THE CONTROL OF A LANDSLIDE BY SUBSURFACE DRAINAGE

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SYNOPSIS

A general description is given of a system of subsurface drainage which has been effective in checking the progress of slides along a portion of the west boundary of the US Naval Station, Seattle, Washington. Vertical wells, filled with pervious materials, were constructed at the crown of the steepest portion of the slope. When constructed, each well of circular section contained an outer vertical layer of graded sand and an inner core of graded gravel. An enlargement (bulb) of the granular materials at the base of each well was penetrated by a 12-in. diameter drain pipe to discharge water into a concrete gutter at the toe of the slope. Slope instability had been caused by infiltration of surface water (back of the locations of the wells) which developed local hydrostatic heads within "blind alley" sand lenses, irregularly dispersed throughout the clay soil of the embankment.

The landslides along the west boundary of the Naval Station, Seattle, have occurred in a hill composed essentially of the sedimentary deposits of the two most recent glacial periods. The discontinuous strata of sand and silt within the present embankment were once at the mouth of a meandering glacial stream. The general slide area is within a peninsula of four to five square miles, one boundary of which is Magnolia Bluff, a sea cliff. Wave action at the foot of this cliff has disclosed the nature of the materials. Near the Naval Station, the upper third of Magnolia Bluff, is a tough stony till of low permeability. This condition, combined with the steep slopes, produces high surface run-off and low percolation. The tough till originated as the ground moraine of the Vashon glacier. The underlying pre-Vashon sediments are alluvial deposits which became consolidated under several thousand feet of ice. The soil borings and the sections of adjacent sea cliffs both evidence the extreme variability of these deposits.

The past high preconsolidation pressures have produced high shearing resistances in the materials under the till, except in the slope areas which, being less confined, have gradually been softened over geological periods of time. In regrading certain sections of the slopes the fine-grained soils could not be compacted to densities of the undisturbed materials and consequently these reconstructed portions of the slopes were less stable than the natural material. Although the fine-grained materials have high percentages of clay-size particles, actually they are rock flours.

At the beginning of the present project, there was a total of eight slides, four of which had occurred within that area with which this report is concerned (Fig. 1). The four slides shown in Figure 1(a) were precipitated by excessive hydrostatic pressure of natural origin, forces that earlier had proved to be too large to be effectively resisted by gravity walls, concrete cribbing, of economical design. The existing slopes are stable during the dry season with prevailing favorable ground water conditions. Theoretical analyses can add nothing to this demonstrated fact. Factors of safety against sliding which are large for dry weather conditions are largely a matter of speculation for the unpredictable, localized conditions that obtain during wet winter periods.

However, the ground water conditions are amenable to control even though they are unpredictable. Boring data can be of almost infinite variety within the area of Magnolia Bluff and the uplift pressures developed within the "blind alley" sand and gravel lenses, typical of which is that shown between elevations + 25 and + 30 of Figure 2, are correspondingly variable. Grading operations, incident to the city development west of the property line and well locations (Fig. 1b), contributed to the infiltration of surface water and rapid build up of localized hydrostatic uplift in the slope. Evidence was disclosed indicating localized uplifts equal to or greater than the weight of overburden. More generally, however, the pressure head of trapped water was considerably less than the weight of overburden but nevertheless sufficient to cause a local slide through reduction in frictional resistance. This condition was first indicated by the Station's report of the presence of springs in the slide areas (Fig. 1a). Moreover R. C. Hennes[1] in a paper descriptive of slides within the Seattle area had clearly depicted these same general conditions.

