be expected under the embankment loads. The preconsolidation pressures estimated, using the Casagrande technique, showed that they were approximately equal to the effective vertical overburden pressures, confirming the geologic history of the deposit as being normally consolidated.

The compression indices, $C_c$'s, for the clayey silts and marls were obtained from the oedometer tests and are plotted versus the natural moisture contents. These results are shown in Figures 4 and 5. Coefficients of vertical consolidation, $c_v$'s, for the clayey silts were plotted versus the natural moisture contents and are shown in Figure 6. Consolidation tests on the marl indicated that the $c_v$ would be greater than 1.0 ft$^2$ per day. The consolidation parameters presented in Figures 4, 5, and 6 represent the entire project area and not just the area under investigation (i.e., other undisturbed laboratory testing results from drill holes not in the immediate vicinity of the 35-ft-high embankment were also included in the summaries). This was done because the subsoils throughout the area are very similar.

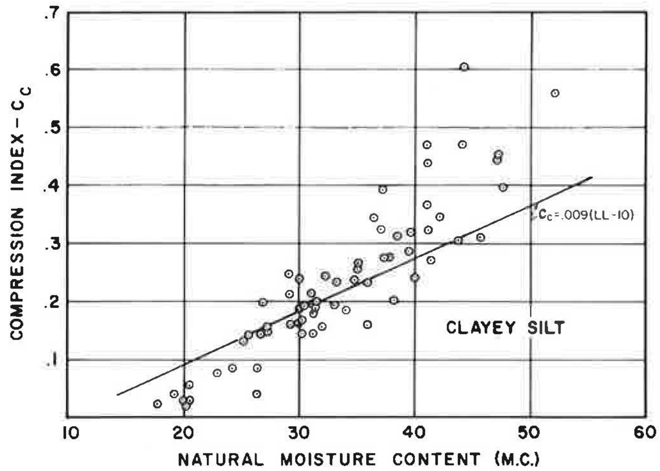


**FIGURE 4 Compression index for clayey silt versus moisture content (total project area).**

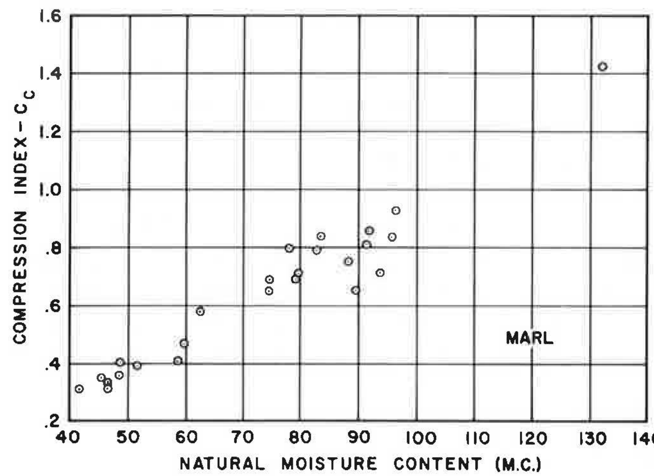


**FIGURE 5 Compression index of marl versus moisture content (total project area).**

## Strength Characteristics

The Soil Mechanics Bureau's past experience with the marls indicated that they drain relatively quickly under loading and can be assumed to act drained, as a sandy soil. This assumption was based on embankment construction instrumentation monitoring results of an interchange in the same area in 1951. A friction angle of 28 degrees was assigned to the marl because of its loose granular nature and previous consolidated drained triaxial compression tests.

The undrained shear strengths for the soft clayey silts were determined by consolidated isotropic undrained (CIU) triaxial compression tests. Since this clayey silt deposit is normally

TABLE 1 SOIL CLASSIFICATION RESULTS (PREFABRICATED DRAIN AREA)

| Soil Type | Liquid Limit (L.L.) | Plasticity Index (P.I.) | Wet Density (pcf) | Specific Gravity of Solids | Natural Moisture Content (%) |
|---|---|---|---|---|---|
| Marl | 40–60 | 8–13 | 90–120 | 2.65± | 50–100 |
| Clayey silt | 21–30 | 5–16 | 110–130 | 2.75± | 20–35 |

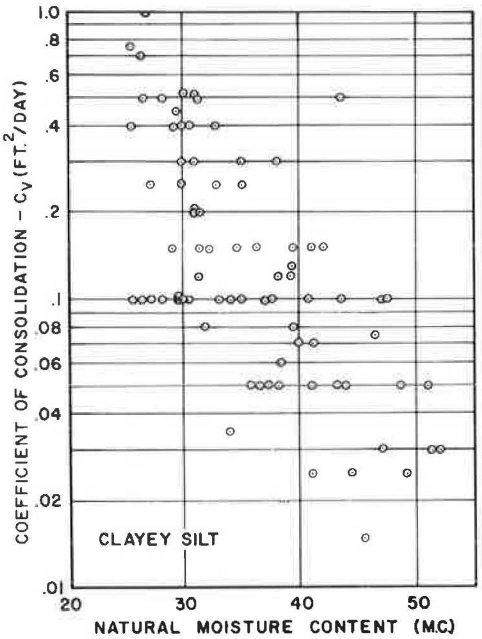


**FIGURE 6 Coefficient of consolidation for clayey silt versus moisture content (total project area).**

consolidated, the undrained shear strength could be expected to increase linearly with depth. This trend, defined by the *c/p* ratio and based on laboratory testing, was determined to be equal to 0.25. Previous testing in the area indicated that the clayey silt had sensitivity values of 2 to 3 (considered insensitive).

