# POTENTIAL USE OF GEOCOMPOSITES TO CONTROL THROUGH SEEPAGE CAUSED EROSION OF HYDRAULIC SANDFILL LEVEES

By


GEORGE JOSEPH MECH

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# INTRODUCTION

During the ten year period between 1961 and 1971, the Rock Island (Illinois) District of the U.S. Army Corps of Engineers was responsible for the design and construction of approximately 200 miles of levees to protect agricultural lands in the Mississippi River flood plain. The design involved improving existing levees by increasing the height and flattening the slopes. Figure 1 is a typical cross section of a main stem levee (i.e., a levee paralleling the Mississippi River). The existing or unimproved levees were generally constructed from impervious clay alluvium. In general, hydraulic sand fill was used to improve the main stem levees. The diversion levees (i.e., levees paralleling tributary streams to the Mississippi River) were improved with semi-compacted impervious fill.

The basic seepage control criteria for the improved levees was two-fold.

1. The control of water entering the riverside face of the levee and exiting on the landside face. The flow domain for this seepage pattern is the hydraulic sand fill placed above the impervious alluvium and the seepage phenomena is termed through seepage in this paper. The through seepage control must prevent slope failures and excessive erosion of the landside slope. Figures 2 and 3 show unimproved levees with inadequate through seepage control.

2. The control of water entering the sand foundation from either the riverbed or existing riverside clay borrow areas and exiting landward of the levee in the drainage ditches, existing landside borrow areas and the adjacent fields. The basic flow domain is in the sand foundation under the levee and this seepage phenomena is termed underseepage. The clay alluvium riverward restricts the entrance to the flow domain and the clay landward

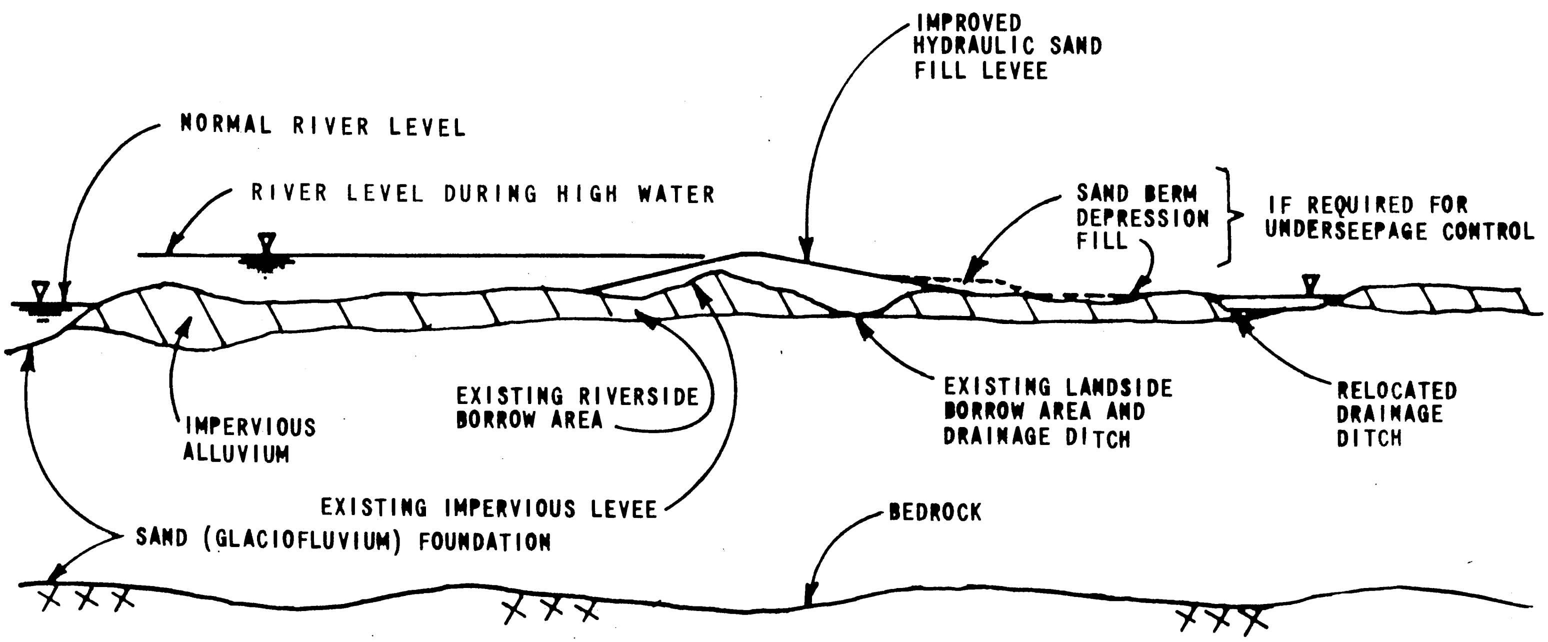



Figure 1 Sketch of Typical Main Stem
Mississippi River Levee.

Figure 2 Unimproved Levee with Inadequate Through Seepage Control at Iowa River-Flint Creek Drainage District (May, 1965). (From U.S. Army Corps of Engineers, 1966).

Figure 3 Unimproved Levee with Inadequate Through Seepage Control at Green Bay Drainage District (May, 1965). (From U.S. Army Corps of Engineers, 1966).

retards the seepage from emerging on the ground surface. The underseepage design criteria must control the hydraulic gradients landward of the levee in order to prevent piping or internal erosion in the sand foundation and excessive uplift pressures on the landside impervious alluvial deposits.

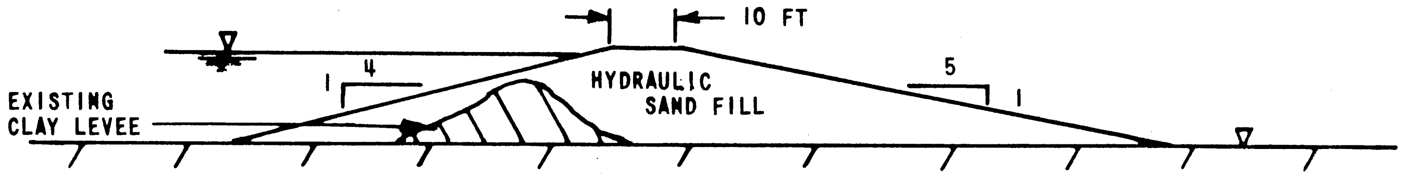
The goal of this paper is to present the current through seepage design of the main stem and levees used by Rock Island District and present an alternative using a geocomposite material to control through seepage. Underseepage will not be considered. The design analysis consists of three items.

1. Analysis of seepage through a hydraulic sand fill levee.

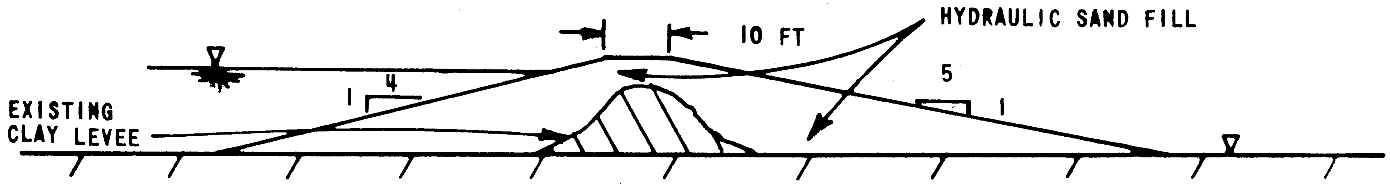
2. Analysis of slope stability to determine the effect of the seepage forces on the stability of the landside slope.

3. Analysis of erosion of the landside slope due to seepage existing on and running down the slope.

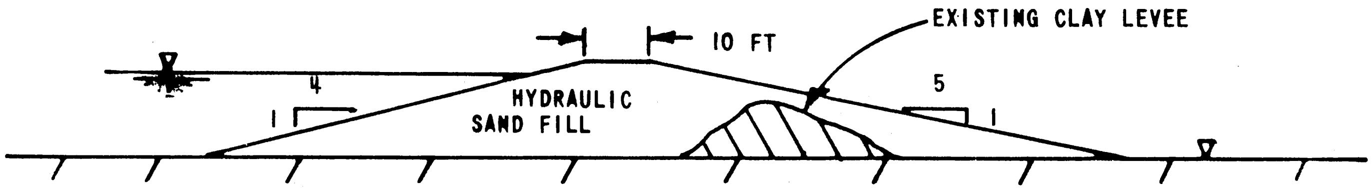
Figure 4 shows typical sections of improved main stem levees. The sand levee section was an empirical design based on the results of a full scale test levee made in the Drury Drainage District (U.S. Army Corps of Engineers, 1962). The test levee indicated that the landside slope would have adequate slope stability and minimal erosion if the levee section had a 1.0 vertical (V) on 4.0 horizontal (H) riverside slope, a 10 foot crown width and a 1.0 V on 5.0 H landside slope. The selected levee section violates a basic criterion for control of seepage through dams and levees. Namely, a toe drainage system is not provided to prevent the free surface of seepage from exiting on the landside slope.




a. Clay Levee Located Riverward of New Levee ℄.




b. Clay Levee Located Near Center of New Levee.




c. Clay Levee Located Landward of New Levee ℄.

Figure 4 Sketch of Improved Sand Levee Cross Sections.

The need for toe drainage is widely recognized. Lambe and Whitman (1969) state:

> "If there were no rock toe in the dam ..., the top flow line would exit on the downstream slope ... The face...would gradually erode away - the water flowing out of the face will carry soil particles with it. This process will eventually cause the entire dam to fail." (p. 270)

Casagrande (1937) points out that levees with landside drainage provisions and relatively steep slopes may be safer with respect to seepage forces and more economical than levees without drainage provisions and relatively flat slopes. Sherard, et al. (1963) states that homogeneous dams with a height greater than 20 feet should be provided with some type of downstream drainage provisions to reduce pore water pressures which increases the downstream slope stability and to control seepage water from carrying away soil particles as it exits on the downstream slope.

In the Spring of 1965, a record high water developed on the Mississippi River. During this event, the performance of the sand levees was documented by detailed field inspections. The through seepage design proved to be satisfactory which further confirmed the results of the levee test section. By 1974, the performance of the sand levees was further documented during high waters in 1969 and 1973. Also at this time, the work load of the Rock Island District was changing from the design of agricultural levees to urban levees. Casagrande (1965) and Cedergren (1973) point out the importance of analyzing and controlling seepage especially if the consequences of failure are serious. Since the chance for loss of life is greater in an urban area than an agricultural area, it was considered imperative to analyze the empirical design of agricultural sand levees before using the design for urban levees. The type of research that was needed was stressed by Harza (1935) in summarizing his work on dams on sand:

> "Mathematics and research by means of hydraulic electric analogy offer valuable information as to fundamental principles of dissipating head in homogeneous materials and relative ability of different foundation types to promote the minimum upward hydraulic gradient at the toe...the writer maintains that a study of the problem should start from a nucleus based upon the law of flow through homogenous material, with modifications introduced in the analogy tray in each individual case based upon such local conditions as can be revealed. Field tests and experience should be grouped around such ideal assumptions instead of about purely empirical and unscientific coefficients.... It is submitted that the proper ultimate method for designing dams on sand is to compare various proposed alternate types of foundation crosssection and abutment designs by the analogy method ... and then to modify these comparisons based on field conditions at the site and empirical data which will have been accumulated in the meantime to aid in interpreting such conditions into their bearing upon the criteria of safety." (pp. 1384-1385)

## SEEPAGE STUDY

During the preliminary design phases of the sand levees, the probable homogeneity and isotropy of hydraulic sand fill was discussed with local dredge contractors. Based on their field experience, the contractors maintained that the hydraulic sand fill is intrinsically homogeneous and isotropic. They cited the following construction procedures as evidence for their opinion.

1. Although the material deposited by the bottom traps generally contains more coarse fractions, it is not significantly more pervious than that deposited by the end discharge because there is less washing action. By properly controlling the traps, the fine sands and silts are retained in the material deposited by the bottom discharges. These sands and silts are washed away in the beach of the end discharge.

2. Clean, medium to fine sand is preferred for borrow and the dredges are specifically moved to maintain this type of material. Coarse sand and gravel reduces pump efficiency and silts tend to destroy the beach ahead of the discharge pipe.

3. Dozers are operated during placement of the hydraulic fill to intermix material and minimize the occurrence of any continuous layers of gravel or silt within the sand fill.

In the initial construction phase of the sand levees, a program was initiated to evaluate the properties of the sand fill. This program was developed to determine the overall range in the gradation of the hydraulic sand levees. A detailed sampling and grain size testing program on sand from the riverside slope, crown and landside slope was conducted at Muscatine Island, Fabius River, and Lima Lake Drainage Districts. The purpose of this sampling and testing program was to ascertain if the hydraulic method of constructing the sand fill resulted in a homogeneous or nonhomogeneous levee cross section. Also, a full scale levee test section was built at the Drury Drainage District in order to observe the actual performance of the hydraulic sand fills.