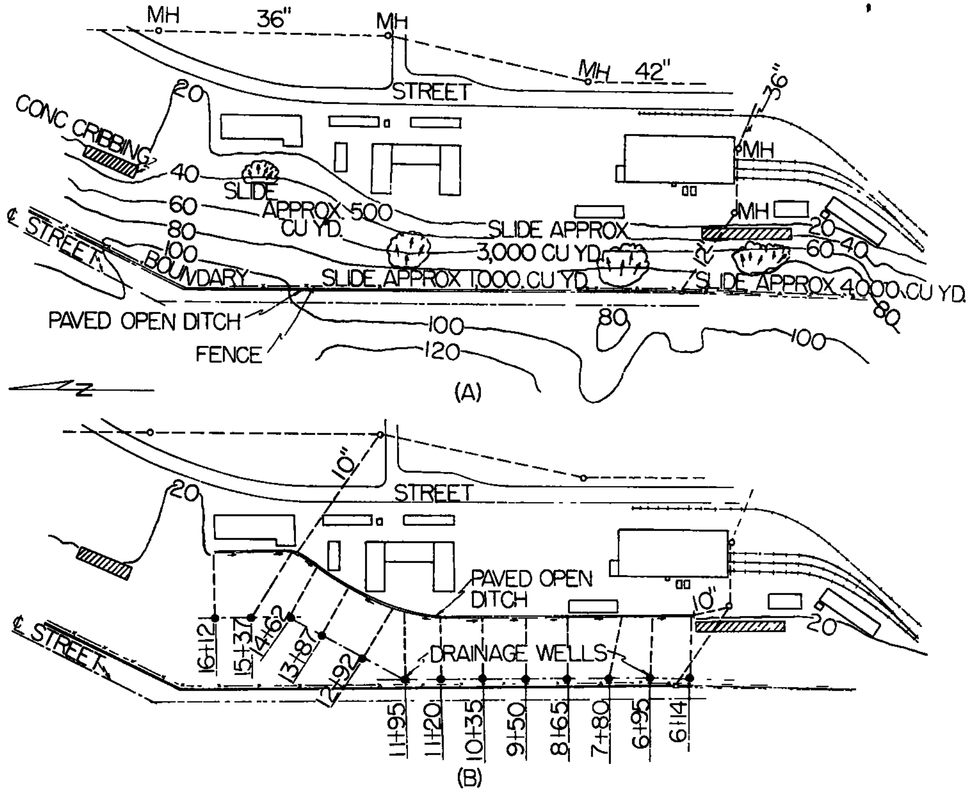

Figure 1. Slide Area Stabilized by Well Drains

DESIGN AND CONSTRUCTION OF WELL DRAINS

The initial design of the vertical wells is shown in Figure 3. The gradation curves for the vertical sand layer 8 in. in thickness, and of the central core of gravel 8 in. or more in diameter are also shown in this figure. The cylindrical mass of sand and gravel, 24 in. or more in diameter, was belled out at the bottom (C in Fig. 3) in a manner and to an extent such that the minimum radial distance from the center to the surface of the bulb was 2 ft. The 12-in. drain pipe (D of Fig. 3) with welded joints and fitted with a well strainer, E, of silicon bronze (slot size 0.01 in. and 2 ft. in length) was designed to discharge water from the well bottom into a concrete gutter at the toe of the slope. These pipes are zinc-coated steel and their pitch is approximately 0.5 percent. After construction was begun it was found expedient to increase the tolerance of lateral displacement of the bottom of the casing, with respect to the cavity end of the well drain, from an originally specified 6 in. to 3 ft. and the cavity was enlarged as necessary to maintain the indicated relationship between sand and gravel. The filter gravel

[1] Paper G-19, Discussion of Section G, "Stability of Earth and Foundation Works and of Natural Slopes", by Robert G. Hennes, Volume III, pages 129 and 130, *Proc.* International Conference on Soil Mechanics and Foundation Engineering, Harvard Univ., June, 1936.