## PREDICTED EMBANKMENT PERFORMANCE

### Stability

Embankment stability analyses were performed in terms of total stress analysis ($\phi = 0$) using the Modified Bishop Method of Slices. Analyses indicated that without foundation treatment the proposed 35-ft-high approach embankment, having one vertical on two horizontal side slopes, could not be safely constructed within a 9-mo period because of insufficient strength gain in the foundation soils. Subsequent analyses indicated that the embankment fill could be safely constructed to a height of 20 ft with no rate restrictions.

### Total Settlement

The total estimated settlement beneath the centerline of the maximum 35-ft-high embankment was in the range of 5 to 6 ft. Most of this settlement (60 to 70 percent) was predicted to be attributed to consolidation of the marl layer with the remainder occurring in the clayey silt layer.

### Time Rate of Settlement

Previous experience in this area indicated that the marl layer would consolidate as fast as the fill was placed. As a basis for design the marl layer was assigned a value of $c_v = 1.0$ ft$^2$ per day and the clayey silt layer was assigned a value of $c_v = 0.12$ ft$^2$ per day based on results shown in Figure 6.

## FOUNDATION TREATMENT

The construction schedule established a time constraint of 9 mo from start of embankment construction to start of paving operations requiring that the embankment be stable and without detrimental differential pavement settlement. It was decided that a vertical drainage system would provide the best method of accelerating the settlement rate to meet this schedule. The contract plans provided the option of installing 12-in.-diameter augered or 18-in.-diameter jetted sand drains. Initially the contractor selected the 12-in. augered sand drain, but later proposed the use of a prefabricated vertical drain system. At that time prefabricated drains had been used only on a few projects in the United States and none in New York State. A prefabricated drain known as Alidrain™ was proposed by the contractor. Alidrain is a proprietary item consisting of a band shaped plastic core wrapped in a geotextile.

Considering a cost savings of approximately $100,000 (Table 2) and a review of other documented experiences with the use of these drains, it was decided to accept the contractor's proposal. In addition, the contractor stated that he would take the responsibility for any increased construction time if delays occurred as a result of less effective performance of the Alidrain.

It was proposed to construct the embankment in two stages. The first stage consisted of a maximum 20-ft-high fill, the safe height determined from stability analyses. After achieving 90 percent dissipation of excess pore water pressures (equivalent

TABLE 2 SAND DRAIN VERSUS PERFABRICATED DRAIN COSTS

| Description | Quantity | Unit Price ($) | Amount ($) |
|---|---|---|---|
| Contract items to be deleted | | | |
| 12-in.-diameter vertical sand drain | 116,500 ft. | 4.70 | 547,550.00 |
| Collector drains | 12,108 ft. | 4.00 | 48,432.00 |
| Embankment-in-place (replaced by stone blanket) | 3,250 yd$^3$ | 9.23 | 29,997.50 |
| Total | | | 625,979.50 |
| Contract items to be added | | | |
| Prefabricated drains | 245,076 ft. | 1.69 | 414,178.44 |
| Stone blanket—north abutment | 2,178 yd$^3$ | 35.16 | 76,578.48 |
| Stone blanket—south abutment | 1,072 yd$^3$ | 31.17 | 33,414.24 |
| Total | | | 524,171.16 |

NOTE: Above costs include both north and south abutment approaches.

to strength gain) in the foundation soils, construction of the remainder of the fill (maximum 15 ft additional fill) could be completed.

## PREFABRICATED DRAINS

### Design

The problem of designing a vertical drain system is to determine the drain spacing that will give the required degree of consolidation in a specified time. Prefabricated vertical drain design was performed using the Barron-Kjellman formula (*1*):

$$t_h = \frac{D^2}{8c_h}\left[\frac{\ln D/d}{1-(d/D)^2} - \frac{3-(d/D)^2}{4}\right]\ln\frac{1}{1-U_r}$$

where

- $t_h$ = time required to achieve $U_r$,
- $D$ = diameter of the zone of influence of the drain,
- $c_h$ = coefficient of horizontal consolidation,
- $U_r$ = average degree of consolidation by radial drainage alone, and
- $d$ = equivalent diameter of prefabricated drain.

### Equivalent Sand Drain Diameter (*d*)

The equivalent sand drain diameter for a prefabricated drain is that diameter that will produce the same time rate of consolidation as that of a sand drain of equal diameter. The Soil Mechanics Bureau conducted a laboratory testing program to determine the range of equivalent sand drain diameters for various perfabricated drains (*2*). The testing program consisted of performing large diameter consolidation tests (Figure 7) using three types of remolded soils. The results of this testing program for the Alidrain are given in Table 3. Based on these results, it was decided that the equivalent sand drain diameter (*d*) for the Alidrain would be 2 in.

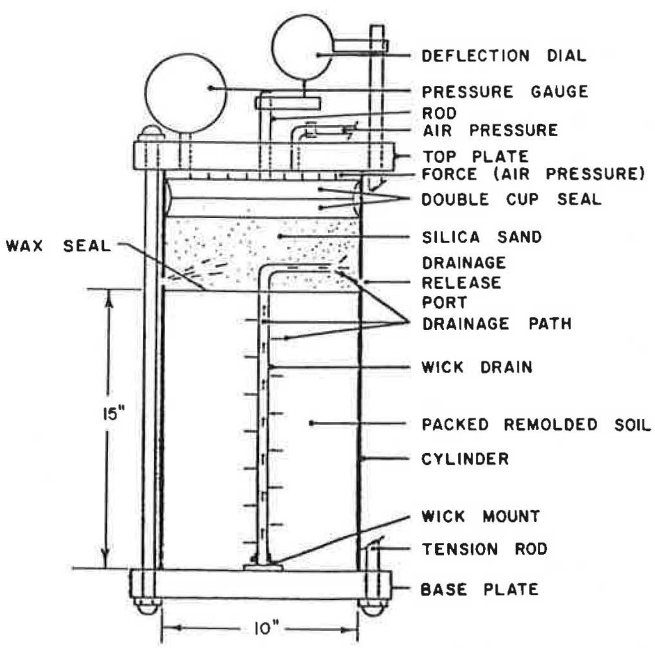


**FIGURE 7 Wick drain consolidometer.**

### Drain Layout

A square drain pattern was selected for this project because it was easier to lay out and control in the field. The effective diameter of the drain was taken as 1.13 *S*, where *S* is the drain spacing.