Figure 5 is a summary of the result of the random sampling and testing program. The range of gradation curves represents 675 tests taken from 8 drainage districts. The sand was randomly sampled along various reaches and at various depths of the

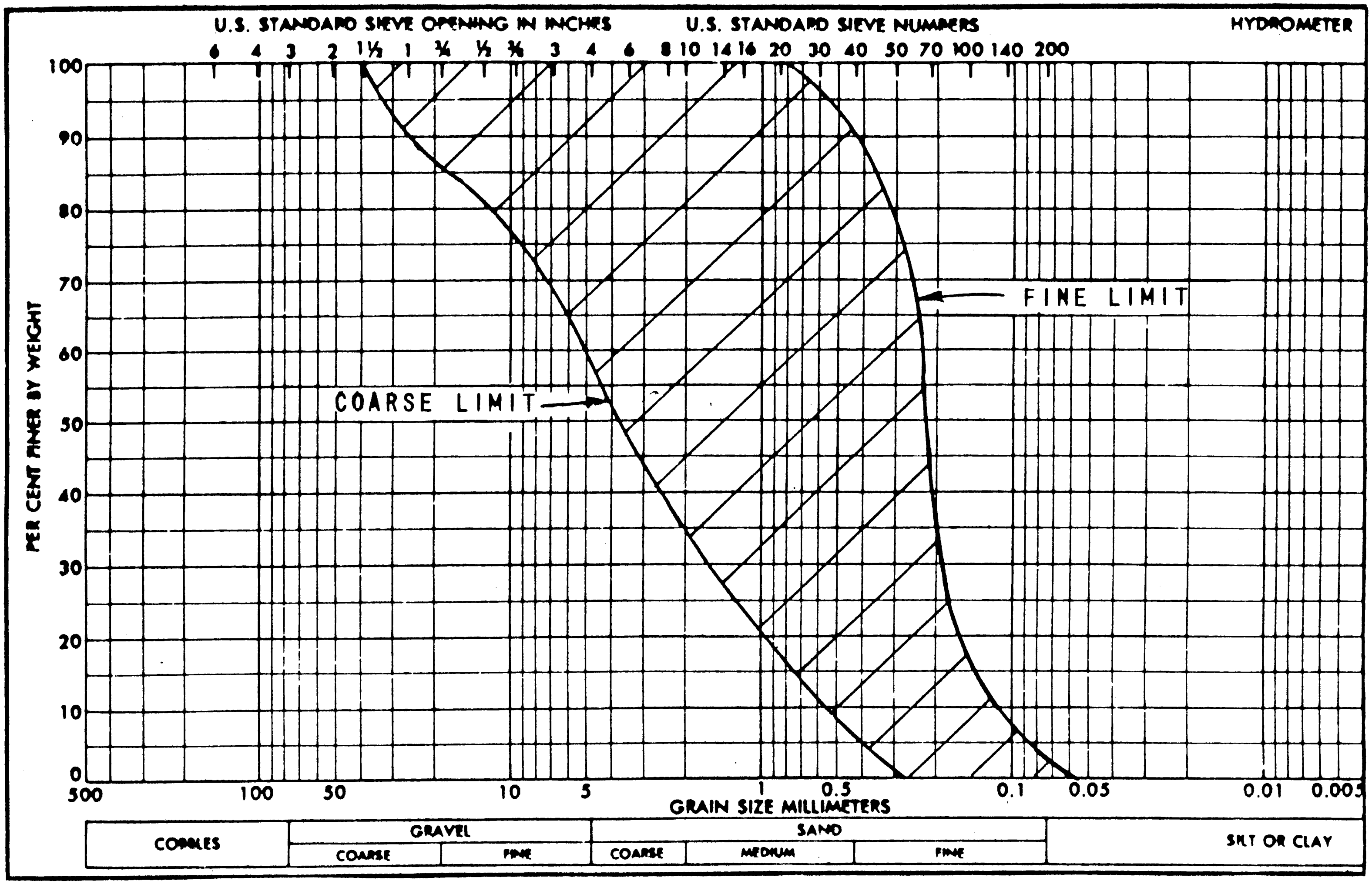



Figure 5 Range of Hydraulic Sand Gradation Curves, 675 Samples from 8 Drainage Districts.

levees. These results show that the $D_{10}$ size (i.e. size that 10 percent of the material is finer than) ranges from 0.12 to 0.55 millimeter (mm) with an average of 0.25 mm. Mansur and Kaufman (1957) used field pumping tests to develop a correlation between the $D_{10}$ size and the horizontal coefficient of permeability for sands in the Mississippi River Valley. The correlation as presented by U.S. Army Corps of Engineers (1956) is shown in Figure 6. By use of Figures 5 and 6, the permeability of the hydraulic sand fill is estimated to vary from 0.08 to 0.80 feet/minute (ft/min) with an average of 0.30 ft/min.

At three drainage districts, samples were collected at various times during the construction of a particular reach. The cross section of the levee was divided into three zones and the locations of the samples were grouped according to these zones. Zone 1 was placed by manipulating baffles on the bottom discharges and by shaping the final slope with bulldozers. Zone 2 was placed by the bottom discharges without any significant reworking of the material. Zone 3 was placed by the end discharge in the form of a beach.

Figure 7 shows the range in gradation curves for the samples collected from a typical drainage district (Lima Lake). The range in gradation curves shown for the Lima Lake Drainage District represents samples taken from three separate reaches. The $D_{10}$ sizes were noted and the average hydraulic conductivities determined from the correlation by Mansur and Kaufman (1957).

A detailed examination of all the results of gradation tests

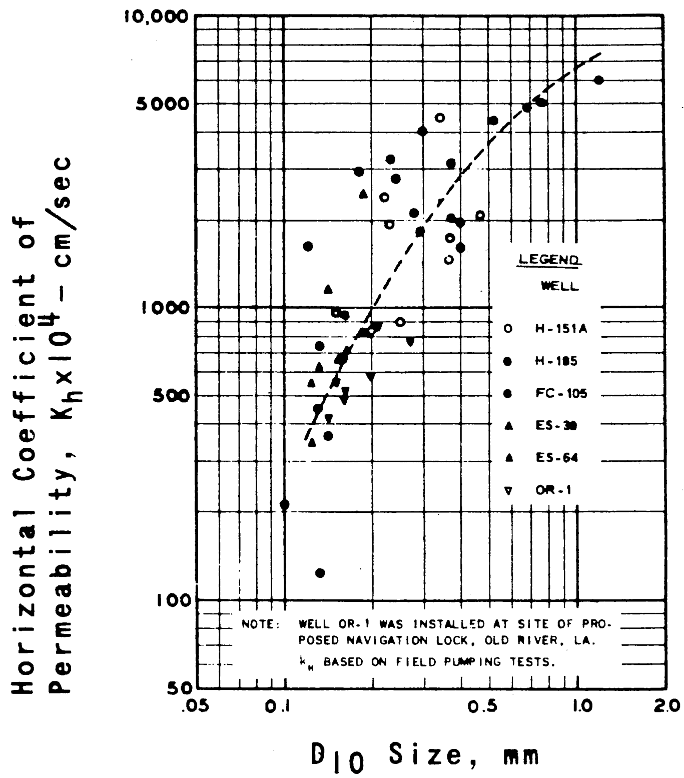



Figure 6 Horizontal Coefficient of Permeability Versus $D_{10}$ Size. (From U.S. Army Corps of Engineers, 1956.)

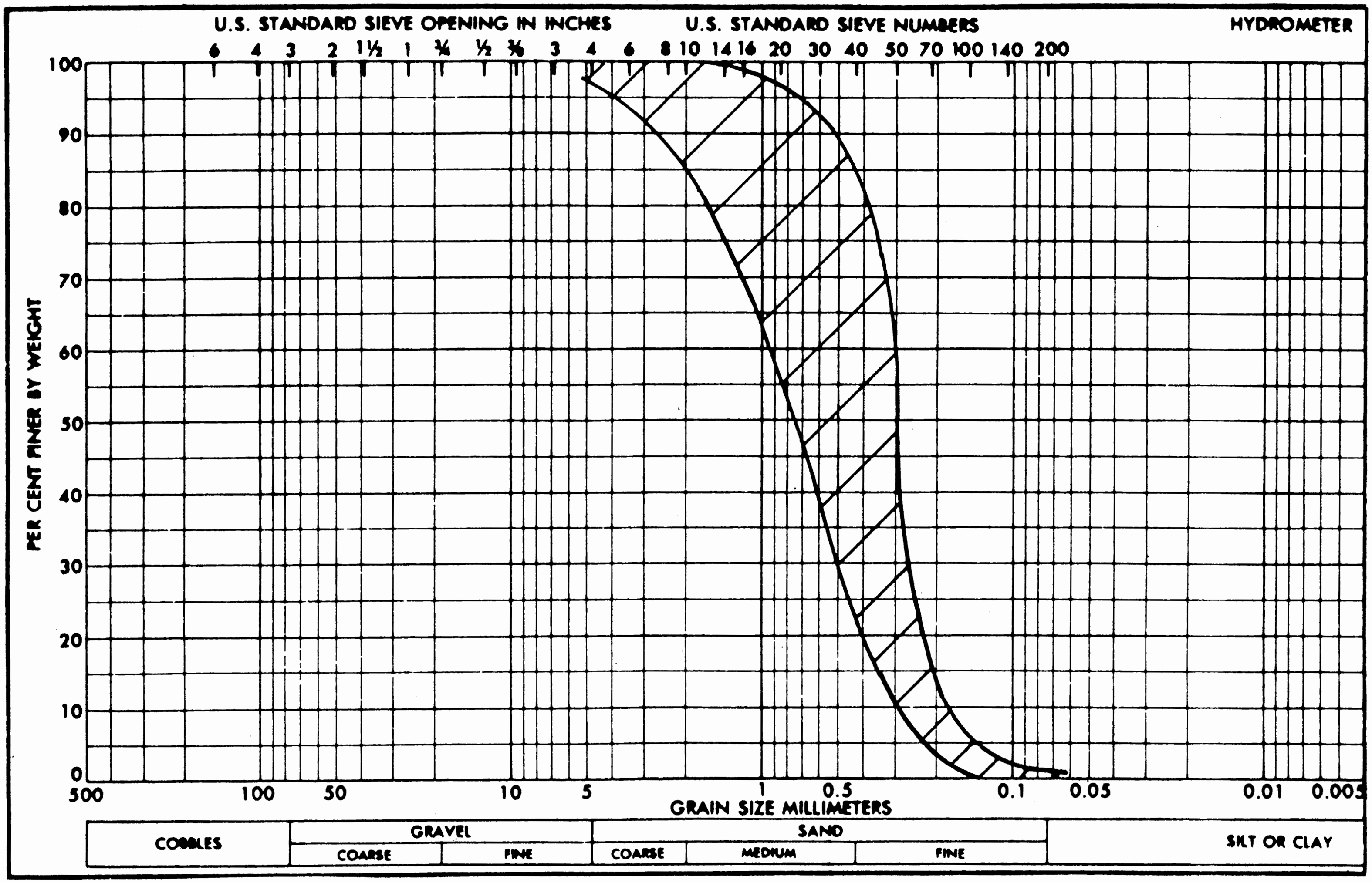



Figure 7 Range of Gradation Curves for Lima Lake Drainage District, 7 Samples.

indicates that the average $D_{10}$ size for zone 3 is about the same or less than the average $D_{10}$ size for zones 1 and 2. The maximum difference in the average $D_{10}$ size for any one levee reach ranges from 0.04 to 0.09 mm. Thus, the sand from the bottom discharges is as coarse or coarser than the sand from the end discharge. However, the difference is small and the opinion of the dredge contractors concerning the nearly equal distribution of fines is borne out by the gradation tests results. Similarly, the average hydraulic conductivity, k, for zone 3 is less than or nearly equal to the conductivity for zones 1 and 2. In general, the conductivity of zone 3 is from 10 to 40 percent less than the conductivity of zones 1 or 2. In order to evaluate the significance of this variation, the degree of accuracy for the hydraulic conductivity of natural soils must be considered.

The natural variation of soil deposits makes it difficult to obtain enough samples to determine the exact nature of the in-situ material. Thus, one would expect the in-situ material properties to differ from the material properties determined from the soil samples. The difference depends on the particular material property and must be estimated on the basis of engineering judgement and experience. In the case of hydraulic conductivity, the in-situ value may vary $\pm 5$ times the value determined either from laboratory tests on soil samples or from correlations based on laboratory tests. This is supported by the following comments made by Mansur and Kaufman (1957) in regard to the data shown in Figure 6.

"1. Little agreement was found between permeabilities determined in the laboratory on remolded samples and those obtained from field pumping tests. There is no reason why the permeabilities should have agreed considering that the aquifer was stratified and that lense of coarse sand and fine gravel existed. Generally, the field permeabilities for any given strata exceeded by from 2 to 4 times the permeability as normally determined in the laboratory.

2. A relationship between the effective grain size, $D_{10}$ , and the coefficient of permeability, $k_h$, as determined from flow from individual sand strata is shown in . . . The scatter of points can probably be attributed to variation in uniformity of the grain-size curves and to the fact that the values of $D_{10}$ used in the plot are based on the average of relatively few values of $D_{10}$ for each sand stratum tested." (pp. 995-996).

In view of the above statements, the test results show that the hydraulic sand fill in zones 1 and 2 is slightly more pervious than in zone 3 but the difference is small and therefore the levees are homogeneous from a functional standpoint.

Owing to the importance of the through seepage performance of the levees, a full scale test levee was constructed. The purpose of the test was to evaluate the field performance of the levees and to select a landside slope that would be stable against seepage and erosive forces. A 1000 foot long reach of levee was constructed in the Drury Drainage District about 5 miles downstream from Lock and Dam No. 16 (U.S. Army Corps of Engineers, 1962). The test levee was founded on impervious alluvium and offset from an existing clay levee so that a pool of water could be maintained between the two levees. The height of the existing levee was increased using sand and the entrance face was lined with polyethylene geomembrane to prevent through seepage and instability of the relatively steep (1.0 V on 2.0 H) exit slope. Figure 8 is a view of the test levee from the upstream (Mississippi River) end immediately after construction. Figure 9 is a view from the downstream end of the levee after the pond was filled with water. The test levee had a crown width of 12 feet and an approximate height of 16 feet.

The 1000 foot reach was divided into four reaches with different cross-sections with landslide slopes ranging from 1.0 V on 4.2 H to 1.0 V on 4.9 H. A line of three observation wells was installed near the center of each of the four sections. The observation wells were 1-1/2 inch diameter, i.e., 2 foot long well points with No. 60 mesh screen and 1-1/2 inch diameter galvanized steel pipe for risers.

The sand for the test levee was dredged from the Mississippi River. The construction of the levee was typical of the hydraulic sand fill placement expected on future projects. Typical gradation curves were taken from the riverside toe, crown and landside toe of test section. These results show the fill to be a uniform medium to fine sand with a $D_{10}$ size of 0.20 mm. Thus, the gradation tests indicate the test levee to be homogeneous and isotropic. Based on the correlation shown in Figure 6, the average hydraulic conductivity is equal to 0.2 ft/min.

There was no evidence of slope instability noted during the test and the landside slopes (ranging from 1.0 V on 4.2 H to 1.0 V on 4.9 H) were considered adequate in this respect. However,

Figure 8 View of Drury Test Levee from Upstream End after Construction. (From U.S. Army Corps of Engineers, 1962).