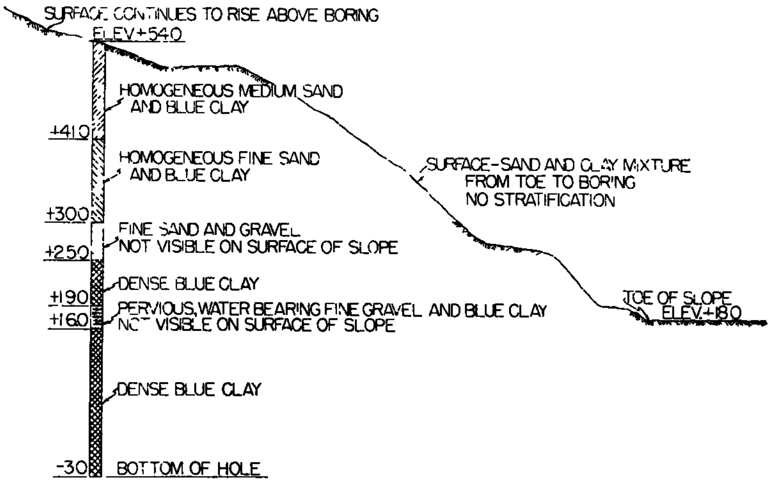

Figure 2. Typical Boring Log in Slide Area

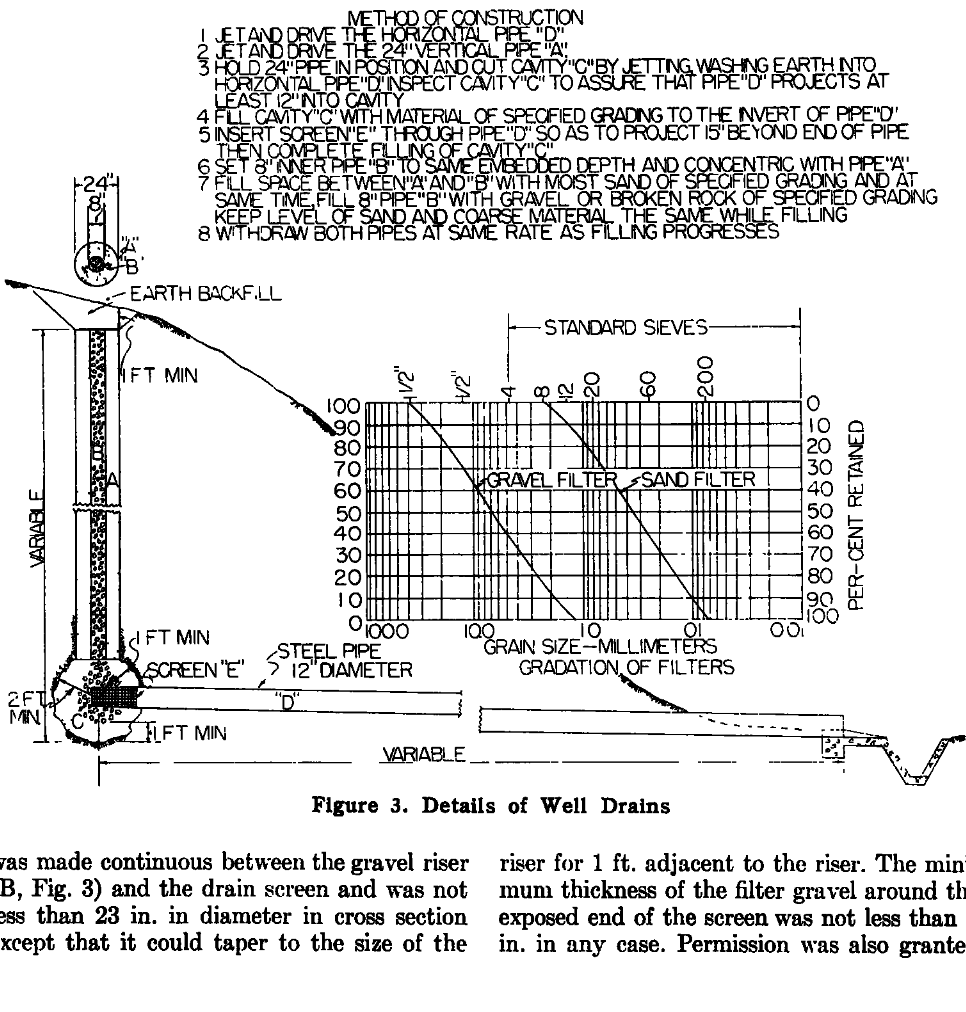

Figure 3. Details of Well Drains

was made continuous between the gravel riser (B, Fig. 3) and the drain screen and was not less than 23 in. in diameter in cross section except that it could taper to the size of the riser for 1 ft. adjacent to the riser. The minimum thickness of the filter gravel around the exposed end of the screen was not less than 6 in. in any case. Permission was also granted the contractor to increase the diameter of the outer casing to any value from 24 (originally specified) to 36 in. if it should become expedient to do so and to increase the diameter of the inner casing for the gravel correspondingly. This was done in a few instances.

The first step in the construction procedure was to install the 12-in. well drain pipes at the locations indicated, by jacking from positions below the toe of the slope, approximately at elevation + 18.0, trenching, if necessary, to permit placing of one section of pipe and to permit the installation of timber guides. Water used in jetting was collected as it emerged at the surface and discharged beyond the toe of the slope through temporary ducts. The casing was driven and jetted down to a depth that was one or more feet above the end of the 12-in. pipe. Then the cavity at the well bottom was cut out mainly by jetting and washing dislodged earth into the drain pipe. Specifications stated that the completion of the cavity would be accomplished by hand labor.