TABLE 3 RANGE OF EQUIVALENT SAND DRAIN DIAMETERS FOR AN ALIDRAIN

| Soil Type | *MC* (%) | $c_h$ (ft² per day) | Equivalent Sand Drain Diameter (in.) |
|---|---|---|---|
| Organic silty clay (westway soil) | 50 | 0.03 | 2.50–1.37 |
| Manufactured clay | 40 | 0.04 | 2.27–1.05 |
| Peat | 338 | 0.04 | 1.89–1.56 |

### Coefficient of Horizontal Consolidation ($c_h$)

An important parameter in designing a vertical drain system is the coefficient of horizontal consolidation ($c_h$). In practice, however, it is often difficult to obtain a realistic estimate of this key parameter. For design, the coefficient of horizontal consolidation $c_h$ was taken as being equal to the coefficient of vertical consolidation ($c_v$), which is considered conservative. At a later date, block permeability tests (*3*) similar to the permeability test developed at MIT (*4*) were performed and the results are given in Table 4. The block permeability test setup is shown in Figure 8.

These tests indicated that a $c_h$ equal to approximately three times the design $c_v$ value would have been more appropriate. The predicted time rate of settlement for drains spaced on 4-ft centers for $c_h = c_v = 0.12$ ft$^2$ per day and for $c_h = 0.3$ ft$^2$ per day is shown in Figure 9.

### Drain Spacing

The prefabricated drain spacing varied from 4 ft at the abutment to 7.5 ft, 274 ft away from the abutment (Figure 2). The closer spacing was used in the abutment area for stability purposes and the spacing was transitioned back to a larger spacing in order to minimize differential settlements because the abutment was pile supported.

## INSTRUMENTATION PROGRAM

An instrumentation program was initiated to monitor the embankment foundation and evaluate the prefabricated drains. Four pneumatic piezometers, four pneumatic settlement systems (surface type), and one pneumatic settlement system (subsurface type) were installed. All piezometers were installed in the soft clayey silt layer. None were installed within the marl stratum because previous experiences indicated

TABLE 4 BLOCK PERMEABILITY TEST RESULTS

| Depth (ft) | Soil Description | MC (%) | Specific Gravity | Limits LL | Limits PI | Percent Passing 0.002 mm | Kh/Kv |
|---|---|---|---|---|---|---|---|
| 39.2 | Gray brown clayey silt | 27.0 | 2.75 | 19.3 | 6.1 | 20.5 | 3.14 |
| 40.1 | Gray brown clayey silt | 26.2 | 2.76 | 17.4 | 3.9 | 16.6 | 3.86 |

NOTE: Sample is extruded from Shelby tube and then trimmed to form a 2-in. cube.

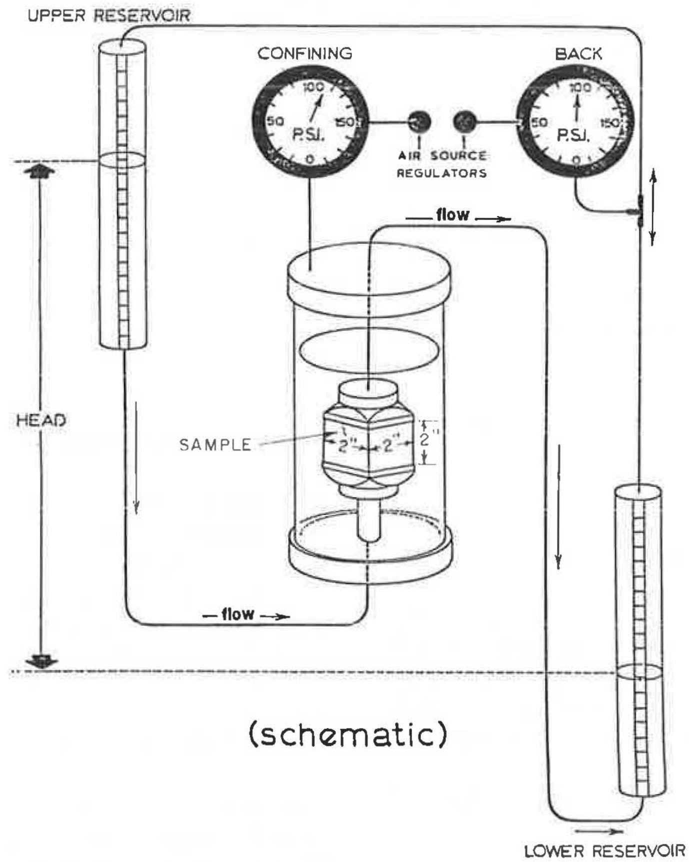


**FIGURE 8 Block permeability test apparatus.**

that excess pore pressure dissipation would occur as fast as the loads were applied.