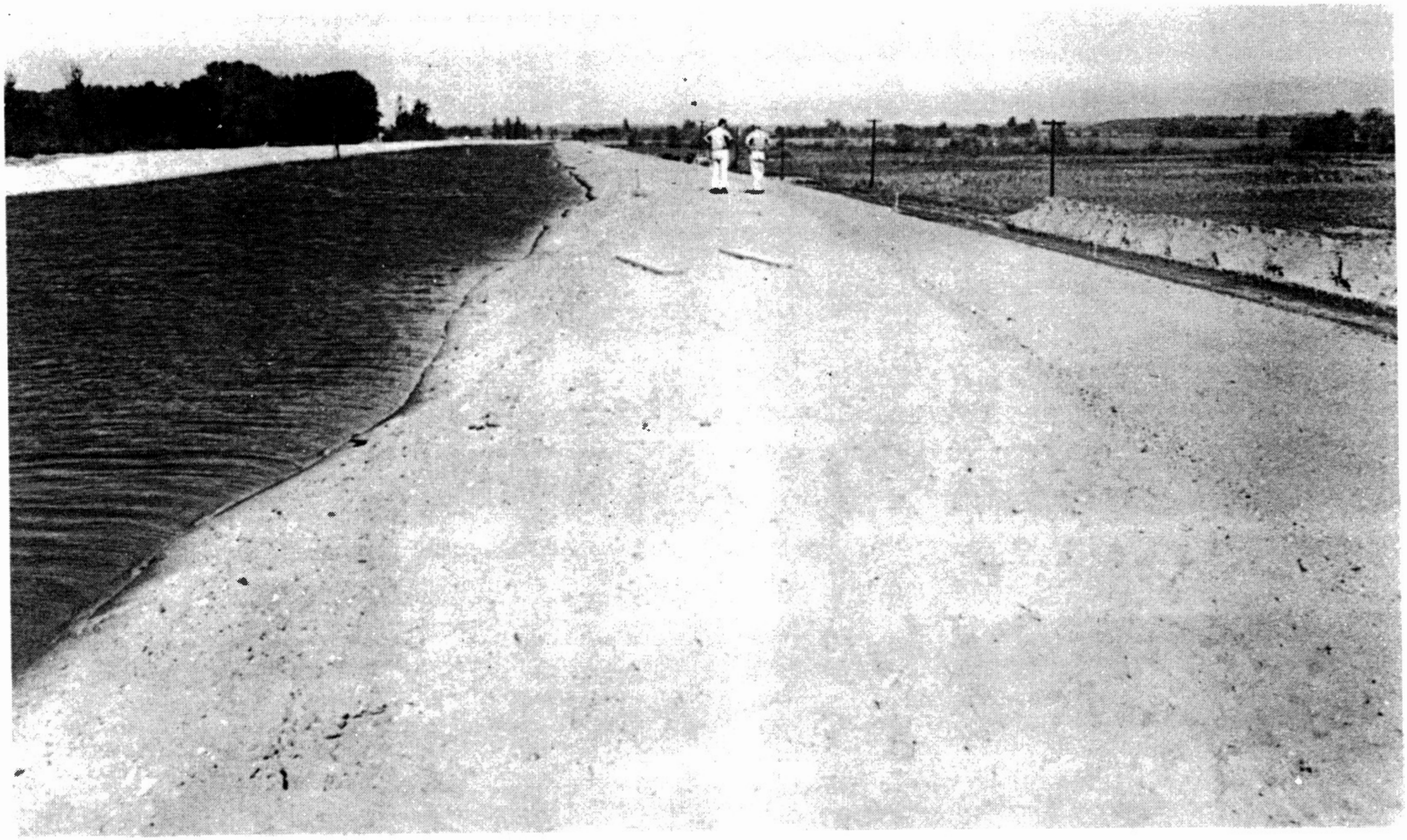


Figure 9 View of Drury Test Levee from Downstream End During Test. (From U.S. Army Corps of Engineers, 1962).

there was surface erosion of the sand on the landside slopes due to seepage emerging on and flowing down the slope. Figures 10 through 13 show the erosional pattern for each test section. These figures show that the relative amount of erosion is proportional to the steepness of the slope. The erosional pattern varies from individual rills with minor surface erosion for the 1.0 V on 4.9 H (figure 10) slope at test section No. 2 to gullies about 1.5 feet deep and 6 feet wide for the 1.0 V on 4.2 H slope at test section No. 3 (figure 13). These gullies are formed by the seepage pattern concentrating at a particular area during development of the erosional pattern. Based on the performance of the test sections, it was concluded that the levee section should have a 1.0 V on 4.0 H riverside slope, a 10 foot crown width, and a 1.0 V on 5.0 H landside slope. The slope stability and erosion of the landside slope will be discussed later in this report.

Since no flow measurements were taken during the levee tests, the values of the average hydraulic conductivity computed from the grain sizes curves could not be checked with field values. However, the observation well data were used to evaluate the homogeneity and isotropy of the sand levee. A finite element analysis (Schwartz (1976)) of the levee test sections coupled with the observation well data indicated a homogeneous levee section. The maximum difference between the computed and measured heads for homogeneous conditions varied from 0.06 ft for a conductivity ratio ($k_h/k_v$) of 1 to -0.06 ft for a $k_h/k_v$ of 4. Based on this data and analysis, the hydraulic sand fill was

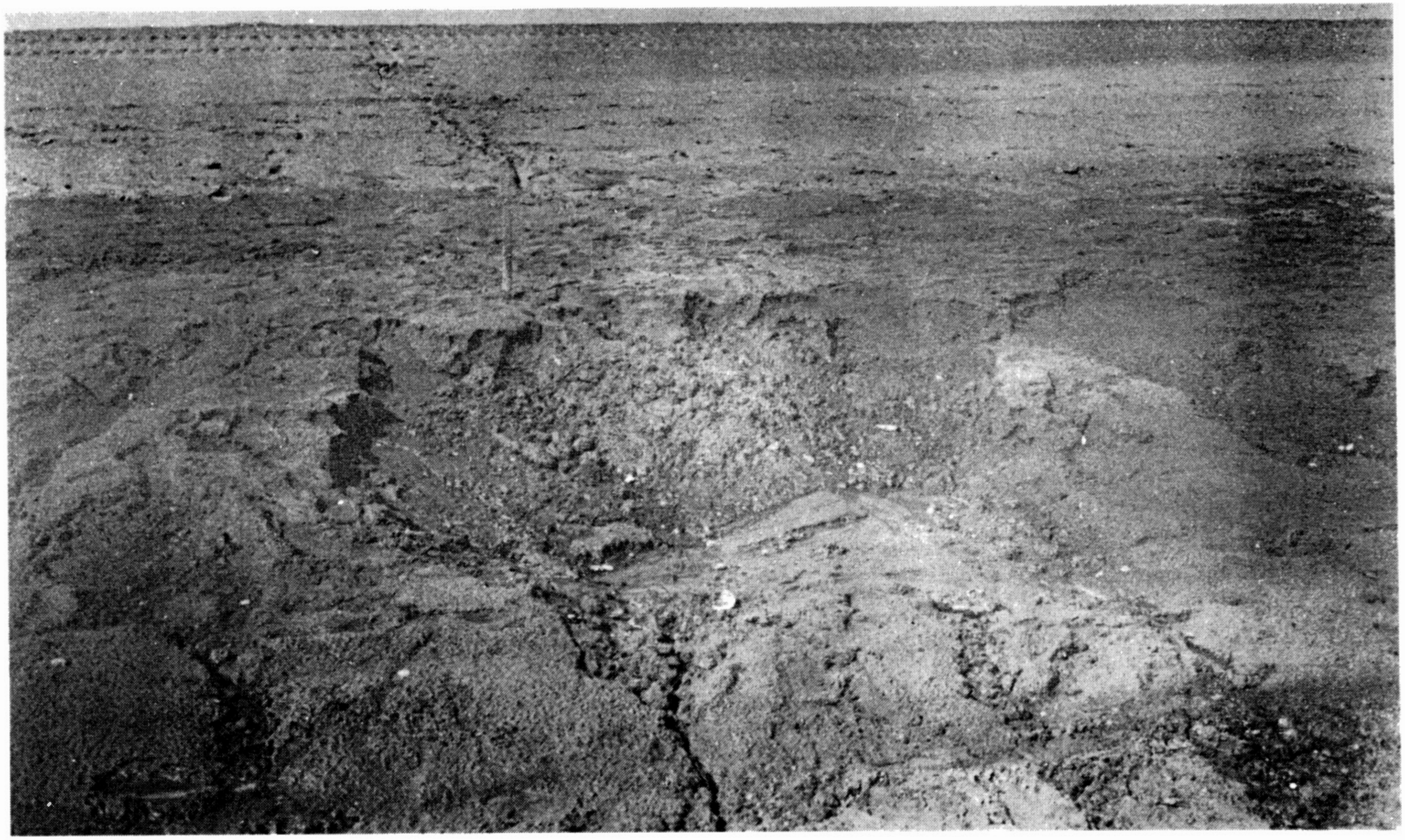


Figure 10 Landside Slope Erosion at Drury Test Section No. 1. (From U.S. Army Corps of Engineers, 1962).

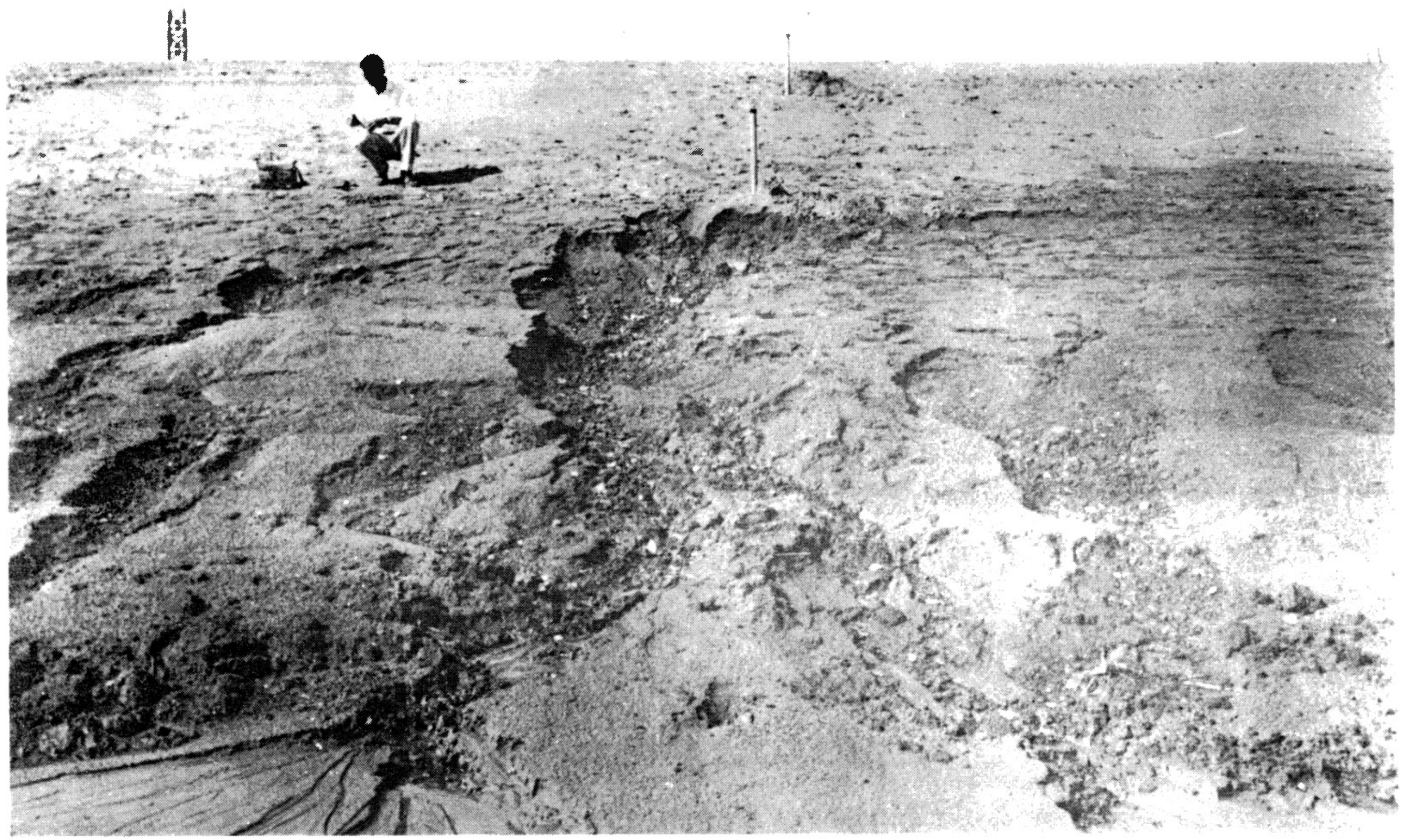


Figure 11 Landside Slope Erosion at Drury Test Section No. 2. (From U.S. Army Corps of Engineers, 1962).