Specifications called for the filter material in the cavity to be placed by an operator in

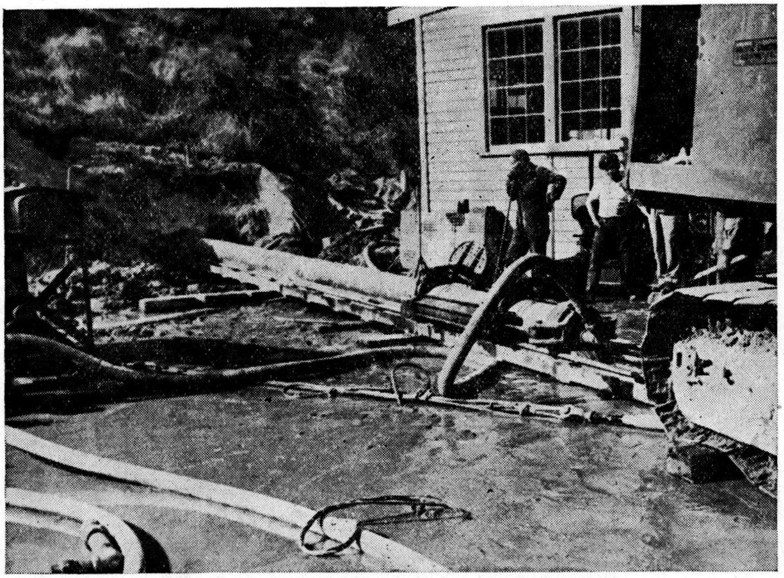

Figure 4. Jacking 12-In. Drain Pipe Into Embankment

Excavation of earth was accomplished from the inside of the pipe by jetting, care being taken not to open the heading larger than the outside diameter of the pipe. When a section of pipe was jacked to its full length, the following section was secured upon the guides, welded to the preceding section, and jacking operations were continued until the entire length of pipe was installed. The lengths of pipes thus installed varies from 110 to 160 ft. Figure 4 is a photograph of the jacking operations.

After the drain pipes were jacked into place the vertical wells to contain the outer sand and inner core of gravel filters were dug by driving and jetting the 24- to 36-in. casing. the cavity. Sand and gravel was first placed to the level of the invert of the well drain pipe after which the well screen was installed. The placing of the sand and gravel was then continued to the top of the cavity with extreme care to insure that the layer of gravel between the well strainer and the layer of sand would not be impaired by any admixture of sand. In placing the filter material above the cavity the filter material was not permitted to extend less than 2 ft. above the bottom of the outer and inner casings at all times to insure the complete separation of the sand and gravel as the sand was fed into the annular space between the two casings and gravel admitted into the inner casing. Both casings were withdrawn upward simultaneously at the same rate. Figure 5 illustrates the method used for withdrawing the outer casing. The graded material comprising the well was covered with two or three thicknesses of tar paper and the space above the tar paper was backfilled with the natural ground and tamped firmly in place. Following installation of the wells certain remedial measures were also taken to effect surface stabilization and surface run-off which are not described here.

The total cost of this project was $117,330.58.

PERFORMANCE

The original plan was to construct 27 of the vertical wells to include slopes not shown in Figure 1 as well as those shown in this figure. The decision to reduce the number to the 13 shown in Figure 1 was made upon advice of Professor Robert G. Hennes, Department of Civil Engineering, University of Washington, who was retained as consultant by the Naval Station at Seattle. In the sections of slope to the south of that shown in Figure 1 and where it was planned to have the other 14 wells, auger borings showed that the underlying aquifiers were more segregated and highly localized than in the area of the 13 wells that were constructed. Where the seams of pervious materials are so discontinuous it is highly improbable that all or even many of the wells located at any arbitrary spacing can function satisfactorily. In the slopes not shown in Figure 1, stability has been improved, temporarily at least, by jacking 4-in. pipe into the side of the slope to the sand pockets located by auger borings. Perforations in the 4-in. pipe were made through the wall after jacking in place. This method of slope stabilization was applied by the Station with Professor Hennes acting as consultant and was accomplished without the aid of the present authors. One advantage of the wells, as described herein, is their relative degree of permanency in the relief of the hydrostatic pressure. There is a tendency for perforated pipe to become clogged after a time.