## INSTALLATION OF PREFABRICATED DRAINS AND INSTRUMENTATION

The ground surface was stripped of vegetation and a 2-ft granular drainage blanket was placed in the prefabricated drain area (5). Initial attempts to install drains failed due to obstructions encountered in the miscellaneous fill. Consequently, preaugering to a depth of 10 to 15 ft became necessary. Upon completion of the preaugering operation, it took approximately 1 min to install each Alidrain to a depth of approximately 72 ft to reach the bottom of the soft clayey silt layer. A total of 2,635 drains were installed.

After the drain installation was completed, piezometers and settlement systems were installed at the locations shown in Figure 10 and at the elevations shown in Figure 11. Actual locations were adjusted in the field to position piezometers at equal distances between drains to measure maximum pore pressures.

## INSTRUMENTATION MONITORING

### First Stage

Construction of the first stage started on June 21, 1982. The 20 ft of first-stage embankment fill was completed in approximately 30 days. There was a 4-mo waiting period during which the piezometers and settlement systems were monitored.

The amount of total settlement that occurred during the embankment construction and the subsequent 114-day (4± mo) waiting period is shown in Figure 9. The amount of settlement that had taken place in the 4.0-, 5.5-, and 7.5-ft drain spacing areas was 4.7 ft, 4.2 ft, and 3.2 ft, respectively. It was unfortunate that the subsurface settlement gauge (SS5) malfunctioned shortly after embankment construction began, because information from this instrument would have indicated

