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Design and Performance of Horizontal Drains

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SYNOPSIS The paper presents a comparison of field and analytical data regarding the performance of horizontal drains installed to stabilize a landslide. Results of the comparison provide generalized guidelines with which to design drain spacing, length and position. The most significant conclusions are, firstly, that horizontal drains were able to successfully depressurize a silty fine sand with up to 60% silt; secondly, that the ultimate drawdown that can be achieved by slotted horizontal drains in fine-grained soils is controlled primarily by the elevation of the drain; and thirdly; that the design drain spacing is dependent primarily on the initial drawdown response time.

INTRODUCTION

Horizontal drains have been used for more than 40 years as a method of depressurizing slopes in potential and existing landslide areas. The method is attractive in that it can be rapidly implemented and is generally cheaper than alternates. However, because of problems with siltation and long-term maintenance, horizontal drains are frequently considered as a short-term solution. In addition, horizontal drains are often over-designed or designed in a qualitative manner with their response seldom analyzed beyond a general appraisal of piezometric drawdown. Thus, when horizontal drains were selected as a long-term remedial measure to stabilize a major highway landslide, a review of existing drain design methodology indicated that in terms of design of horizontal drains, only limited guidelines and data exist in the literature.

This paper presents a case history of horizontal drains used as a long-term solution for stabilizing a slide area along a highway embankment. The case history includes a review of methods (theoretical and numerical) for analyzing both steady-state and transient response of the drains, a comparison of the analytical and field data, and presentation of guidelines for horizontal drain design.

DESCRIPTION OF SLIDE ACTIVITY

The slide area is located east of Seattle, Washington along a highway completed in 1975. At the location of the slide area, the highway was constructed on up to 40 feet of embankment fill. The roadway instability was first noticed when cracks occurred in the pavement on February 17, 1982 during a period of heavy rain. Displacements increased steadily throughout February and March; vertical displacement at several points along the roadway had reached 13 feet by the end of March. A plan of the slide area is given in Figure 1.

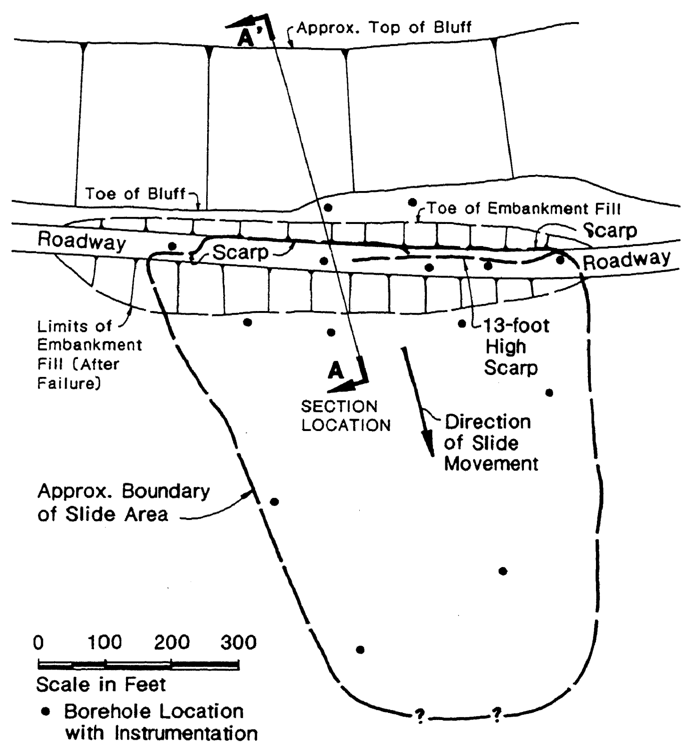

Fig. 1 Plan of Slide Area

The embankment is constructed of silty sand, sand, and gravel, and is situated at the base of a bluff of glacial outwash silt and sand. Periodic sloughing of the bluff has produced a 40-foot-thick layer of loose, silty sand which forms the foundation for the embankment. Underlying the