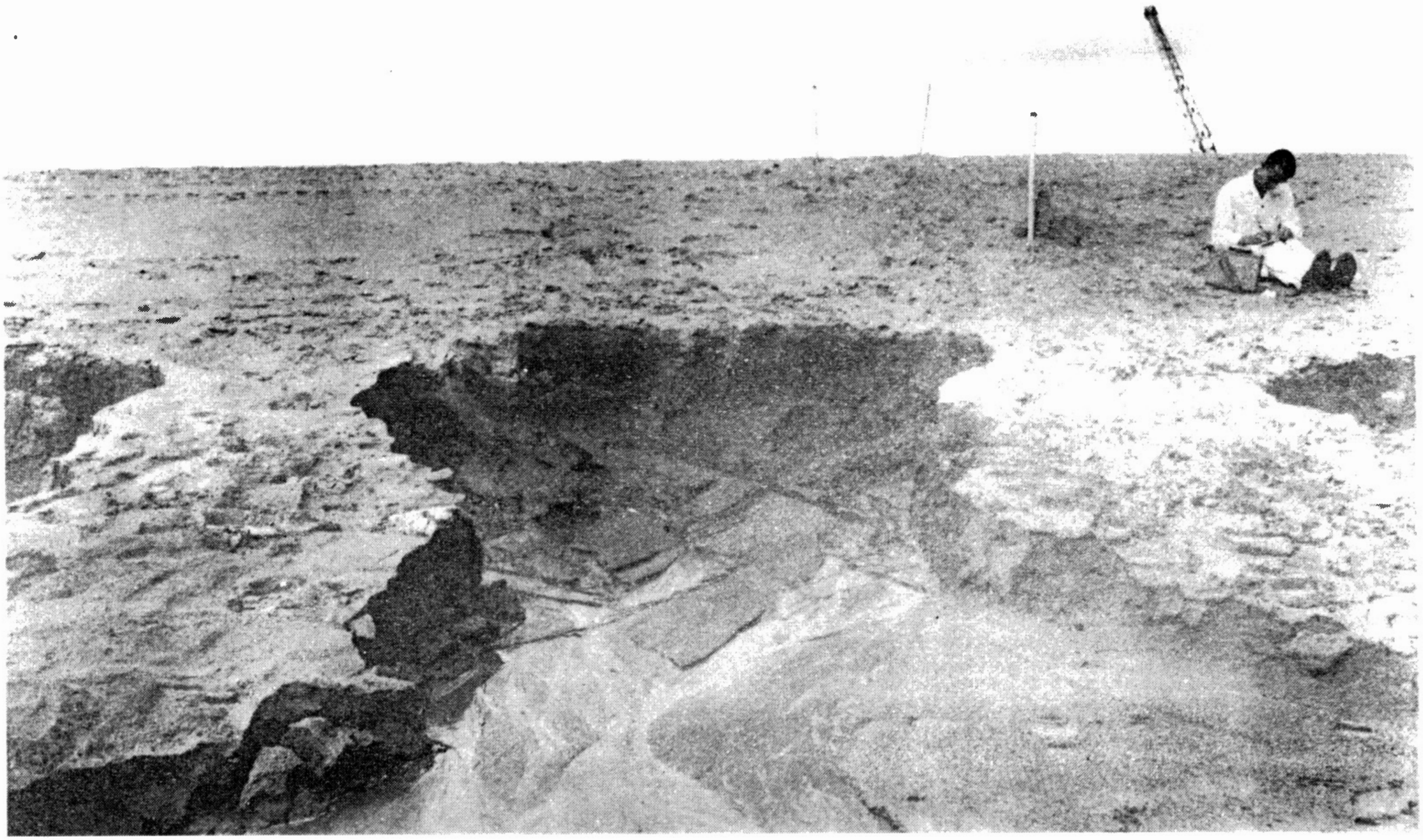


Figure 12    Landside Slope Erosion at Drury Test Section No. 3.
(From U.S. Army Corps of Engineers, 1962).

Figure 13 Landside Slope Erosion at Drury Test Section No. 4. (From U.S. Army Corps of Engineers, 1962).

considered to be homogeneous and isotropic with respect to hydraulic conductivity. The horizontal to vertical conductivity may vary from 1 to 4.

## SLOPE STABILITY ANALYSIS

The purpose of this section is to show the effect of the seepage on the stability of the landside slope. Since the impervious base and partial clay cores are outside of the flow domain, the analyses are confined to the hydraulic sand fill. The stability analyses are basically limiting equilibrium methods using a free body. The body forces of a typical free body element of soil are the weight of the soil grains and the weight of the water in the soil. These forces are in equilibrium with the boundary forces consisting of the water forces, the effective normal forces and the shear forces. The shear force required along the failure plane to maintain equilibrium is determined in the stability analyses. The relative stability of the slope is measured by computing the portion of the available soil strength that is required to develop this shear force.

In general slope stability terminology, the study is an effective stress analysis of cohesionless materials subject to seepage forces. The particular method employed is the infinite slope method (Cedergren, 1967; Lambe and Whitman, 1969; and U.S. Army Corps of Engineers, 1970). The infinite slope is the method used to analyze the sections.

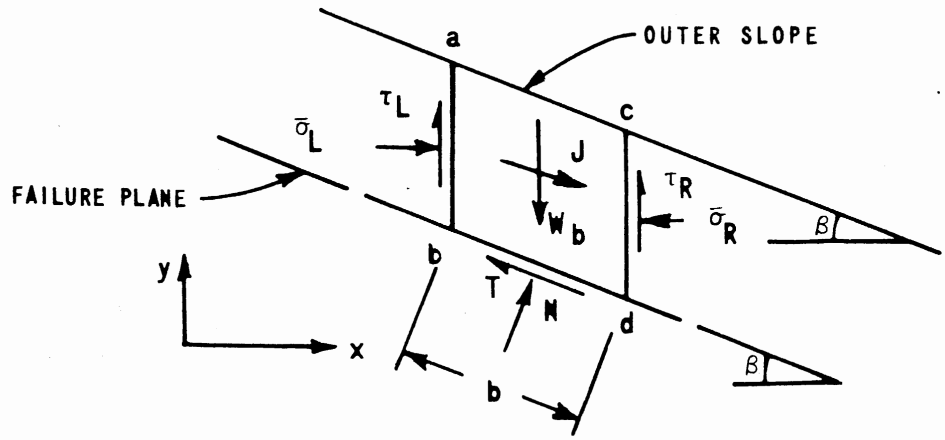
For cohesionless materials such as hydraulic sand fills, the slopes tend to fail along relative shallow failure surfaces which are parallel to the slope. If the slope is infinitely long and the soil conditions are uniform, the stability of the slope can be analyzed by considering the equilibrium of a free body consisting of a vertical slice bounded by the surface of the slope and the failure surface parallel to the slope. Figure 14 shows a slope with a typical free body slice abcd. This method can be used to approximate the stability of a finite slope if the following assumptions are made.

1. The effective normal and shear stresses on the left side of the slice ( $\bar{\sigma}_L$ and $\bar{\tau}_L$ ) are equal to the effective normal and shear stresses on the right side of the slice ( $\bar{\sigma}_R$ and $\bar{\tau}_R$ ).

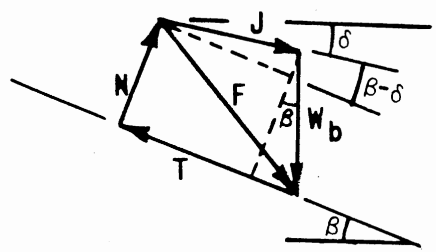
2. The material properties of the soil are uniform throughout the slope. If the slope is subject to seepage, the seepage pattern must also be uniform throughout the slope.

The method is termed the infinite slope method and is documented in detail by Cedergren (1967) and Lambe and Whitman (1969). In general, the free body represented by the slice abcd is assumed to be held in limiting equilibrium by the effective normal force, N, and the shear force, T acting on the base of the slice. Figure 15 provides equations used in the analysis.

Slope stability analyses have been used in conjunction with seepage analyses to determine slope stability of hydraulic sand fill levees. The infinite slope stability method is considered to be an applicable method. The actual factor of safety required for stability is based on engineering judgement and performance




a. Free Body Element.




b. Force Diagram.

Figure 14 Sketch of Free Body Element and Force Diagram for an Infinite Slope Stability Analysis.

## Infinite Slope Analysis for Cohesionless Soils

1. Infinite Slope Computations. For cohesionless materials $(c = 0)$, equations applicable to an infinite slope may be used to obtain an estimate of the stability of the slope of an embankment where seepage is involved. It is assumed that the seepage flow is uniform throughout the soil mass.

2. General Case. The safety factor for the general case where seepage flow is neither parallel nor horizontal to the outer slope is

$$F.S. = \frac{\gamma' - (\gamma_w \frac{\tan \alpha}{\cot \beta})}{\gamma_{sat}} \cot \beta \tan \phi$$

where

$\gamma'$ = submerged unit weight of soil

$\gamma_w$ = unit weight of water

$\alpha$ = angle between seepage flow line and embankment slope

$\beta$ = angle of inclination of embankment slope with horizontal $(\cot \beta = b)$

$\gamma_{sat}$ = saturated unit weight of soil

$\phi$ = angle of internal friction

3. Seepage Parallel to Slope. For seepage flow parallel to and coincident with the embankment slope $(\alpha = 0)$, the safety factor becomes

$$F.S. = \frac{\gamma'}{\gamma_{sat}} \cot \beta \tan \phi = \frac{\gamma'}{\gamma_{sat}} b \tan \phi$$

where

$b = \cot \beta$

4. Horizontal Seepage. Where seepage flow is horizontal $(\alpha = \beta)$, the factor of safety is

$$F.S. = \frac{\gamma' - \frac{\gamma_w}{\cot^2 \beta}}{\gamma_{sat}} (\cot \beta \tan \phi) = \frac{b^2 \gamma' - \gamma_w}{b \gamma_{sat}} (\tan \phi)$$

**Figure 15 Infinite Slope Stability Analysis for Cohesionless Soils.**

5. <u>No Seepage.</u> Where no seepage forces exist, i.e. for a dry slope, the factor of safety is

$$F.S. = \frac{\tan \phi}{\tan \beta} = b \tan \phi$$

6. <u>Earthquake.</u> The effects of an earthquake loading can be applied to all of the previous equations for factor of safety by replacing b with the term b' where

$$b' = \frac{b - \psi}{1 + b_{\psi}}$$

$\psi$ = seismic coefficient (see fig. 6, main text)

7. <u>Example.</u> An example of the influence of the direction of seepage flow on the factor of safety is illustrated in the following tabulation.

| Assumed design values | Factor of safety for: Seepage parallel to outer slope | Horizontal seepage | No seepage |
|---|---|---|---|
| $b = 3.5$ | | | |
| $\gamma_{sat} = 2\gamma_w$ | 1.23† | 1.13† | 2.45† |
| $\tan \phi = 0.7$ | | | |
| $\psi = 0.1$ | 0.88†† | 0.74†† | 1.76†† |
| $b' = 2.52$ | | | |

† Without earthquake loading.
†† With earthquake loading.

Figure 15 Infinite Slope Stability Analysis for Cohesionless Soils (cont'd) (From U.S. Army Corps of Engineers, 1970).

of existing earth structures. The U.S. Army Corps of Engineers (1970) recommends a minimum factor of safety of 1.50 for steady seepage through earth dams and levees. For a 1.0 V on 5.0 H landside slope, the factor of safety varies from 1.55 at the exit point of the free seepage surface to 1.50 at the landside toe. Thus, these levees should have adequate stability.

The normal design procedure in the Rock Island (Illinois) District is to use a 1.0 V on 5.0 H landside slope with the partial clay core located at or riverward of the levee center-line. These are the conditions that give a factor of safety equal to or greater than the recommended minimum of 1.50 and the levees should have adequate stability. However, it should be noted that stability analyses were not the basic factor used to establish the design procedure. The selection of the landside slope was based on the field performance of the various levee sections at the Drury test section with respect to landside slope erosion with the slope stability analyses verifying that there is adequate stability.

## EROSION ANALYSIS

In this section, the surface erosion on the landside slope of a hydraulic sand fill levee is presented. The levee is assumed to be homogeneous and isotropic, and may contain a partial clay core. The erosion is due to through seepage exiting on and flowing down the slope. The region of concern is the surface of the free discharge face. As discussed earlier a typical sand levee cross section used in the Rock Island District contains an abnormality in the seepage control design in that a toe drainage system is not provided to prevent the free seepage surface from exiting on the landside slope. Thus, the surface soil particles are acted upon by gravitational, seepage and hydrodynamic surface shear forces.

The mechanics of the erosional phenomena were developed by Schwartz, (1976) and expressed the through seepage exiting on the slope as an equivalent spatially varied surface flow. The hydrodynamic equations for an increasing spatially varied flow given by Chow (1959) for open-channels, and for overland or sheet flow given by Izzard (1944) for runoff on a plane surface as a result of rainfall, were modified to obtain the surface shear stress developed from the seepage water flowing down the slope. This surface shear stress is the interfacial shear stress developed along the soil-surface water boundary. The equilibrium of the surface soil grains under gravitational, seepage and surface shear forces is analyzed to determine the critical surface shear stress in a similar manner as Chow (1959) analyzes the stability of open-channels. The critical surface shear stress is defined as the shear stress that causes impending motion. Chow (1959) terms this shear stress the permissible unit tractive force. The resulting critical stress is expressed in terms of the levee geometry and through seepage parameters in order to obtain a measure of the erosion susceptibility of the landside slope.

The erosion susceptibility parameters are based on the shear stress at impending motion because the flow pattern becomes extremely complex once erosion is initiated. The complexity of the flow problem once erosion has started was summarized by Chow (1959) while studying open-channel erosion:

> "The behavior of flow in an erodible channel is influenced by so many physical factors and by field conditions so complex and uncertain that precise design of such channels at the present stage of knowledge is beyond the realm of theory. The uniform-flow formula, which is suitable for the design of stable nonerodible channels, provides an insufficient condition for the design of erodible channels.
>
> This is because the stability of erodible channels, which govern the design, is dependent mainly on the properties of the material forming the channel body, rather than only the hydraulics of the flow in the channel." (p. 164)

The maximum erosion susceptibility (M) and the relative erosion susceptibility (R) can be used to determine the probable erosional distress of the landside slope. The expressions for the susceptibility parameters are as follows (from Schwartz (1976)).

$$M = \frac{\lambda_2 \quad y_e^{0.6}}{\lambda_1 \quad \tau_{co}}$$

$$R = \frac{y_e - \left(\frac{\lambda_1 \quad \tau_{co}}{\lambda_2}\right)^{1.67}}{H}$$

where:

$$\lambda_1 = \cos\beta - \frac{\gamma w}{\gamma b} \sin\beta \tan\ (\beta - \delta) - \frac{\gamma sat}{\gamma\ b} \frac{\sin\beta}{\tan\phi}$$

$$\lambda_2 = \gamma w \ \sin^{0.7}\beta \left(\frac{n}{1.49}\right)^{0.6} \left|K \tan\ (\beta - \delta)\right|^{0.6}$$

$\beta$ = angle of inclination of landside slope

$\delta$ = angle of inclination of hydraulic gadient with a positive value measure clockwise from the horizontal

$\phi$ = angle of internal friction for sand

K = coefficient of permeability in ft/sec

n = Manning's coefficient of roughness for sand

$\tau_{co}$ = critical surface shear stress (i.e., interfacial stress at soil-water boundary at impending motion) for a horizontal surface without seepage forces in p.s.f.