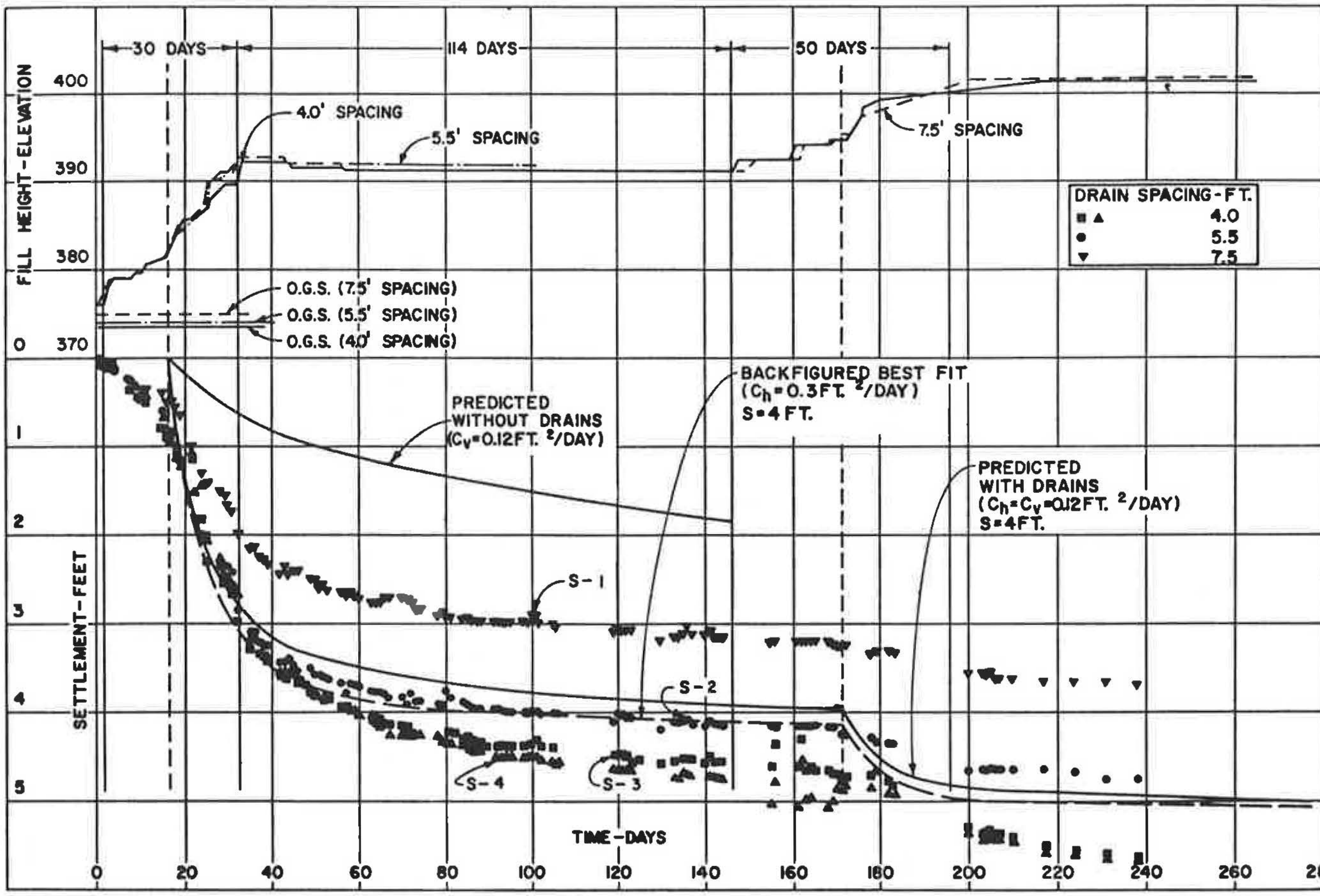


**FIGURE 9 Predicted and measured settlements.**

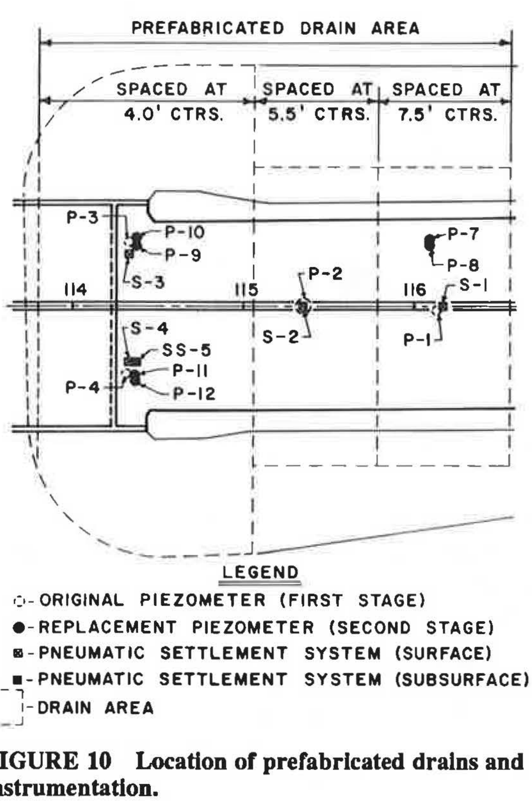


**FIGURE 10 Location of prefabricated drains and instrumentation.**

what percentage of the total settlement was occurring in the clayey silt. Without this information, any evaluation of the effectiveness of the prefabricated drains could not be made based on settlement data. Therefore, piezometer data were used for this purpose.

Figures 12, 13, and 14 show the measured excess pore pressure response to the fill placement. Piezometers P-1, P-2, P-3, and P-4 were functioning normally at the start of embankment construction. Piezometer P-4 ceased to operate within 20 days. The measured excess pore pressures increased in piezometers P-1, P-2, and P-3 as the embankment height increased, as would be expected.

At the end of the first stage embankment loading the measured excess pore pressures at P-1, P-2, and P-3 were 20 psi, 15 psi, and 9 psi, respectively. If dissipation of excess pore pressures had not taken place, the 20± ft of fill would have produced a maximum excess pore pressure of approximately 20 psi. The difference between the theoretical maximum and the measured excess pore pressure at the end of the loading period clearly indicates that dissipation was taking place as expected. Piezometer P-1, which was located in the 7.5-ft drain spacing area, showed virtually negligible dissipation of pore pressure at the end of the first stage fill, and P-3, which was located in the 4.0-ft spacing area, indicated that 50 percent of the expected excess pore pressure had dissipated at that time.

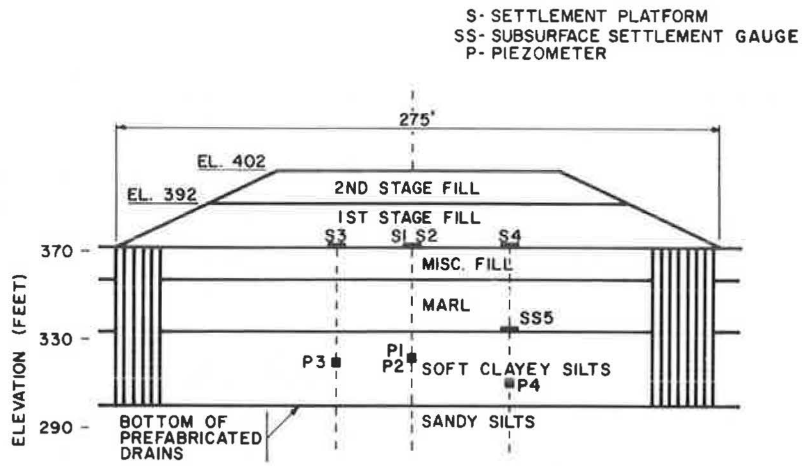


FIGURE 11 Embankment section.