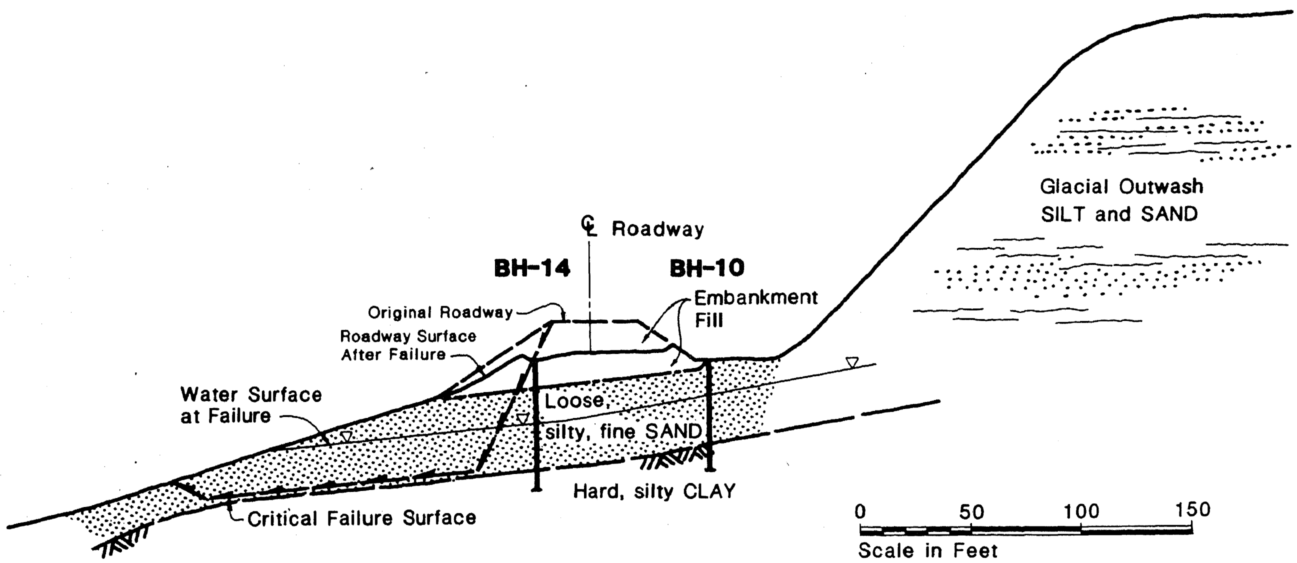

Fig. 2 Section A-A' through Slide Area

loose, silty sand is a layer of hard silty clay. The relative location of these soils and the initial failure surface is given in Figure 2. A large deposit of older slide debris, consisting of a heterogeneous mixture of sand, silt and clay exists a few hundred feet downslope of the embankment toe. Initially, it appeared that the failure was within the embankment itself; however, further reconnaissance led to the conclusion that the embankment and foundation soils were sliding along the contact with the hard silty clay and that the initial movement had reactivated the older slide debris downslope. The slope at the contact of the loose silty sand and hard silty clay corresponds to the direction of slide movement and is inclined approximately at 7 degrees from the horizontal in the area of the initial slide.

A review of precipitation records indicated that rainfall prior to and during the slide was greater than at any other time during the life of the roadway. Water levels, as indicated by piezometers, were within the loose, silty sand and are perched on the hard silty clay. The source of the groundwater is the bluff (i.e., the glacial outwash sand), direct precipitation and runoff infiltration. The lowest piezometer level was 16 feet above the hard silty clay and occurred in the summer.

Gradation analyses of the loose, silty sand indicated that the percentage of silt varies from 15 to 60 percent and averages more than 30 percent. Based on this gradation, correlations give a value of hydraulic conductivity on the order of .0001 cm/sec. Falling head tests conducted in boreholes indicated values of hydraulic conductivity between .0001 and .00001 cm/sec for the silty sand.