$y_e$ = vertical distance between the exit point of free seepage surface and the impervious base in feet

$\gamma w$ = unit weight of water in lb/cubic foot

$\gamma b$ = bouyant unit weight of soil in lb/cubic foot

$\gamma sat$ = saturated unit weight of soil in lb/cubic foot

H = height of hydraulic sandfill levee in feet

The seepage exiting on the free discharge face of a sand levee has been expressed as an equivalent increasing spatially varied surface flow. Considering both seepage forces and the surface shear force, an expression for the critical surface shear stress has been developed. By comparing the developed surface shear stress to the critical, two erosion susceptibility parameters are proposed to indicate the probable erosional distress of the levee.

Computations for typical sand levees (1.0 V on 5.0 H landside slope and $\delta = 0$) indicate that the critical shear stress is exceeded (i.e., $M > 1.0$) about 3 feet down the slope from the free seepage surface exit or a vertical distance of about 0.6 feet below the exit point. Thus, theoretical design criteria for no erosion of the slope would require the exit point of the free surface to be less than a foot above the impervious base.

Field observations on the condition of the landside slopes with respect to erosion was compared to the two erosion susceptibility parameters. This comparison was used to show that the susceptibility parameters are valid indicators of the actual performance of the slope.

The above correlation was used to establish design criteria for both slope stability and erosion susceptibility. And, a design procedure formulated to select the optimal levee section with respect to landside erosion.

The performance of the levees was documented during high waters on the Mississippi River in the Spring of 1965, 1969, and 1973. The levees were grouped according to drainage districts and were inspected as the peak of the high water hydrograph passed each district. The observations were made by personnel of the Foundations and Materials Branch of the Rock Island (Illinois) District. These personnel were divided into inspection teams consisting of two people. The individuals comprising an inspection team were interchanged so that each knew what terminology was being used by the others to describe the condition of the as-built slopes. The purpose of this procedure was to gain uniform and unbiased descriptions of the relative condition of the slope. The levees were inspected continuously along their length and were divided into lengths having the same observed slope condition. The observations were documented on the plan and profile sheets of the construction drawings for each drainage district.

The field performance of the levees was correlated with the theoretical susceptibility parameters in order to test their validity as indicators of the slope condition. This correlation indicated that the levees will have acceptable performance if $M \leq 5.0$ and $R \leq 0.30$. The levees will have inadequate through seepage control if $M \geq 11.0$ and $R > 0.80$.

## SUMMARY OF CURRENT CRITERIA AND DESIGN

The anticipated seepage through hydraulic sand fill levees is analyzed and the analytical results are compared to the actual performance of as-built levees constructed by the U.S. Army Corps of Engineers, Rock Island (Illinois) District in the upper Missi-ssippi River basin. The hydraulic sand fill levees are unique in that they do not contain a toe drainage system to prevent the free surface of seepage from existing on the landside slope. The improved sand levees generally range from 15 to 25 feet in height.

The physical problem is a two-dimensional, steady state, free surface flow of an incompressible fluid through an incompressible, saturated porous medium. The porous medium is a hydraulic sand fill levee with a 1.0 V on 4.0 H riverside slope, a 10 foot crown width and a 1.0 V on 5.0 H landside slope. The sand levee rests on an impervious foundation.

Finite element analyses were used in conjunction with data from a full scale test levee to establish the material properties of the sand levees and to determine the exit point of the free seepage surface, the quantity of through seepage and the exit gradients along the free discharge face. Since the free seepage surface is a boundary of the flow domain and its location is not known a priori, an iterative procedure is required for the finite element seepage analyses. A parametric study was performed to establish dimensionless design charts suitable for determining the free seepage surface exit point, the quantity of seepage and the exit gradients, which are now used. The effect of the seepage forces on the stability of the landside slope is analyzed. This is generally accomplished using the infinite slope analysis.

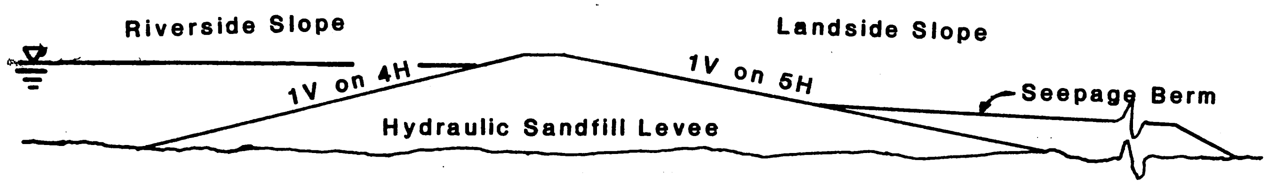
The erosion susceptibility of the landside slope is then analyzed. This analysis considers both the internal seepage forces and the surface shear forces resulting from the seepage exiting on and flowing down the slope. Two parameters are formulated to measure the erosion susceptibility of the slope.

Using the field performance of approximately 100 miles of levees, a correlation was established between the field performance and the fundamental parameters of the theoretical analyses. Design criteria for the slope stability and erosion susceptibility were established from this correlation. Computed erosion parameters are compared to this correlation.

## CURRENT THROUGH SEEPAGE CONTROL MEASURES

The current through seepage control measures considered by the Rock Island District, U.S. Army Corps of Engineers in the event the theoretical erosion susceptibility parameters exceed the acceptable design criteria are: (a) granular material of sufficient size (crushed stone) placed over the 1V on 5H landside slope, (b) a compacted impervious riverside face tied into the alluvial foundation, or (c) reduction of the landside slope length by the addition of a berm. These control measures are shown on Figures 16 and 17.

The first project in which through seepage control measures were considered, beyond the 1.0 V on 5.0 H slope resulting from

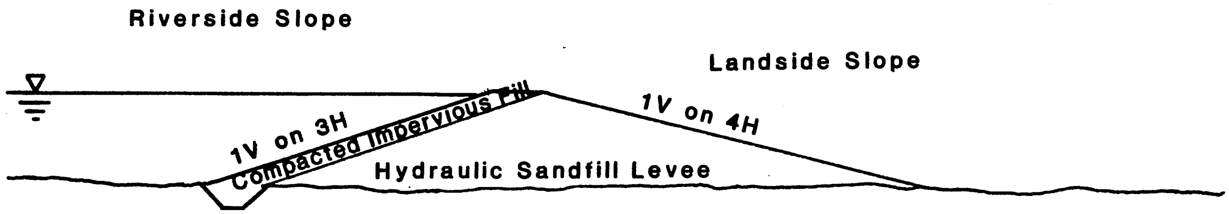



Seepage berm used to control through seepage erosion by reducing the length of the 1V on 5H landside slope and thus, preventing erosion rills from initiating. The berm may also be required to control underseepage.


Use of Granular Material (Bedding) of sufficient size to resist external shear force acting along the interface of the flowing water and the slope.

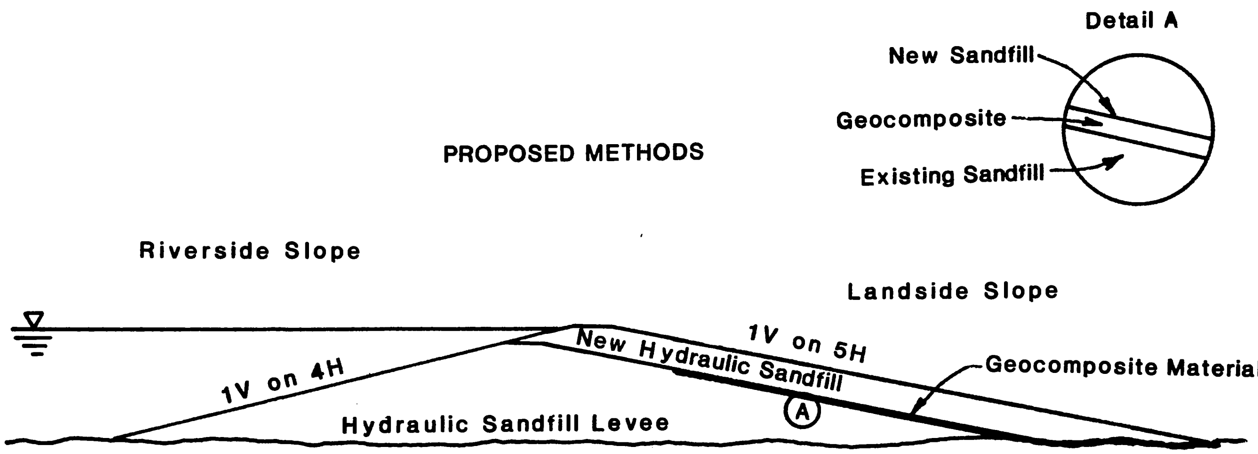
Figure 16 Current Through Seepage Control Measures.

CURRENT METHODS




Composite section with compacted impervious material sealing the riverside face of the levee i.e., prevent through seepage from occurring.

PROPOSED METHODS




Use of Geocomposite Material in levee raise projects to act as an internal drain thus preventing the free surface from emitting on landside slope.

Figure 17 Current and Proposed Through Seepage Contol Measures.

the Drury Drainage District test section, was the Fulton Local Flood Protection Project (1977). The Drury test section had a levee height of 16 feet with the results being assumed applicable for levees of greater height. Based on additional work and performance observations of existing levees, as discussed earlier, this certainly was not the case, but fortunately the majority of existing levees were in the 16-18 feet height range and therefore considered safe from adverse through seepage effects. The Fulton project, for some reaches of levee, was approaching levee heights of 22-23 feet, and using erosion analysis procedures, unacceptable erosion due to through seepage would occur on the landside face of the levee. Since a portion of this levee was going through an urban area, a compacted impervious riverside face was considered the best solution. The granular fill on the landside slope was discounted due to the possibility of the material being disturbed by four-wheel drive vehicles (maintenance problem) as well as providing a source of stones which would be used for vandalization of businesses in the area by area children. A berm to reduce the length of slope subject to erosion was considered, but quantities of sand available for dredging from the Mississippi River were in short supply and addition of features requiring additional borrow was not a viable option. In the analysis performed for inclusion into the Fulton project design documents, a general criterion (rule of thumb) was developed that for a levee of maximum height of 18 feet, no additional through seepage control is required.

Currently, the Rock Island District is studying raising existing hydraulic sandfill levees along the Mississippi River. The first of these projects being analyzed is the South Quincy Drainage District, Illinois (1986). The existing system there was constructed in the 1960's and consists of about 8.8 miles of levees. These structures afford a 50-year level of flood protection to the area. In general, the existing hydraulic sandfill levees were built with 1.0 V on 4.0 H riverside slopes and 1.0 V on 5.0 H landside slopes. Of the many solutions reviewed by the District to provide the greatest net benefits to the area, the 500 year level of protection was selected. This plan entails raising the nearly 8.8 miles of existing levee 3 to 4 feet with hydraulic sand fill obtained from the Mississippi River, and placing this material on the crown and landside slopes. Construction would be limited to the landside slope, since there are environmental concerns of the dredge effulent effects on fish life in the area.

This raise of the South Quincy levee system, in general, means the improved embankments will reach heights of 18-22 feet along the main stem of the system. This increase puts the project now in the area of concern due to erosion caused by through seepage.

The levees were analyzed to determine if any problem existed and from the analysis, erosion problems would be expected from Station 75+00 to station 139+00 or 6400 feet of levee. Since borings showed enough borrow material available, sand berms to reduce the length of landside slope subject to erosion would be provided. This translated into an additional 25,000 cubic yards of material which would be required. A compacted impervious riverward face, although by far the best solution, is impractical due to the length of haul distance of impervious material borrow areas. This is also the major reason the Rock Island District investigated (Drury Test Section) the use of hydraulic sandfill materials in the early 1960's because of the great cost of obtaining impervious materials.

In all probability, the only methods which will be utilized in projects currently undertaken by the District to prevent erosion due to through seepage will be berms or riverside impervious faces. The granular material concept, although theoretically correct, the landside slope faces of levees, due to there relative flatness (1.0 V on 5.0 H) are used by four-wheel drive vehicles which subject the slopes to distress and would disrupt the granular materials. This could lead to localized high through seepage concentrations creating a situation more critical than had nothing (no crushed stone) been placed on the slope.

Considering the levee raise issue will surface for nearly all the existing mainline levees within the Rock Island District, nearly 200 miles, it is imperative that any new through seepage control/erosion control measures be considered beyond those already available. These methods need to be developed and compared to current methods to see if they are a cost effective alternative. It appears geotextile and geocomposite materials can provide this alternate solution.

## THROUGH SEEPAGE CONTROL MEASURES USING GEOCOMPOSITES

As noted, the hydraulic sand fill levees are unusual in that they contain no provision for control of seepage through them. This leads to erosion problems of the landside slope due to the water emitting on and flowing down the slope, when enough of the slope is subject to the flowing water. Generally, when levees exceed about 18 feet in height. It is suggested that a geocomposite material could remedy this situation.

Since construction will be accomplished using hydraulic sand fill for upcoming levee raise projects it is suggested that prior to fill placement a geocomposite be installed on the landside slope. This material would be a geotextile fabric on both sides acting as a filter and having a large open area in between to drain the water away. Using this approach the free surface developed through the sandfill material would be kept within the section and thus preventing erosion of the landside slope. Figure 17 shows the recommended location within the levee section.

Using information provided in Christopher and Holtz (1985), design of the geotextile to act as a filter can be readily accomplished. Geocomposite drainage characteristics can be obtained from manufacturers or tested and compared to values obtained for the through seepage amounts to be anticipated for the levee section. This through seepage value can readily be obtained from design charts currently used to determine values for incorporation into interior drainage reports and pump plant requirements. The geocomposite can exit at a drainage ditch and water routed to pump plants, which is quite similar to what is done currently, i.e., drainage ditch provided for water flowing down slope. Appendix 1 is a typical geotextile design for the material for use in the geocomposite for one of the existing hydraulic sandfill levees. Figure 7 shows the gradation curve used in the design.