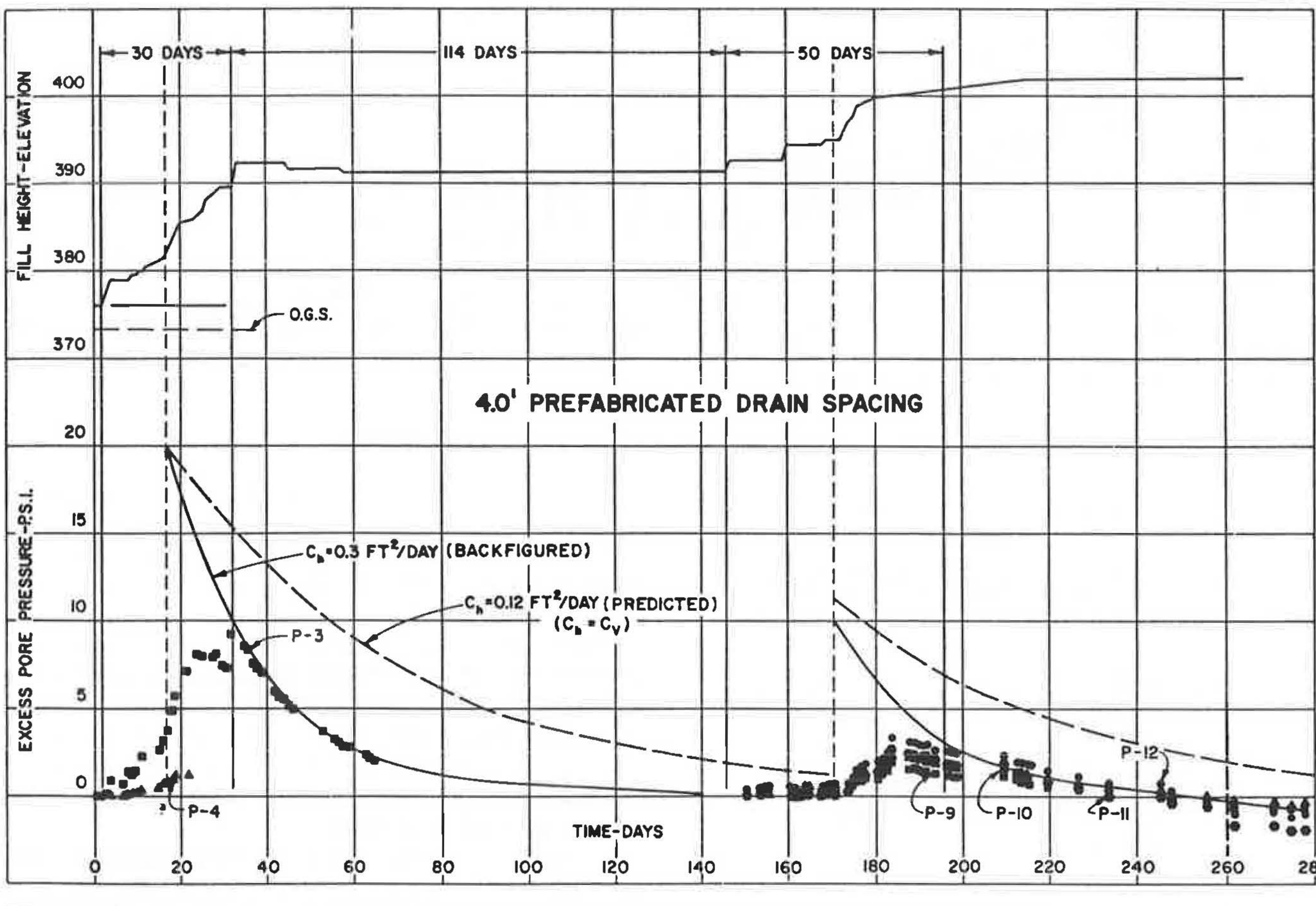


FIGURE 12 Predicted and measured excess pore pressure with 4.0-ft prefabricated drain spacing.

### Second Stage

All of the piezometers ceased to function at the end of the 4-mo waiting period. Six new pneumatic piezometers were installed before placement of the second stage at the locations shown in Figure 10. Four piezometers were installed in the area of the 4.0-ft drain spacing and two piezometers in the area of the 7.5-ft drain spacing.

The second stage construction for the remainder of the fill was completed in approximately 50 days. An additional 0.5 to 1.0 ft of total settlement occurred in conjunction with the second stage loading (Figure 9). The excess pore pressures as shown in Figures 12 and 14 appeared to respond to the fill placement and dissipated with time as expected.

## EVALUATION OF FIELD PERFORMANCE

### Field Coefficient of Consolidation

A determination of the field coefficient of horizontal consolidation ($c_h$) was made based on the rate of dissipation of excess