Atterberg limits were determined for the hard silty clay, with the liquid limit varying from 36 to 51 and the plastic index varying from 8 to 19. Strength tests of the silty clay were not performed, as considerable local experience in similar slides and previous strength testing exists.

The value of residual sliding friction for thes materials is on the order of 15 degrees. Stabil ity analyses assuming a translational failure, a angle of internal friction of 30 degrees in th fill and silty sand, and an angle of sliding fric tion of 15 degrees along the silty clay indicate that an increase in piezometetric levels of ap proximately 7 feet over the summer levels woul lead to instability. Subsequent movement of th failed mass occurred during January 1983 wit piezometric levels approximately 6 feet over sum mer levels, indicating that the angle of slidin friction is less than 15 degrees.

HORIZONTAL DRAIN DESIGN CRITERIA

The factor of safety for summer water levels an the reduced value of shearing resistance was 1.1 Remedial design criteria were chosen to increas this factor of safety to 1.4 by lowering th steady state groundwater level and to provide suf ficient transient response for heavy precipita tion events that the factor of safety never fall below 1.1. Stability analyses indicated that t achieve a factor of safety of 1.4, the water leve above the silty clay should be decreased to feet, a reduction of 15 feet from the failure con dition. To maintain a factor of safety of 1.1 transient response had to be provided to limit th water level to 18 feet above the silty clay.

REVIEW OF HORIZONTAL DRAIN DESIGN METHODOLOGY

The issues involved in the design of horizonta drains are as follows:

- The spatial drawdown that can be achieved b the drains (steady-state response),
- The time that is required to achieve tha drawdown (transient-response),
- The mounding of the water surface that occur between the drains during periods of pro longed infiltration.

These issues are influenced by the following parameters:

- Soil Characteristics: hydraulic conductivity and specific yield,
- Groundwater Regime: initial position of the water surface and flow boundaries,
- Horizontal Drain Characteristics: spacing, length, position.

Relatively few references were found that specifically discuss the above issues. The following discussion reviews those references found to be applicable.

Steady State Response

Research by Kenney, et al. (1977) describe suggested guidelines with which to design horizontal drains for steady-state response. The work was based on model tests of drawdown due to horizontal drains for a 3H:1V embankment slope. Values for length of drain and drain spacing are presented for a given increase in the factor of safety against slope failure. A second paper, Nonveiller (1981) used a two-dimensional finite difference computer solution to verify the results of the previous work and determined that time to "stabilization" is within 1 month for sandy or silty soils and within 6 months for clay soils.

Transient Response

Several publications were found that discuss the transient response of flat drains above a horizontal impermeable surface (e.g. U.S. Bureau of Reclamation, 1978, and van Schilfgaarde, et al., 1956). The response is given by

$$Y=\frac{4}{\pi}Yo\ \exp(-\pi^2\frac{K}{Sy}.\frac{D}{S^2}.\ t) \qquad (1)$$

where Yo=initial height of water above drain elevation
Y=height of water above drain elevation at time, t
K=hydraulic conductivity of the soil
Sy=specific yield of the soil,
D=average water height above impermeable layer=d+Yo/2 where d=height of drains above the impermeable layer
S=lateral spacing of drains

A second method with which to determine transient response is the use of numerical methods, finite element or finite difference. The authors utilized a finite difference computer program available from the Illinois State Water Survey (Prickett and Lonnquist, 1971).

Steady-state response during periods of infiltration

This case differs from the previous steady-state case in that mounding of the water surface occurs between and above the elevation of the drains as a result of constant infiltration (rainfall). Solutions with which to compute the height of mounding above the drains are given by Dagan (1964), Kirkham (1958) and the U.S. Bureau of Reclamation (1978).