Use of this proposed section would also have additional benefits of making the slope more stable since the seepage surface is kept within the embankment section. However, an analysis must be performed based on the friction value between the fabric and sandfill materials obtained from tests, but with the relatively flat slopes this is not anticipated to be a problem. For relatively large levee raises, the possibility exists that a steeper landside slope can be utilized, saving quantities of needed hydraulic sandfill materials. Although protection of the material due to four wheel drive vehicles must be considered, i.e., depth of cover must be 1-2 feet. This is not expected to be a major problem since levee raises are typically expected to be in the 3-4 feet range, as was selected for the South Quincy Levee Raise project (1986).

Construction consideration of this type of section would not be significantly different that what would be expected without the geocomposite, except to place the geocomposite. Since due to environmental concerns of "muddy" water returning to the Mississippi River and killing marine life, all construction would be restricted to the landward slope. The hydraulic sandfill would be deposited at the toe of the existing slope and pushed by the slope to the required section. This would provide protection of the (protected) geocomposite and thus it may only have to specified as a Class A material, Christopher and Holtz (1985), the depth of material over the geocomposite must be considered which would weight from 360 to 480 pounds/square foot (3-4 feet) when crushing of the material is evaluated, and would need to be tested for this weight plus weight of equipment spreading the material. It is anticipated that construction stresses would be the biggest test of the material. Clogging of the geocomposite is not expected to be a problem since it will be placed against the existing slope which can be tested (gradation curves) prior to actual construction, and the sandfill is generally a very clean material, as shown by gradation curves (Figure 5).

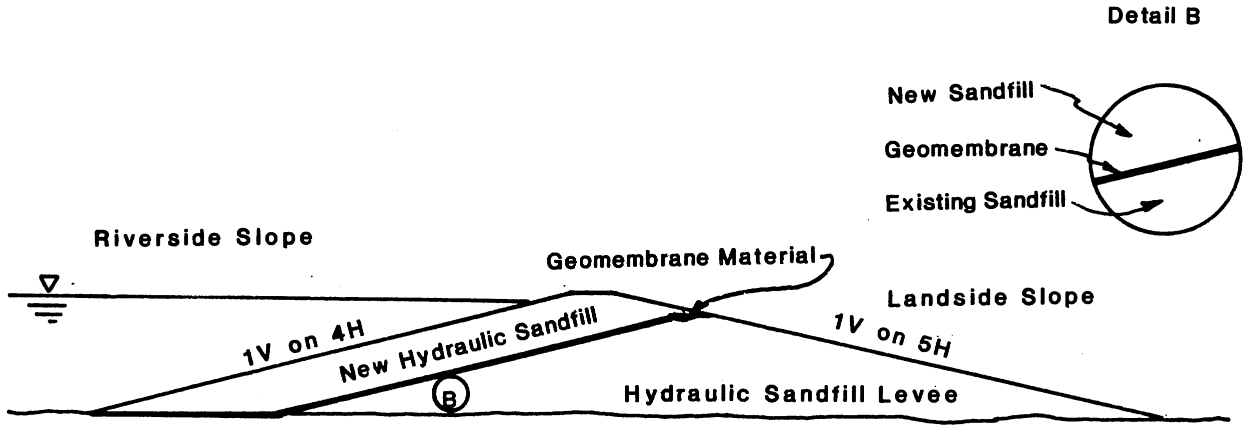
Evaluation of the effectiveness of the geocomposite can be accomplished after construction and during a highwater event by measuring the seepage surface through the levee. This can be done by utilizing existing observation wells that are presently used to measure through and underseepage at existing projects. There are currently about 20 observation well ranges throughout the Rock Island District which are read during highwater events as an aid in determining the performance of the sandfill levees and underseepage berms. An interesting test would be to provide a noncontinuous geocomposite i.e., perhaps highway edge drain material laid perpendicular to the levee centerline and determine the spacing required between strips to ensure the free surface is kept within the levee portion. This would save material quantities. It appears that a geocomposite material is well suited in this type of application, but would require an analysis to determine its cost effectiveness.

It is interesting to note that geotextile/geomembrane could accomplish the same things as currently proposed (used) methods within the Rock Island District. Figure 18 shows these proposed methods. The use of the geomembrane has the greatest potential since through seepage is completely cut-off. If the environmental concern could be alleviated, this would be the method of preference.

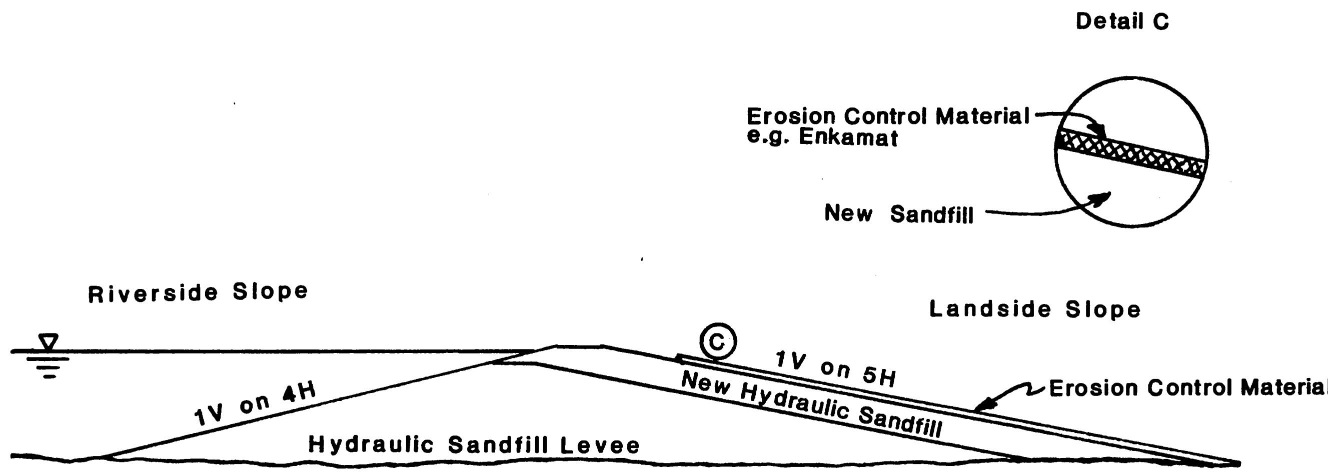
Construction procedures for installation of geomembranes would be similar to those anticipated for use in installing a geocomposite. Critical point of installation would be the tie in to the existing impervious riverside clay blanket as well as extension of the geomembrane to the top of the levee. These two areas would be critical to ensure no seepage undercuts or overtops the membrane.

Evaluation of the geomembrane's effectiveness can be deter-

**PROSPOSED METHODS**




Use of Geomembrane Material in levee raise projects to act as an impervious barrier on the riverside slope i.e., prevent through seepage from occurring.




Use of Erosion Control Material e.g., Enkamat Soil Erosion Matting, to dissipate external shear forces along the interface of the flowing water and the slope.

**Figure 18 Proposed Through Seepage Control Measures.**

mined using existing observation well ranges to measure the seepage surface, which are already in place. Since no seepage will be emitting on the landside slope, consideration can be given to steepening the 1.0 V on 5.0 H slope to possibly offset the additional material needed for the entire levee project. This would alleviate the need to obtain additional dredged materials and the environmental concerns withdrawn.

## SUMMARY AND CONCLUSIONS

The design of structures using geotextiles has progressed greatly in the last few years with design being based on sound engineering principles. These design analysis are now presented in the literature for use by the engineering community, although there is still some resistance in the use of geotextiles (See Appendix 2). Extending the design analysis of hydraulic sand fill levees to include geotextiles (geocomposites). It appears these materials can significantly improve the performance of these structures during highwater events. The peace of mind (an intangible benefit) of seeing sand levees without through seepage emitting on the landside slope is a factor which is hard to quantify and incorporate into a cost analysis.

# APPENDIX 1

## Typical Geotextile Design and Selection Criteria

Typical Geotextile Design and Selection Criteria for Typical Through Seepage Control (Lima Lake Drainage District)

Use "Summary of Minimum Geotextile Design and Selection Criteria for Drainage, Filtration and Erosion Control Applications modified from Tables 3-3, 3-4 and 3-5 in Christopher and Holtz (1985) FHWA Geotextile Engineering Manual."

I. Soil Retention (Piping Resistance) Criteria Lima Lake Drainage Materials are medium to fine sands

| Soil | Steady State Flow |
|---|---|
| < 50% Passing | AOS or $O_{95} = (B)(D_{85})$ |
| U.S. No 200 sieve | For $C_u < 2$ or $> 8$, B=1 |
| | $2 < C_u < 4$, B= $0.5C_u$ |
| | $4 < C_u < 8$, B= $8/C_u$ |

$C_u = D_{60}/D_{10}$ Using smallest soil size $D_{60} = 0.3$ mm

$D_{10} = 0.18$mm

$= 0.3/0.18 = 1.67$ B = 1

AOS = $(B)(D_{85})$ = (1)(0.43mm) = No. 40 sieve

II. Permeability Criteria

(a) Critical/Severe Application

$k_{geotextile} > 10k_{soil}$

(b) Less critical/non severe application

$k_{geotextile} > k_{soil}$

Use relationship "Horizontal Coefficient of Permeability versus $D_{10}$ Size" from U.S. Army C of E,(1956)

$D_{10} = 0.40$mm $k = 3000 \times 10^{-4}$ cm/sec = 0.3 cm/sec.

check using Hazen equation for permeability

$k = (100)(D_{10})^2$ where $D_{10}$ is in cm

$k = (100)(0.040)^2 = 0.16$ cm/sec

Use k = 0.3 cm/sec from graph

## III. Clogging Criteria

a. Suggest soil-geotextile filtration test be run (Gradient Ratio (GR) Test). Suggested maximum criterion $GR<3$.

b. Geotextile with a maximum opening size from retention criteria (I above) shall be specified.

c. Effective Open Area Qualifier
Woven geotextile: Percent Open Area $>$ 4%
Non woven geotextile: Porosity $>$ 30%
AOS or $O_{95} > 3D_{15} = 3(0.35) = 1.05$ mm = U.S. No. 16 sieve

## IV. Durability Criteria

a. Fibers used in the manufacture of geotextile shall consist of long chain synthetic polymers, composed of at least 85% by weight of polyolephins, polyesters, or polyamides.

b. "Non-stabilized" geotextiles, with low resistance to ultra-violet degradation (more than 30% strength loss at 500 hours exposure ASTM D-4355), should not be exposed to sunlight for more than 5 days. Geotextiles with higher resistance to ultraviolet degradation should not be exposed for more than 30 days.

c. The geotextile should not be exposed to any unusual or extreme biological and/or chemical conditions.

## V. Minimum Physical Property Requirements for Constructability and Survivability (Adapted from AASHO-AGC-ARTBA Task Force No. 25 recommendations, see FHWA Manual and ASTM Committee D-35 for details and recommended test procedures).

Notes:

1. All numerical values given represent minimum average roll values. These values are considerably lower than average values or those commonly given in a manufactures literature.

2. Installation conditions and classes.

<u>Class A</u>. More severe installation stresses. In filtration and drainage applications, where very sharp angular aggregate is used, a high degree of compaction is specified, or depth of trench is greater than 10 feet. In erosion control applications, where riprap placement heights are up to 3 feet and stone weights are up to 250 pounds, unless field trails indicate otherwise

satisfactory performance.

<u>Class</u> <u>B</u>. Less severe installation stresses. In filtration and drainage applications, where geotextile is used on smooth graded surfaces having no sharp angular projections, sharp angular aggregates are not used, compaction requirements are light, and trenches are less than 10 feet in depth.

In erosion control applications, where geotextile is used in structures or under conditions where it is protected by a sand cushion or riprap is hand placed (zero drop height).

| Test Method | Class A (Unprotected) | Class B (Protected) |
|---|---|---|
| | (Minimum values in either machine or cross-machine direction) | |
| Grab Tensile Strength (ASTM D-1682, Method 16) | 200 lbs | 100 lbs |
| Elongation at Maximum Tensile Stress | 15% | 15% |
| Puncture Strength (ASTM D-751, modified) | 80 lbs | 40 lbs |
| Burst Strength (ASTM D-3786, Mullen Burst) | 320 psi | 320 psi |
| Trapezoidal Tear (ASTM D-1117) | 50 lbs | 30 lbs |
| Abrasion Resistance | N/A | N/A |
| | (not applicable in this situation) | |

## APPENDIX 2

"Overcoming Psychological Hang-Ups Is Biggest Drainage Challenge"
By: H.R. Cedergren

**CEDERGREN, H. R.**
Consulting Engineer, Sacramento, California, USA

# Overcoming Psychological Hang-Ups is Biggest Drainage Challenge

# Vaincre les préjugés est le problème prioritaire du drainage

Drainage systems, often incorporating synthetic textiles, are helping to make Civil Engineering projects safe from damaging actions of water. Great progress has been made in recent years, but several archaic and unrealistic negative beliefs and attitudes are hampering progress. Overcoming these "hang-ups" is a bigger challenge than designing good drainage systems. Upmost examples are the following: (a) The belief that drainage itself is not necessary, not practical, or too expensive, (b) The belief that nearly every drainage problem can be solved by the use of blends of sand and gravel containing moderate amounts of fines, and (c) A general reluctance on the part of specialists to try any new idea or product that does not have a long experience record. Until these psychological hang-ups can be overcome, the potential benefits of drainage and synthetic textile products in them cannot be fully realized. Meetings such as the 1st International Conference on the Use of Fabrics in Geotechnics and the present one can do a lot to open the eyes of people designing engineering works needing good drainage systems.

Les systèmes de drainage, qui comportent souvent des textiles synthétiques, contribuent à protéger les ouvrages de génie civil des effets destructeurs de l'eau. De grands progrès ont été faits ces dernières années, mais un certain nombre de préjugés archaiques, inexacts et négatifs font encore obstacle au progrès. Surmonter ces idées préconçues est une tâche plus considérable que de calculer de bons systèmes de drainage. Les exemples les plus criants sont les suivants: a) l'idée selon laquelle le drainage lui-même n'est pas nécessaire, pas efficace, ou trop cher; b) la notion selon laquelle tout problème de drainage ou presque peut être résolu au moyen de mélanges de sable et de gravier contenant une fraction modérée de fines; c) la réticence généralisée de la part des spécialistes à essayer toute idée nouvelle et tout produit qui n'a pas des références nombreuses et anciennes. Tant que ces préjugés ne seront pas éliminés, les avantages potentiels de l'emploi des textiles synthétiques dans le drainage ne pourront pas être pleinement reconnus. Des manifestations comme le 1er colloque international sur l'emploi des textiles en géotechnique et comme le présent congrès peuvent faire beaucoup pour ouvrir les yeux de ceux qui établissent les projets des ouvrages de génie civil où un bon drainage est nécessaire.

