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STABILITY OF THE LOWER FRASER VALLEY DYKES

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Introduction

In order to evaluate the stability of the existing Lower Fraser Valley dykes under design flood conditions, a subsurface investigation of these structures was carried out between September 1962 and March 1963. Seventy-one vertical, approximately 40-foot test holes, from which disturbed and undisturbed samples were obtained, were drilled into the dykes and dyke foundation soils. The results of various laboratory tests carried out on the samples are included in drill logs prepared by the Federal Government Department of Northern Affairs and National Resources. Nine triaxial and four laboratory vane shear strength tests were carried out on selected samples by Soil Mechanics Services Ltd. of Vancouver. Maps showing the locations of drill holes and dyke cross-sections through drill hole locations were prepared by the Provincial Government Water Resources Service. All of the above data have been submitted separately to the Fraser River Board and do not accompany this report.

The following report discusses the stability of the dykes against failure by shear under design flood conditions and recommends a method for stabilizing sections of the dykes liable to this type of failure. The possibility of dyke failure by local piping of foundation soils is discussed, and recommendations for control of this danger put forth. The stability of sections of dyke composed predominantly of peat under flood conditions is briefly discussed.

Dyke and Dyke Foundation Soils

The predominantly fine-grained Fraser Valley floodplain sediments were the soils used in the reconstruction of most of the dykes following the 1948 flood. The most common types of soils encountered in the drill holes penetrating the dykes were sandy silts and silty sands. In the dykes, these soils range from very loose to dense and from essentially non-plastic and cohesionless to medium plastic where considerable organic material occurs. Stratification was observed in some undisturbed samples taken from the dykes indicating the presence of relatively large lumps of undisturbed sediment in these structures. Silt lumps were found separated by sand in some drill holes and sand occurs as distorted lenses in silt locally. Rarely the silts are clayey for a few feet and lumps of clay occur in the coarser sediments locally. The above suggests that little or no compaction of layers of these sediments was done during dyke construction. Locally these soils contain various amounts of partially decayed organic matter ranging from very fine, often vertical fibrous roots to relatively large pieces of wood and bark. In some drill holes, relatively thick layers of organic silt and peat were penetrated near the base of the dykes, suggesting that at least along some sections of dyke little or no stripping of the sod was done prior to construction. Some sections of the dykes, especially in the downstream part of the valley, appear to be composed predominantly of peat with little or no sediment admixed with or covering this material. Generally poorly graded, silty sandy gravel was encountered in a few drill holes in the dykes, and in places sections of dyke are composed entirely of this material. Occasionally the gravel is clean.

As stated above, the dyke foundation soils are generally similar to the dyke soils. However in some areas they tend to be considerably coarser grained at depth. The fine-grained foundation soils are generally well stratified and loose. They are saturated due to the water table being generally very near the ground surface in much of the Lower Fraser Valley. Organic material, although more prevalent near the ground surface, was encountered to a depth of 40 feet in some areas.

Numerous unconfined compression tests were carried out on the above soils and the results of these tests are shown on the drill logs. Their strengths, determined from the tests, range from less than 200 p.s.f. to over 1,000 p.s.f. with values between 300 and 600 p.s.f. being the most common. Some of the very soft, apparently weak, organic samples gave results as high as 900 p.s.f. suggesting that fibrous roots may add to the strength of these soils locally. The nine triaxial tests on samples from six dyke districts were made to determine the friction and cohesion components of shear strength. The procedures used and the results obtained are included in the Soil Mechanics Services Ltd. report. The samples tested were well-graded, non-plastic, fine silt and sandy silt containing sand lenses locally, the most common soil types encountered in the dykes. Organic material was observed in two of the samples. The results of these tests indicate that these soils possess effective angles of internal friction ranging from 34 degrees to 39 degrees and averaging 35 degrees, and have little or no cohesion. Soil Mechanics Services engineers suggest that the relatively high friction angle values could be caused by the generally well-graded nature of the soils, stratification, fine fibrous roots, or combinations of these factors. Because of the presence of considerable organic material in sections of the dykes, particularly in layers between the dykes and the foundation soils, four laboratory vane shear tests were carried out on peat and silty peat samples. The "maximum" or undisturbed values of shear strengths obtained ranged from 345 to 496 p.s.f. and the values obtained from samples remolded in the laboratory ranged from 130 to 245 p.s.f.

Stability Analyses of the Land Side Slopes

The existing dykes are relatively stable when dry but when largely saturated during an extended period of flooding could fail locally by shear of the soils under the land side slopes as discussed below.

Most of the existing dykes appear to be roughly homogeneous in cross-section, and consequently any sections which are sufficiently narrow and composed of sufficiently permeable sediment will transmit water completely through them during the period of design flood assumed.* This through seepage has the effect of increasing the possibility of failure of the

*The hydrograph of water levels for the Fraser River Board design flood regulated indicates an average rise of the river of about 1 foot per day for six days to a maximum elevation of 26 feet at Mission, minor fluctuations for the next thirteen days and then a more gradual recession with occasional minor rises during the following twenty-two days.