The authors utilized Equation (1) and the Prickett and Lonnquist finite difference program to analyze the drains. Comparison of the predicted and actual data are presented herein. Estimates of mounding between the drains range up to a few feet and were taken into account in the design of the drains. No field data is available for mounding between the drains.

HORIZONTAL DRAIN DESIGN AND CONSTRUCTION

The use of horizontal drains at this site was initially questioned as the hydraulic conductivity of the loose silty sand represents the usual lower bound for the use of gravity drainage systems. This concern led to the decision to install the drains in two phases—Phase I was designed to penetrate into the bluff and validate the performance of the drains and Phase II, to complete the system. In the design it was assumed that the initial water surface is approximately 20 feet above the silty clay, that K=0.0001 cm/sec and that Sy=20% (Johnson, 1976). Spacing of the drains was based on an arbitrary choice of a 4-day transient response.

A plan of the horizontal drain layout is shown in Figure 3. The Phase I drains were subdivided into three sections—two outer fans and a central parallel section. The drains were spaced radially at 5 degrees in the fans and at 20 feet in the central parallel section. The Phase II drains were installed in two fans with 5 to 7 degree spacing. The lengths of the Phase I drains varied from 275 to 400 feet while the length of the Phase II drains varied from 150 to 360 feet. Several of the drains were wrapped with a woven monofilament geotextile to compare their performance with the unwrapped drains.