## INTRODUCTION

Drainage, often with the aid of synthetic textiles, is doing a great deal to make Civil Engineering projects safe from detrimental actions of water. Significant progress has been made in the past few years, but a number of unfounded negative beliefs and attitudes (hang-ups) are greatly interfering with progress, with the result that many engineering works create unnecessary hazards to the public, and deteriorate prematurely from the effects of water. Several prime examples discussed in this paper are the following:

(1) The belief that drainage itself is not necessary, is not practical, or is too expensive is a prime hang-up. This attitude has resulted in widespread practices that eliminate drainage as a design consideration in some important areas of engineering. Two areas are discussed here: (a) One example is the thousands of miles of dikes and thousands of small dams which have been built without drains. All of these structures would be safer with good drains. Upgrading existing structures and providing drains in the ones built in the future would be of great benefit to people in virtually every part of the world. (b) The second example is the common practice of designing pavements as "strong" but undrained systems. In this area alone, a lack of drainage is causing premature damage that is costing taxpayers throughout the world countless billions of dollars a year.

(2) The belief that nearly every drainage problem can be solved by the use of drains constructed of blends of sand and gravel containing not more than 5% of fines (material finer than 0.074 mm (No. 200 sieve) is a hang-up of major proportions. This widespread fallacy is a major obstacle to the use of the textiles in engineering projects requiring drainage, as it implies that they are seldom needed. Single-layer (and even some multiple-layer) drains constructed with sand and gravel blends seldom have the conductivity ($k$ x thickness) needed to accommodate all of the water entering drains and thus protect Civil Engineering works from damaging actions of water. Quantities of water needing to be removed by drains for engineering works should always be estimated by appropriate calculations with Darcy's law and flow nets, or other suitable methods. Such calculations nearly always show the need for a layer of open-graded (narrow size-range) aggregate in the conducting part of a drain. And this mandates the use of some kind of filter--either specially processed good quality aggregate or a suitable textile--to prevent clogging of the open-graded layer.

(3) A general reluctance on the part of specialists of all kinds to try any new idea or product that does not have a long track record has impeded progress in all fields of technical and medical work. It has kept the textiles out of many projects where they might have been of significant benefit. A coordinated educational program is needed to overcome negative attitudes about textiles in drainage systems.

## FIRST HANG-UP:

<u>The belief that drainage is not necessary, is not practical, or is too expensive.</u>

Drains are routinely designed for large earth dams, concrete dams, drydocks, large retaining walls, basements for large buildings, and many other major Civil

Engineering works needing protection from water. But they are almost never provided for many small, "unimportant" structures or to remove surface water entering pavements. An unwillingness to even consider drainage systems for many facilities is a hang-up of major size that can be blamed for huge economic losses and the existence of greater hazards to the public than would exist if the majority of these works were well drained. Examples are:

(a) Levees and Small Dams. Any water-impounding dam or levee that is not provided with a drainage system is susceptible to the development of concentrations of seepage on the downstream slope and beneath the downstream toe, as shown in Fig. 1. Any such structure is

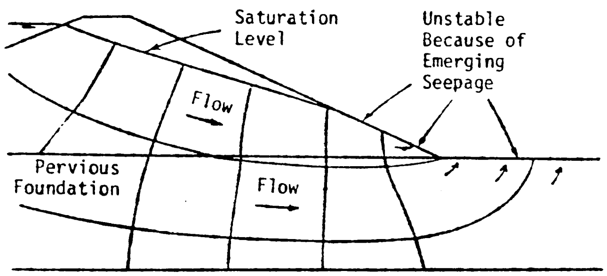



Fig. 1 Cross section through a typical dam or levee having no drainage system.

potentially likely to fail from seepage at some time. Although any dam or levee designer will probably admit that good drainage is a nice idea, he may say it isn't worth the cost for the countless small dams and thousands of miles of small levees around the world. Hopefully as more and more people become aware of the fact that the probability of failure of any dam or levee can be substantially reduced by a drainage system constructed at its landside toe, funds will be made available to upgrade the safety of many of these structures not having drains.

Dams and levees that have not been provided with good drainage systems can fail from seepage with little or no advance warning, because undermining ("piping") from seepage can be occurring without much external evidence. One such small irrigation dam in a Western State had been given its regular annual safety inspection and reported "safe for continued use". That night it failed by piping, culminating years of undermining by seepage that had gone unnoticed. Seepage exit areas that are not covered with good filters and drainage layers can loose significant amounts of materials that are washed away and not even noticed, as was the case with this project, until a break-through occurs and a rapid failure ensues. When all important seepage exits are protected with good filters and surcharged with clean drainage aggregate and gravel fill, failures of this kind can be virtually eliminated because the soil particles are trapped by the filters, and the piping actions are not allowed to start.

Figure 2(a) and (b) shows two applications of geotextiles in drains for levees and small dams. Figure 2(a) shows a substantial toe drain that controls seepage in both the dam and the foundation. A suitable synthetic cloth or filter fabric protects an open-graded drainage layer from clogging, and another fabric keeps dirt out of its upper sides. To insure adequate permeability the open-graded layer should contain no material finer than about 1 cm. size. A pipe at the bottom conducts the seepage to gravity outlets or to sumps for removal by pumps. Most of the volume of the trench can be any stable earth fill, suitably compacted. A drain of this kind can be used for upgrading the seepage safety of countless miles of levees, and thousands of small dams with seepage problems. It is also a good type for new dams or levees to be constructed on permeable foundations.

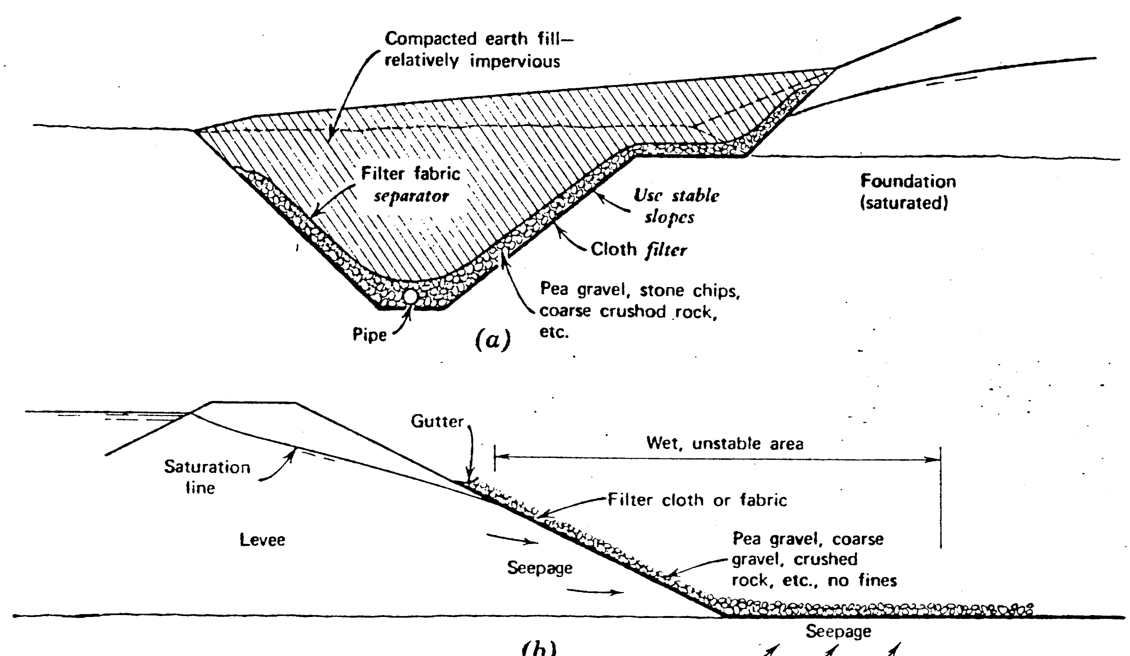



Fig. 2 Illustration of two potential uses for fabrics in drains for dams and levees. (a) A toe drain controlling seepage through dam and foundation, (b) A temporary measure for preventing imminent failure. (After Fig. 5.16 of "Seepage, Drainage, and Flow Nets," 2nd Ed., H. R. Cedergren, Copyright © 1977, John Wiley & Sons, Inc. N. Y. Reprinted by permission of John Wiley & Sons, Inc.)

Figure 2(b) shows a type of filter blanket that can be used to provide protection to levees and small dams with shallow seepage problems, such as minor sloughing, pin boils, and other shallow instability problems caused by seepage. If stockpiles of suitable porous aggregates and suitable fabric filters are kept on jobsites, and shallow seepage conditions should develop under high water stages, protective blankets of this kind can be quickly placed on troublesome areas. The fabric should have sufficient permeability to allow free flow of seepage into the aggregate layer which should be coarse gravel, crushed stone, railroad ballast, or comparable good-quality, highly permeable material. If additional weight is needed, any stable granular material can be placed over the drainage layer, provided a filter layer is placed where necessary to keep fine soils out of the coarse layer. This is not a recommended treatment for deep-seated instability problems, unless it is heavily ballasted.

Before the development of the synthetic fabrics, drains of the kinds shown in Fig. 2 would have been constructed as graded filters (1), with a fine aggregate filter layer being placed against the soil surfaces on which the drains were to be constructed, and one or more coarser layers to provide for water removal and weight (2), (3). Many water-impounding structures have been provided with drains constructed of durable, natural mineral grains; however it can be difficult to obtain filter aggregates fine enough to provide filter protection, yet permeable enough to freely remove all of the incoming water without the build-up of excessive head. Also, if the filter aggregates are placed on wet, soft ground, as for the levee in Fig. 2(b), the filter material tends to mingle with the wet soil, and its permeability is greatly reduced. Under such conditions, if a selected filter fabric is carefully rolled out over the surface needing protection, it serves as a separator, holding the soil in place, and preventing it from entering into the open-graded drainage layer and reducing its permeability.

The use of filter fabrics had a great deal of impetus in regions in which problems developed with sand and gravel filters that did not meet specification needs, and in cases where a lack of space made it difficult to place graded filters. In places where strong currents and heavy wave action would physically remove filter aggregates under large rock used for riprap and breakwaters, the synthetic fabrics have been particularly helpful. Barrett (4), Dunham and Barrett (5), and other workers describe early shore-erosion protection structures such as stone seawalls and jetties that used the fabrics to hold fine soils in place and thus prevent undermining of these works. Seemel (6) presents a good summary of the development and use of the fabrics. Numerous papers in the First International Conference on the Use of Fabrics in Geotechnics describe usages of fabrics in projects around the world.

In both of the examples given in Fig. 2, the drains are in exterior parts of the structures, and are accessible for removal and replacement if problems should develop over the life of a project.

If the belief that drains are not needed in many small structures such as levees and small dams can be overcome, drainage systems--often with the incorporation of fabrics-can be of great benefit in upgrading the seepage safety of countless structures around the world. An outstanding example of the use of fabrics to help upgrade seepage safety of existing structures is the dike for the Florida Power & Light Company's cooling water reservoir in Florida, where 1.5 million square yards (1,250,000 $m^2$) of nonwoven fabric was used in constructing a drain of the general type shown in Fig. 2(a) (7).

(b) Pavements. The modern belief of most pavement designers that internal drainage is not needed is a psychological hang-up of gigantic proportions, and one that defies explanation, as it cannot be justified on any engineering or economic basis. Because of it pavements deteriorate 3 to 4 times faster (annually) than if they were well drained.

Historically, road builders have believed in good drainage; however hardly any modern road designers do. As a consequence, nearly all of the important pavements that have been built in the past several decades have failure mechanisms built right into them. During the periods of time that structural sections are filled with water, the rates of damage (per traffic impact) can be hundreds to thousands of times greater than when there is little or no free water to be acted upon by traffic and climate. For centuries road builders have known that coping with the water that gets into pavements and the soils under them is the biggest obstacle to having long-lasting, trouble-free pavements. Even the Ancient Romans built their famous road system above the surrounding terrain to help eliminate water damage. In 1820 John L. McAdam (8) said that ". . . if water pass through a road and fill up the native soil, the road whatever may be its thickness loses support and goes to pieces." And, "The erroneous opinion . . . that (by) placing a large quantity of stone under the roads, a remedy will be found for the sinking into wet clay or other soft soils . . . (so) that a road may be made sufficiently strong artificially to carry heavy carriages though the subsoil be in a wet state . . . has produced most of the defects of the roads of Great Britain." His complaint is valid today.

Shortly after enactment of the U.S. Federal Aid Act of 1916, pavements were designed on the basis of the Soil Classification (A-1, A-2, etc.) and a designers experience and judgment. With the advent of modern Soil Mechanics methods, pavement designs have been based almost entirely on strength factors obtained by making tests on saturated samples of base and subgrade materials. Designers have tended to believe that these methods guarantee that any and all problems with water are automatically eliminated. Build pavements sufficiently "stout" and there is no need for drainage, is the idea. Since the loads applied in the tests are generally "static" and traffic impacts are "dynamic" one might expect shortcomings in the designs using these methods. Much of the damage to pavements is caused by the pore pressures and actions in the water impacted by heavy vehicles; other severe damage is caused by climatic actions on the trapped water, such as freezing, "D"-cracking, blow-up, shrinkage cracking, premature oxidation, and the break-out of chunks of pavement to create the well-known "pot holes". Most of these actions do not occur at all in well drained pavements.

In the period from about 1950 to 1962 several highly instrumented and documented "road tests" were made to determine what makes pavements break up and what can be done about it. The primary (but almost totally ignored) finding in these tests was that during periods when free water was trapped in the test pavements, each traffic impact produced up to hundreds and thousands of times more damage than when there was no free water in their sections. In the WASHO road test (9), damage rates were up to 70,000 times greater (per impact) with free water than with no free water present; in the AASHO road test (10), the wet damage rates were 10 to 40 times greater than the dry; in the Univ. of Ill. Circular Test Track experiments (11) they were 100 to 200 times greater with free water than without.