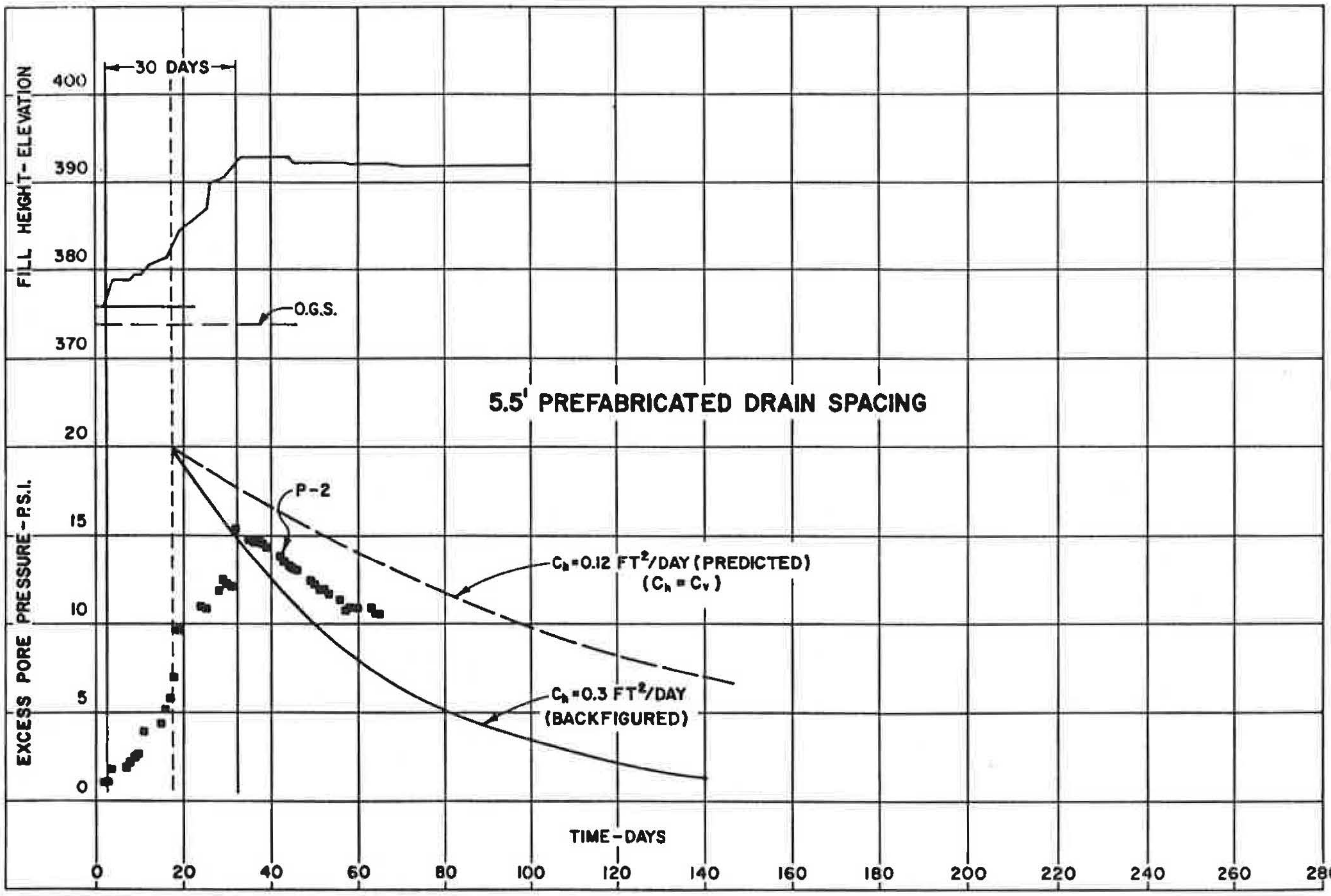


**FIGURE 13 Predicted and measured excess pore pressure with 5.5-ft prefabricated drain spacing.**

pore pressure recorded in the piezometer located in the 4.0-ft spacing area (Figure 12). A value of $c_h = 0.3$ ft$^2$ per day was determined based on the assumptions of radial drainage only and the piezometer being located in the center of the drainage grid. If the piezometers were actually located at the $^1/_4$ points between drains, the value of $c_h$ determined based on the assumption of the piezometer being equally spaced between drains would only be overestimated by a maximum factor of 10 percent (Figure 15). The value of $c_h$ determined based on the assumption of the piezometer being located in the center of the drainage grid is, therefore, reasonable and corresponds well with the block permeability test results which indicated $c_h \approx 3c_v$.

### Performance of Different Vertical Drain Spacings

The efficiency of the drain spacing based on an evaluation of the excess pore pressure dissipation is shown in Figure 16. This figure shows the recorded change in excess pore water pressure with increasing embankment pressure during embankment placement. In the area of the 4.0- and 5.5-ft drain spacings, excess pore pressures were dissipating during embankment construction. On completion of the first stage, approximately 50 and 75 percent, respectively, of the total anticipated excess pore pressure had dissipated. Although no measurable pore pressure dissipation was recorded where the drains were spaced at 7.5 ft, it would be expected that approximately 12 percent dissipation should have taken place based on $c_h = 0.3$ ft$^2$ per day. Twelve percent equates to approximately 2.4± psi pore pressure dissipation, which can be considered small enough to be nondefinitive in verifying the actual field measurement (i.e., $\Delta u/\Delta p = 0.88$ in lieu of 1.0). It is also possible that the piezometer may have started to malfunction.

## SUMMARY AND CONCLUSIONS

The design and performance of an approach embankment constructed over soft subsoils was investigated. Construction scheduling and weak soils necessitated a foundation treatment that consisted of prefabricated vertical drains and a two-stage embankment construction. In order to investigate the behavior of the embankment during construction, field measurements using settlement gauges and pneumatic piezometers were undertaken.

From the data presented herein and for the soils on this particular project, the following conclusions can be drawn.

- The results of field instrumentation indicated that prefabricated drains can be used with confidence in providing a cost effective means of accelerating foundation settlements and allowing for safe construction of embankments within a reasonable time period.
- Back analysis of the excess pore pressure dissipation rates indicated that the coefficient of horizontal consolidation ($c_h$) was approximately three times greater than what was assumed