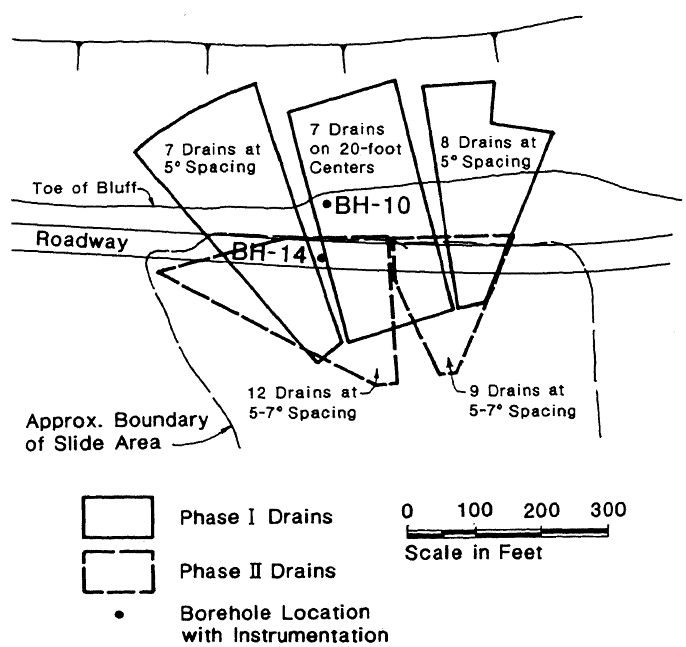

Fig. 3 Horizontal Drain Layout

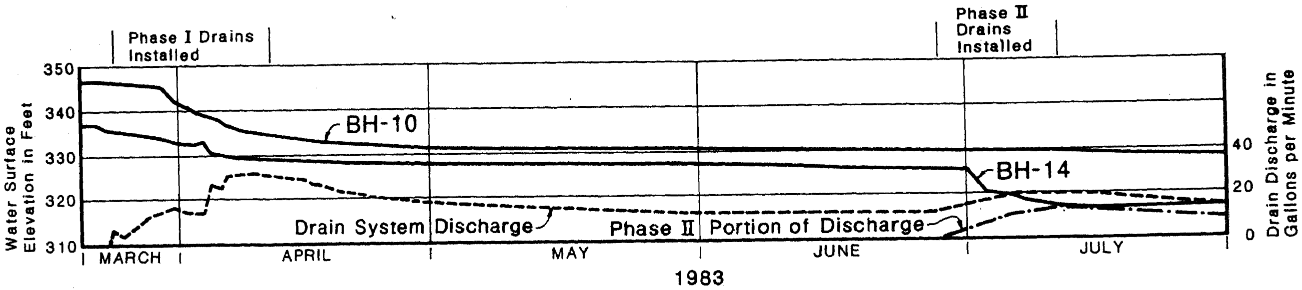

Fig. 4 Water Surface and Drain Discharge Records

The drain pipe was installed through the drill rod utilizing a disposable bit. In this case, drill rods of $2\frac{1}{2}$-inch I.D. were used with $1\frac{1}{2}$-inch O.D. slotted (0.01 inch) PVC (unwrapped) and $1\frac{1}{4}$-inch O.D. slotted (0.04 inch) PVC (wrapped) drain pipe. The last 20-foot section of drain pipe was not slotted. At completion, the elevation at the end of the drain was verified by measuring the hydraulic head on the drill rod and also by a tube inserted to the end of the drain.

FIELD PERFORMANCE OF DRAINS

The Phase I drains were installed during the period of March 24 to April 13, 1983 and significantly lowered the water surface. Based on this success, the Phase II drains were installed during the period of June 28 to July 11, 1983.

The section of the slide (Section A-A') selected for discussion in this paper is given in Figure 2. Water levels were determined from BH-10 and BH-14. The drain locations (both Phase I and II) are relatively parallel in this area, facilitating analysis. Another reason for selecting this section is that a number of the Phase I drains were terminated in the hard silty clay in the area of the bluff. Thus, any drawdown near Section A-A' is due solely to the length of drain within the loose silty sand.

The response of BH-10 and BH-14 to the drain installation is given in Figure 4, along with the total discharge from the drain system. It is apparent from Figure 4 that the installation of the drains dramatically reduced the water surface elevation and that this reduction occurred relatively rapidly. The Phase II installation had just as dramatic an effect on BH-14 while having no effect on BH-10 situated beyond the end of the drains.

Flows from individual drains for Phase I ranged up to 1.5 gallons per minute (gpm) after stabilization. These flows were substantially higher than the 0.05 gpm flow estimated to exist beneath the embankment prior to the drain installation, indicating that water was being withdrawn from the more permeable bluff soils. Flows from individual drains for Phase II were less than Phase I.

The reduction in the water surface, immediately after completion of the Phase I and Phase II installations, is presented on Section A—A' in Figure 5. In the immediate area of the initial slide, the average height of the water above the clay is $5\frac{1}{2}$ feet which is equivalent to a reduction of approximately 16 feet below the assumed failure water level. This corresponds to an average factor of safety of 1.5, slightly better than the 1.4 value selected as a design criteria.

It is interesting to note that the water surface corresponds to the horizontal drain profile after the Phase I installation, despite the increased flow area downslope below the drain. The reason for this is, in the authors' opinion, that water is flowing out of the drains back into the soil. This is supported by the fact that the total discharge from the Phase I and Phase II drains, after a brief surge, is similar (17 gpm) to the pre-Phase II, Phase I total discharge (see Figure 4). Other possible explanations are more circumstantial in nature, such as changes in the silty clay slope or permeability. The water surface resulting from the Phase II installation also approaches the elevation of the drains and is marked by a gradual drawdown of the water surface from the Phase I drain elevation. It is thus evident that the drawdown would have been significantly less had the Phase I drains not been installed.

The transient response of the drains was equated to the time required for water levels to stabilize. Considering the response of BH-10 and BH-14 during Phase I, the time to achieve 90% of the maximum possible drawdown (t90) was 13 and 10 days respectively, values greater than the 4 days selected as a design criteria. The slower response is discussed subsequently.

To date, all of the drains (including those that are wrapped with filter fabric), that flowed after the initial flow stabilization, have continued to flow. Further, no appreciable soil has collected in the sedimentation basins for any of the drains and flows do not appear to differ significantly. There does not therefore appear to be any difference between the performance of the wrapped and unwrapped drains.