Those planning the Road Tests were thinking only in terms of finding the strongest combinations of pavement and base materials to resist damage, not at all in terms of eliminating the free water with good drainage. As a consequence, not a single one of the hundreds of combinations tested was well drained! And so the prevailing practice of pavement designers continued to be to design pavements "stout", but not even think of drainage as a viable design concept.

Confidence in the "un-drainage" philosophy has been so high that many designers look with disfavor on anyone who dares to question this approach, and deeply resent any suggestion that drainage is a better concept. As part of the work carried out to develop the FHWA's "Guidelines for the Design of Subsurface Drainage Systems for Pavement Structural Sections," (12) field interviews were conducted with State Highway engineers throughout the U.S. in the 1971-72 period. Comments made by persons interviewed probably represent a good cross-section of the opinions of pavement designers everywhere. One of those interviewed said, "I have nothing but contempt for anyone who thinks pavements can be drained." In a major Western state, a top pavement designer said, "But, of course, it is neither necessary, practical, nor economical to drain pavements." In all of these states, pavements were breaking up prematurely from traffic and undrained water.

An engineer in one state interviewed said, "Anyone who thinks pavements can be drained is a fool." He said they had tried drainage and it just doesn't work. They had meticulously compared hundreds of miles of "drained" pavements with many miles of similar, but "undrained" pavements and couldn't detect any difference in performance. On inquiry, we found that a "drained" pavement was a stretch of road (on their standard low-permeability sand base) with a narrow cross drain every 300 to 400 feet (91 to 122 m) with a drain pipe in a trench backfilled with concrete sand. Such a drain could not have had much influence in draining water out of more than a strip 2 or 3 feet (0.6 to 0.9 m) wide above each one of the drains; therefore there should have been no noticeable difference between "drained" and "undrained" pavements. Yet, these engineers were so firm in their convictions they were almost willing to come to blows with anyone disagreeing with them. What a hang-up they had!

In 1977 the U.S. Department of Transportation (DOT) released a report of a major study of the condition of our Nation's highways (13). That report said that out of a $450 billion total outlay expected for American roads between 1976 and 1990, $329 billion will be needed just to keep our roads in their 1975 condition. Applying information I have gathered in investigations of deteriorating highway and airfield pavements across the United States (12), (14), I estimate that the modern belief that pavements don't need to be drained can be blamed for about 2/3 of the repair and replacement costs facing our nation, which represents an unnecessary and avoidable loss of at least $15 billion a year (15) to American taxpayers, and on a world-wide basis, at least a trillion dollars over a 40 year period (16). Even though it is not possible to pin-point the exact losses caused by this hang-up, it must be evident that they represent a severe drain on already overburdened taxpayers.

Taxpayers, public officials, and members of the public media have become alarmed at the rapid deterioration of our "Magnificant Pavement System" that was supposed to represent the best thinking and modern technology, and consumed vast sums of materials, energy, and money. With over 116 million potholes jarring American drivers, and their cars and trucks, an emergency pothole filling bill by Congress was decried as "A Poor Choice of Patchwork" by Engineering News-Record (17). A U.S. News & World Report feature (18) says that Congress' increase in the allowable loads on federally-aided roads in 1974 from 73,280-lb. to 80,000-lb. was an "unwise" act by our legislators. It says that American roads--the most expensive public works undertaking of all time--are being battered to pieces. Numerous other national publications have expressed concern over the deteriorating pavements. Though the increase in loads voted by Congress has been a large factor in the accelerated damage to our undrained system, it would have had a great deal less impact had these pavements been well drained.

Many things contribute to pavement failure; however water is by far the greatest contributing factor (other than traffic impacts). The antiquated "un-drainage" philosophy is a hang-up that must be overcome, before significant improvement in the pavement deterioration dilema can be expected.

SECOND HANG-UP:

The belief that blends of sand and gravel containing a few percent of fines are suitable for all drainage needs.

This ill-conceived belief is a hang-up of major proportions, and one that is an obstacle to the use of the fabrics in drains as it implies they are seldom needed.

Human actions are often a matter of simply following a "popular" practice, no matter how rational or how irrational it may be. Popular practices, like a pendulum, often swing from one extreme to another. Beliefs about what kinds of aggregates are good drainage materials have gone through pendulum-like cycles over the years. After about 1750 there was a period when designers of roads and other engineering works believed that the coarse rock and boulders employed in "French Drains" were ideal materials. Under favorable conditions these drains often served their purpose, at least in part. But, when the large head-size boulders and rocks were placed in trench drains in erodible silts, fine sands, and the like, the drains often became clogged the first time the adjacent soils became saturated. So this kind of material fell in bad repute because of the piping and clogging it produced.

As engineers became concerned over the need to prevent piping, there developed a tendency to use blends of sand and gravel in drains without determining if the blends were permeable enough to remove the water without excessive build-up of head in the drains. They began to believe that concrete sand and other materials containing less than about 5% of fines (silt and clay) were satisfactory for virtually every drainage need. This widespread belief (hang-up) has led to the design and construction of many dams, levees, roads, and other Civil Engineering works that are poorly drained. It persists in spite of world-wide experience proving that this belief belongs in the fairy-tale world.

If errors in thinking about drainage requirements are to be eliminated, we must not forget that every seepage and drainage situation follows specific laws of Nature, and depends on physical factors such as coefficient of permeability, hydraulic gradient, and area of cross-section in which water is flowing. Seepage and drainage are quantitative problems, not qualitative. Each problem has its own specific solution. To ensure that drains will be able to remove the water reaching them, the inflows must be estimated with seepage principles and the required permeabilities and dimensions of drainage layers must also be calculated using the same fundamentals (3), (19), and (20).

Even the simplest "thumb-nail" calculation that uses reasonable values for permeabilities, gradients, and dimensions will almost always show the need for a layer of highly permeable, open-graded aggregate in drains for Civil Engineering works. And this dictates the use of filters--either special aggregates meeting accepted filter criteria, or suitable textiles--to prevent piping and clogging actions, together with a layer or zone of highly permeable aggregate that removes the water.

The filter requirements of drains have been described in many books and publications (21), (22), (23), etc., and the desired properties of fabric filters are discussed in recent publications (24), (25), etc.. Although the discharge needs of drains have had hardly any attention at all until recently, the principles that can be used in making these determinations (largely Darcy's law and flow

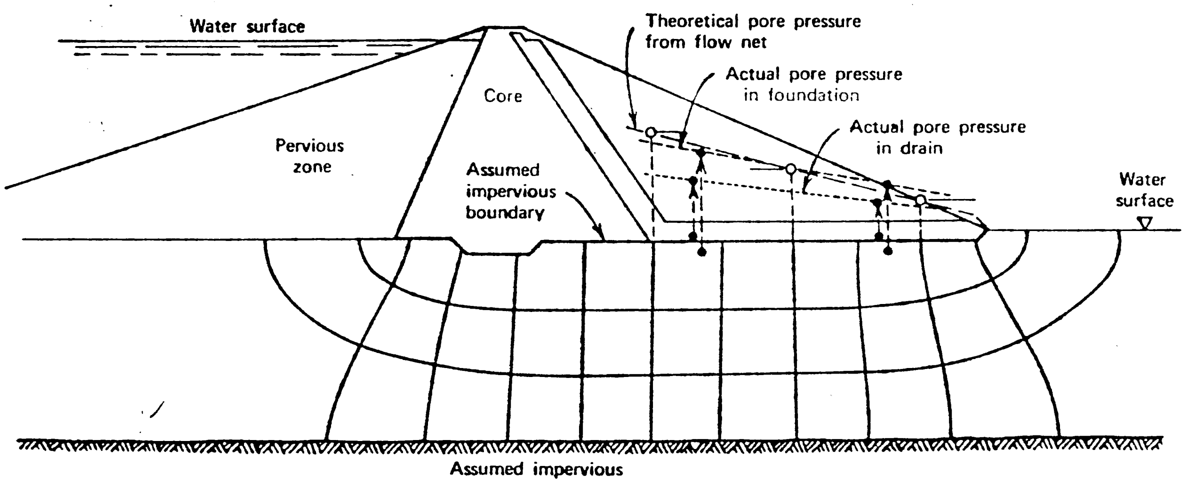



Fig. 3 Uplift pressures that built up under an earth dam with an expensive but ineffective drain were not measurably influenced by the drain (From Embankment-Dam Engineering, Casagrande Volume; 2nd Chapter, by H. R. Cedergren, "Seepage Control in Earth Dams." Copyright © 1973, John Wiley & Sons, Inc. New York. Reprinted by permission of John Wiley & Sons.

nets) have been described in many publications over the past 40 years or so (26), (27), etc.

The dam illustrated in Fig. 3 epitomizes the extremes to which the sand and gravel blend idea has been used in drains. Here, a 60 m high earth dam was built in 1965 in a Western state of the U.S. Designed and built by a major dam design firm, it has a very costly and complex drain system that provided no discernible benefits in controlling uplift pressures under the dam. All drain zones were constructed in three layers, with outer "fine filters" on both sides of an inner "coarse filter". All zones were allowed to contain up to 5% of fines, on the presumption that this amount of fines is satisfactory in drains. After water had stood in the reservoir for several months near the position shown, pore pressures in the drain and in the foundation built up to the levels shown. In order to better understand the problem I constructed the flow net shown, on the assumption that the drain was completely impervious (accepting no foundation seepage). Piezometric heads obtained from the flow net are also shown in Fig. 3. It can be seen that they agree almost exactly with those measured, showing that the drain in fact has no effect on uplift pressures. This dam is a remarkable illustration of the invalidity of the premise that blended aggregates containing not more than about 5% of fines are suitable drainage aggregates.

A great many of the dams I have been asked to investigate because of seepage problems, have had problems caused because drains were either not used, or those that were contained too many fines and did not have the levels of permeability needed to remove the water without excessive head build-up. Only when the discharge needs of drains are properly estimated, and the drains are designed to carry these amounts of water can such deficiencies be eliminated.

Not being willing to analyze drains as conveyors of water, while being willing to follow a misguided concept is a major psychological hang-up that needs to be combated by extensive educational and promotional programs.

THIRD HANG-UP:

The unwillingness to try a new material or idea that does not have a long track record.

Progress in all areas of technology (and medecine) has been hampered by this kind of hang-up. Ideas that happen to be popular remain in vogue for years while eminently superior ideas remain untried, largely because of psychological hang-ups and a "fear of the unknown."

Specialists in all fields have a reluctance to try new ideas that have not been accepted by their peers. A person who follows a practice used by his predecessors or fellow workers (no matter how bad) is generally not blamed if something goes wrong. But let him try something new or innovative (no matter how fundamentally superior it may be) and if problems develop he is usually called "reckless", "irresponsible", or at the minimum "careless". There is often a tendency to be extremely critical about something new and to look "through rose-colored glasses" at the conventional, no matter how poor its record.

This kind of behavior has gone on throughout recorded history. When Galileo furnished evidence that proved the earth revolves around the sun and is not the center of the universe as was believed in his time, he was forced to spend the last years of his life under house arrest after being tried by the Inquisition in Rome for suggesting such a radical idea. Engineers, doctors, and other specialists have stubbornly ignored new ideas that later proved vastly superior to ideas that had been popular at a given time. The potential benefits of open-graded aggregates and synthetic fabrics in drains for engineering works are not being fully realized because of mental or psychological hang-ups such as are discussed in this paper.

Ideas eminently superior to prevailing ideas and practices have been rejected, even ridiculed. When Dr. Simmeliveis in Vienna suggested in 1880 that simple sanitation (washing hands, etc.) could reduce Streptococal infections he was scoffed at and not allowed to operate. Not until 1920 were his cleanliness ideas accepted by the medical profession. In about 1780, Dr. Benjamin Rush of Philadelphia said Cholera and Typhoid were spread by contaminated well water and bottled water. He was ridiculed and his idea lay unaccepted for more than 150 years! Even in the 1920's many children were dying from infantile diarrhea because of inaction and refusal to accept a concept that was very sound. Examples of this kind of human behavior can be found in all major fields.

Why do experts in all fields resist new ideas? Largely because it is "safe" to stay with an accepted procedure--no matter how poor its record. "Don't rock the boat" is an expression that aptly expresses the general attitude about making changes or trying something new. It is very hard to overcome.

One of the problems that needs to be overcome with the synthetic fabrics is the fact that people generally have come to look upon the synthetic products as being rather fragile and of limited life expectancy. Many of the synthetic products such as water hoses and black plastic sheeting become brittle and badly deteriorated in just a few years. Even though much of the deterioration is caused by exposure to sun light, and the fabrics in engineering will be protected from such exposure, there is a tendency to look upon the fabrics skeptically in relation to long-time performance.

Engineers in decision-making administrative positions sometimes don't keep themselves informed about new products or ideas, and reject an idea simply because it is new to them. In 1974 I reviewed a near-failure of a dam in California, caused by an ineffective internal drainage system containing aggregates with 6% of fines. Seepage had caused the near collapse of this dam when its reservoir was quickly filled. I recommended a new toe drain of the general design shown in Fig. 2(a), and such a drain was designed. Just before the opening of bids, an engineer with an agency having control over part of the funds said he would not permit any plastic materials to be used. A redesigned drain with vertical walls was subsequently built with no fabrics used. Severe caving problems had to be fought and the redesigned drain cost substantially more than my original design with fabrics. Unfortunately, this kind of reactionary attitude is altogether too common. It is a handicap to progress.

SUMMARY COMMENTS:

Only part of the potential benefits of good drainage systems and good drainage products is being realized because of some unrealistic and unfounded beliefs (hang-ups). Examples of three major areas where psychological hang-ups are impeding progress as discussed in this paper are:

(1) The belief that drainage itself is unnecessary, impractical, or too costly,

(2) The belief that aggregate blends of sand and gravel materials that provide good filter protection will automatically provide good drainage, and

(3) A general reluctance to try any new idea or product before it has a long experience record.

Collectively these misguided concepts are restricting progress in drainage and the use of new products in drainage systems. Major educational and promotional programs are needed to overcome these hang-ups.

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