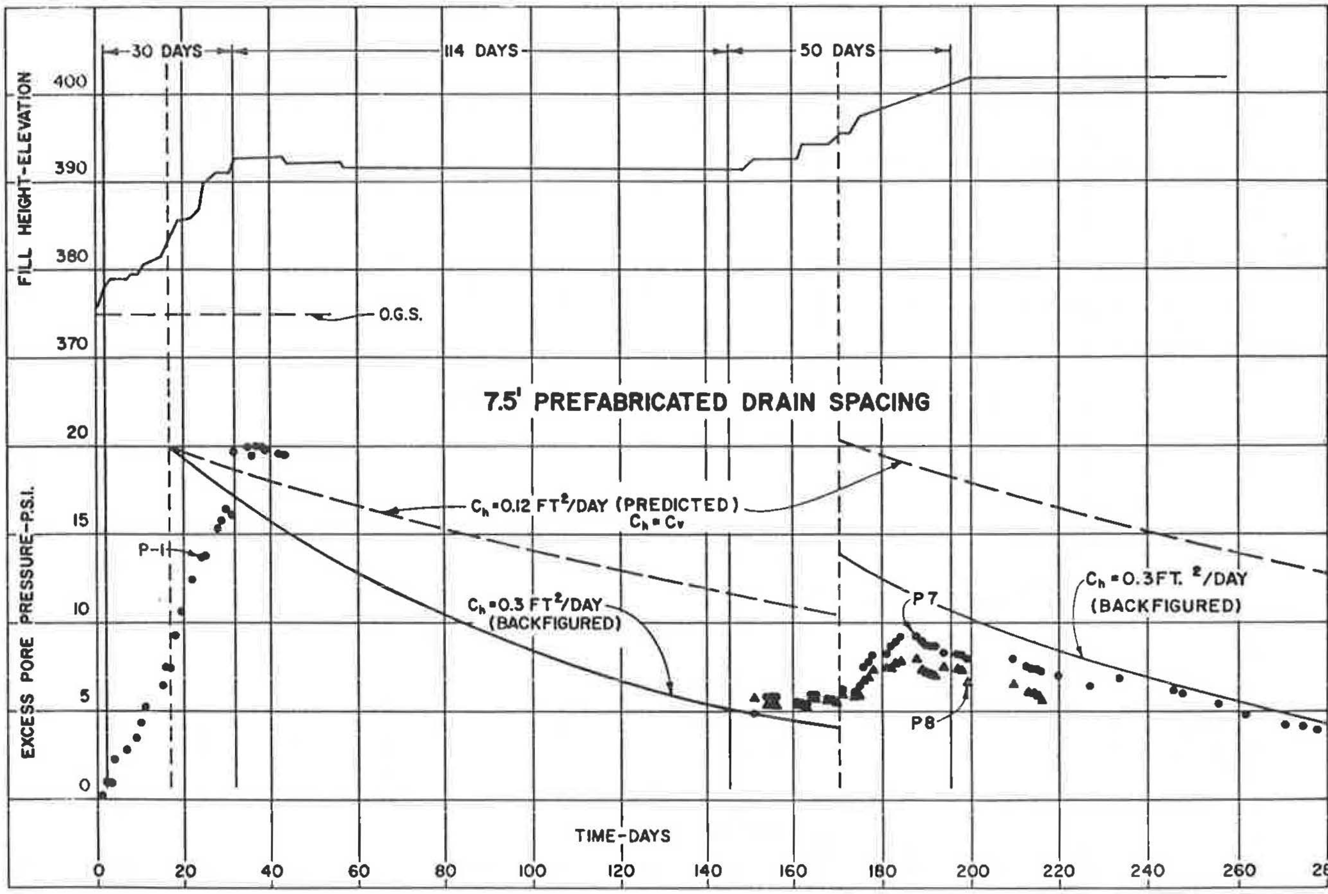


**FIGURE 14** Predicted and measured excess pore pressure with 7.5-ft prefabricated drain spacing.

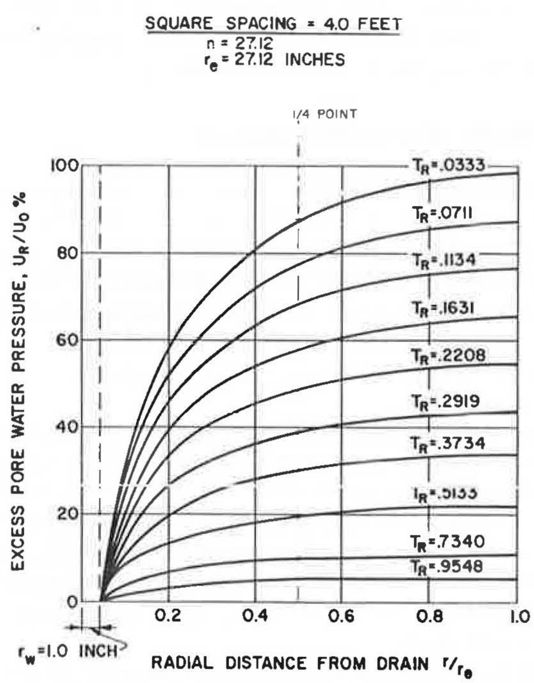


**FIGURE 15** Excess pore water pressure versus radial distance from drain.

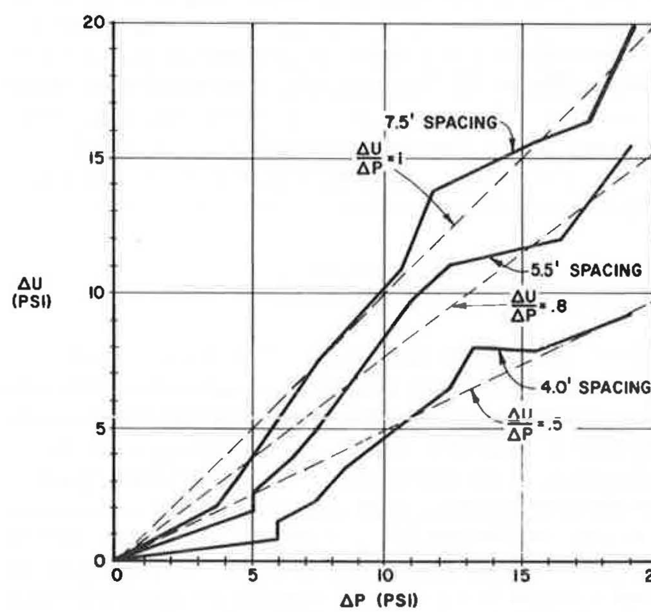


**FIGURE 16** Piezometer response during embankment loading.

in design based on the assumption that the piezometers were equidistant between drains. If the piezometers were actually located at the 1/4 points between drains, less than 10 percent overestimation of $c_h$ would occur. Therefore, this assumption of the piezometers being equidistant between drains is a reasonable one in the determination of $c_h$. The value of $c_h$ obtained from piezometer data is in agreement with results obtained from laboratory block permeability tests on the clayey silt soil.

- Settlement predictions from one-dimensional consolidation tests agreed reasonably well with what was observed in the field.
- The drains were effective in reducing the required amount of excess pore pressure buildup during and after embankment construction.
- An equivalent sand drain diameter of 2 in. was considered as being an appropriate value to use in design for the Alidrain based on laboratory large-diameter consolidation tests and field performance from piezometer readings.
- The Barron-Kjellman formula for predicting time rates was considered to be an appropriate model.

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