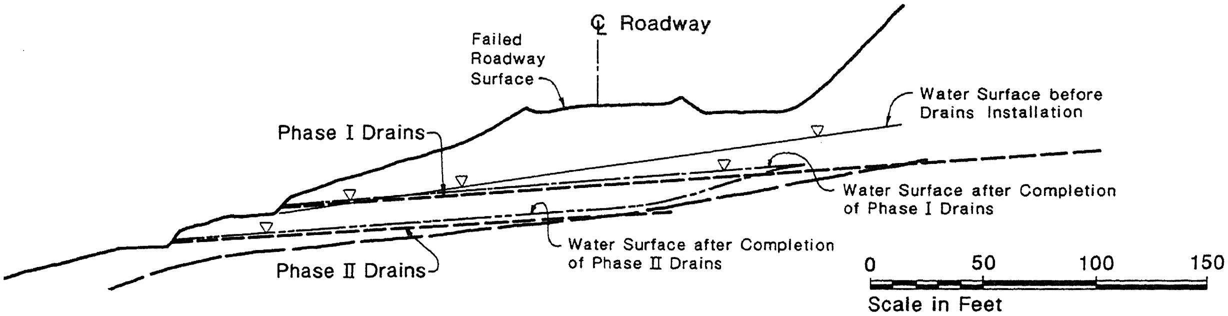

Fig. 5 Water Surface After Drain Installations

COMPARISON OF PREDICTED AND FIELD PERFORMANCE

The drawdown-versus-time curves for the Phase I installation are shown plotted in Figure 6 for piezometers BH-10 and BH-14 corresponding to Section A-A' in Figure 2. Also shown in the figure are the calculated drawdown-versus-time curves using Equation 1 and the finite difference program using a K of 0.00005 cm/sec. This value of K was selected as providing the best fit of the field data. It should be noted that the comparison of the drawdown versus time curves is made assuming that the field data for BH-10 and BH-14 corresponds to the mid-point between the drains.

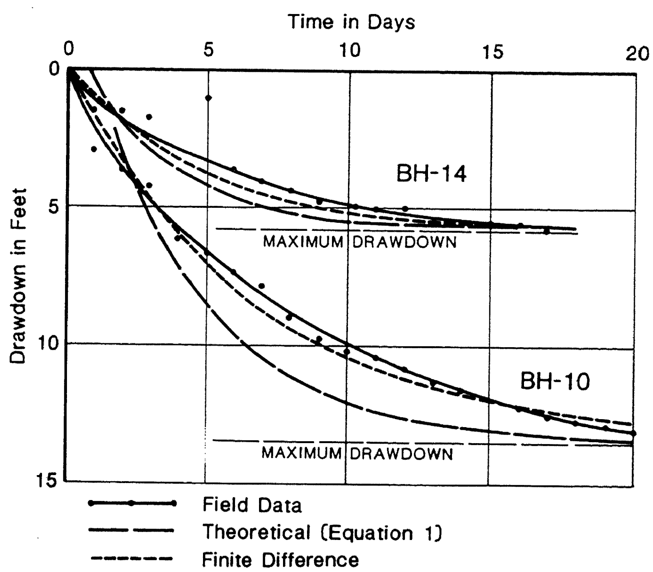

Fig. 6 Comparison of Drawdown Response

As shown in the figure, there is reasonably good agreement between the calculated and the field curves. This was generally the case for other sections analyzed through the slide area. Generally, the finite difference curve is similar to the actual field curve throughout most of the drawdown period while the curve yielded by Equation 1 is not as satisfactory. In general, Equation 1 yields times to achieve 90% of the maximum possible drawdown (t90) equal to approximately 60 to 80% less than by the finite difference method. This includes the case of a horizontal impermeable surface for which Equation 1 was originally derived.

The value of t90 using K=.00005 was 10 to 13 days and is more than the 4-day response time, probably due to the higher value of K used in the design analysis. However, the relatively large increase in water level required to affect the stability of the embankment was decided to be adequate to circumvent the need to improve the transient response.