The unusual amount of rainfall occurring at the site from January to April of 1949 was a severe test for the system of subsurface drainage as herein described. A slide developed slightly to the north of the northernmost well (Fig. 1). This slide came out approximately half way up the slope and progressed until this well itself was involved. The probable cause of this slide has now been located by the station. Several borings were made along the crown of the slope back of the slide and a narrow, 15-ft. wide, and shallow, about 3-ft. depth lens of water bearing sand was located. The elevation of this lens is approximately the same elevation at which the slide came out on the side of the slope. Apparently this lens was not punctured by the northernmost well. When this sand pocket is drained this slide will probably be stabilized. Elsewhere the slope has been effectively stabilized. The discharge has varied from a bare trace to 100 gal. per hr. per well. It is entirely possible that a considerable lowering of hydrostatic pressure may be effected by almost imperceptible discharge from an individual well.

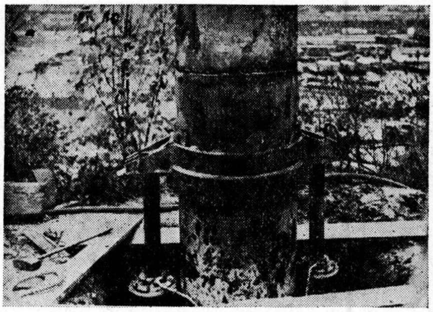

Figure 5. Installation of Vertical Well Drains —Withdrawal of Outer Casing

DISCUSSION

D. P. Krynine, *University of California*—Borings at the site of the Naval Station, Seattle, Washington, indicated the presence of water bearing sand inclusions within a practically impervious glacial till mass. During rainy seasons high hydrostatic pressures develop in these small enclosed aquifiers and make them explode thus causing slides. Presumably dense soil suspension formed in such instances tends to flow out along a path of least resistance which is toward the slope. Hence under given geological conditions slides at the slope are natural safety valves already open and should not be regraded if such regrading may prevent the enclosed water from flowing out. The rest of the slope should be stabilized, however. Given two enclosed aquifiers of equal size located at equal horizontal distances from the slope but at different elevations the lower is more likely to produce a slide than the upper. This is because the rain water falling at the crest of the slope may reach the lower aquifier through fine fissures in the till and thus create in it pore pressures of considerable magnitude when the fissure is completely filled. As a rule, the higher the aquifier, the less the potential danger of slide provided that other circumstances are the same. Hence it is necessary to conduct the percolating rain water from the top of the slope to its foot thereby by-passing the lower aquifiers. This is cleverly done by the authors of the paper by conducting the rain water first vertically and afterwards practically horizontally thus creating a "safety zone" for the lower aquifiers. In some old important railroad cuts the same purpose used to be attained by constructing a deep drainage or even a continuous drainage gallery parallel to the slope. The comparative economic factor in this case is obviously in the favor of the authors of this paper. The idea of Professor Hennes consists in spotting the aquifiers by auger borings and providing exit for water before it accumulates in the aquifier which is done by placing horizontal pipes. This is a still more economical method, but in this case the pipes providing exit for water may be exposed to silting. Cleaning of these pipes is possible, but difficult. The work done at Seattle, Washington, as described in this paper is interesting and merits attention of engineers

EXPEDIENT METHODS FOR STABILIZING SWAMPS

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SYNOPSIS

Porous vertical drains appear to hold the greatest promise for stabilization of swamps for road construction. Vertical drains shorten the settling time and minimize differential settling.

Laboratory experiments are described using porous concrete piles (porous-walled pipe) in place of the usual sand drains. The concrete piles are easier to install and are unaffected by shearing stresses which may develop. They appear to plug less easily also. In the experiments the concrete piles were up to 29 per cent more efficient as drains than the sand columns.

An economic analysis shows that the concrete piles are approximately 30 per cent less expensive than sand drains, without considering the increased efficiency. It is concluded that on the basis of the laboratory tests, full-scale field trials of the porous concrete drains seems justified.

Stabilizing a swamp presents a problem of completely different aspect and nature from stabilizing a road sub-grade. Swamps cannot be called ready to support a road after stabilizing the uppermost few feet—on the contrary, such a procedure may lead to disastrous results.

So far, the four methods most commonly practiced in this country as a solution to the problem are: dumping the fill on the swamp, removing the muck completely; dynamiting after loading the muck, with the idea of pushing it to the sides; and lastly, vertical sand drains. To these methods should be added stabilizing by the use of vertical porous concrete drains.

Dumping leads to the formation of gigantic mud waves which, in addition to slowing the work, present an intricate and costly problem. The removal procedure is almost a foolproof method. (Removal can be effected in two ways, either by clamming the muck out or by dredging it, and dredging is not always 100 percent effective. Dredging was used on a section of the Baltimore-Washington Expressway, but when the fill was placed, mud waves and local failures were observed. Investigation revealed that the last couple of feet or so of