It is of interest to compare the drain performance with the design guidelines suggested by Kenney, et al. (1977). Using these guidelines the drain spacing is three times the height of water measured at the crest of the slope using the toe of the slope as a datum. In this case the drain spacing should be 60 feet and t90, using Equation 1 and a K=0.00005 cm/sec, is 70 days. Further, for drains installed in silts and clays with 20-foot spacing and K=.000001 cm/sec, t90 is 200 days. Although this may be acceptable for some applications it would generally be unacceptable, in the authors' opinion, for the normally expedient measures demanded in landslide stabilization.

The finite difference studies were also used to investigate the water level surface at the end of the drain. The drawdown of the water surface at various times for the Phase I drains is shown on a longitudinal section at the mid-point between the drains in Figure 7. As shown in the figure, the water surface decreases to the elevation of the horizontal drain; however, the drawdown rate decreases near the end of the drain. After 40 days the water surface approaches a point approximately 35 feet from the end of the drain at the mid-section between the drains. Parametric studies indicate that the flow-through effect is essentially independent of hydraulic conductivity. The effect is considered to be primarily a function of drain spacing and the initial water surface elevation relative to the drain elevation.

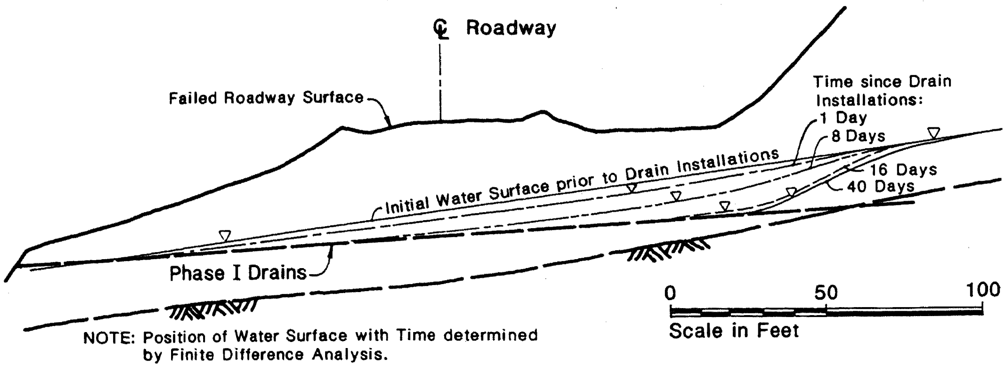

Fig. 7 Time-Development of Water Surface at End of Drains

CONCLUSIONS

The conclusions that may be made from the case history and analyses described in the paper are as follows:

- Horizontal drains were able to successfully depressurize a silty fine sand with up to 60% silt. There appear to be no problems with piping or clogging even though the drain slot size (0.01 inch) is wider than recommended using typical well screen design.
- The particular geometry of the case history presented required two layers of drains to work in tandem to provide the desired drawdown to stabilize the embankment.
- The ultimate drawdown that can be achieved by slotted horizontal drains installed in similar soil is controlled primarily by the elevation of the drain. It may be possible to achieve additional drawdown by using unslotted drains in the area where the water surface is permanently lowered.
- In this case slotted drains were required along the entire length to provide transient response to infiltration from precipitation.
- The design drain spacing is dependent on both the initial drawdown response and mounding between the drains during prolonged infiltration conditions.
- The finite difference program of Prickett and Lonnquist (1971) gave reasonable agreement with the observed drawdown response.
- A theoretical solution presented as Equation 1 gave a predicted response on the order of 60 to 80% faster than that measured.
- The length of the drains should extend beyond the area to be depressurized to allow for flow between the drains. The additional length depends on the spacing of the drains and the difference in elevation between the water surface and the drain. In this case an additional 35 feet of drain is required.

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