land side slope of the dykes. The shape and position of the top flow line and the highest point of emergence of the seepage on the land side slope can be readily determined for any given cross-section and head. An example is shown by the upper dashed flow line in Figure 1. The most unstable section of a cohesionless land side slope is from this highest point of seepage emergence to the toe of the dyke. The stability of this section of slope is largely dependent upon the magnitude and distribution of pore pressures in this area, both of which are determined by the path but not velocity of the seepage flow. The detailed shape of the flow net from which the pore pressure at any point beneath this slope can be determined is difficult to construct in this immediate slope area without extensive knowledge of hydraulic boundary conditions and the composition and other characteristics of the slope. If the slope area is relatively homogeneous most of the flow lines will meet the slope at some acute angle. However, immediately below the emergent point of the top flow line seepage will probably occur very nearly parallel to the slope, particularly in the flatter slopes. Where this is the case, the saturated slope required for equilibrium can be determined for any value of effective friction angle and soil density by the simple relationship:

$$\tan \beta = \frac{\gamma'}{\gamma} \tan \phi'$$

where β = slope angle

ϕ' = effective friction angle

γ = total weight of soil

γ' = submerged weight of soil

Figure 2, which is meant only to serve as a very rough guide to the stability of the lower portions of the land side slopes of the dykes, since it strictly applies only to areas where seepage is parallel to the slope, has been prepared by applying the effective friction angle and density ranges of the dyke soils to the above equation. From this figure the saturated lower slopes, where the above seepage condition occurs, required for equilibrium for the given ranges of soil conditions, can readily be found.

An examination of numerous dyke cross-sections indicates that, provided that the friction angles and densities used are representative of most of the dyke soils and that seepage parallel to some areas of the slopes occurs, areas of the existing land side dyke slopes will be unstable under the design flood conditions. The mechanism of deterioriation of saturated slopes has been discussed by Haefeli (Ref. 4). In the case of seepage flowing parallel to a slope, theoretically failure is along planes parallel to the slope. In the case of the dykes, material would be carried away from the toe areas, thus undermining and causing general instability of the slopes.

Bishop and Morgenstern (Ref. 1) have proposed a graphical method for determining the safety factors of simple homogeneous embankments in terms of effective stress by the use of "stability coefficients" and average values of pore pressure ratio. The reader is referred to Reference 1 for a detailed description of the analytical methods used. The average pore pressure ratio was determined for several dyke sections assuming steady seepage and by the use of the graphs for determining the "stability coefficients" for the appropriate values of effective friction angles and slopes, the safety factors of the existing land side slopes of these sections were determined. In cases where the slope was steeper than 4:1 the safety factors determined by this method are near or below unity.

In order to further evaluate the possibility of failure of the land side slopes of the dykes, numerous stability analyses were carried out of cross-sections tested by drilling. Analyses were made by the conventional method of slices. Neutral pressures and consequent effective normal stresses along failure planes were determined from the position of the calculated top flow lines. For simplification, the slopes between the highest point of seepage emergence and the dyke toes were considered to be flow lines although, as explained above, this is probably only the case over limited areas, particularly in steeper slopes. In order to analyse for the worst case, it was assumed that steady seepage existed. Densities used were those obtained from the laboratory analyses, and soil strengths were those obtained from the triaxial tests. It was assumed that the soils along the failure planes were essentially cohesionless. In this regard it is considered reasonable to assume that no effective cohesion will exist to contribute to shear strength along the section of any assumed failure arc below the top flow line, at least in the more sandy sections of dyke, although in the possibly partly saturated zone above the top flow line a small apparent cohesion may exist which would add very slightly to the total strength along the assumed arc. The addition of this small apparent cohesion would have little effect on the results of the stability analyses.

The results of analyses of shallow potential arcs of failure, extending from the toes of the dykes to points some distance above the upper points of seepage emergence, indicate safety factors below unity in all cases analysed. Analyses of deeper arcs, extending from the toes of the dykes to the crests, indicated safety factors near or below unity in many cases even when removal of material from the saturated toe of the dyke was not allowed for.

Because of the presence locally of relatively thick organic layers between the dykes and the foundation soils, stability analyses based on failure along such layers were carried out for a few sections tested by drilling where this type of failure is considered possible. Such a failure would probably begin as a circular arc extending down to the organic layer and continue as a shear along this layer. Flood water levels and densities used were similar to those used in the foregoing analyses. An average shear strength across the organic layer of 130 p.s.f. was assumed in order to analyse for the worst case. The safety factors determined in these analyses were generally below unity, indicating that failure of the dykes by this mechanism can also occur under the design flood conditions. However, the results of these analyses are not considered to be particularly reliable, as a wide range of shear strengths of organic material was obtained from the laboratory vane tests. That is, the average shear strength across an organic layer could differ considerably from that assumed in the analyses, and the actual value is impossible to predict.

The results of the foregoing various approaches to the problem of predicting the stability of the land side slopes of the dykes under design flood conditions suggest that sections of the dykes will fail if these flood conditions should arise. Consequently, stabilizing measures should be taken before a flood occurs. Evidence is on hand (Ref. 5) to show that shear under land side slopes along circular arcs did in fact occur during the 1948 flood in dykes of similar cross-section and height as those existing today.

Piping of Foundation Soils

In some of the dyke areas, "boils" occur on the flat ground on the land side of the dykes. Their distance from the dyke toes varies from a few feet to several hundreds of feet.

They are caused by relatively strong, upwardly directed seepage pressures which cause the fine, light silts and sands to pass locally into a semi-liquid state. During times of high water, these seepage pressures increase and can cause soil to be actually carried away and produce continuous holes or "pipes" from the inside toe of the dyke to the river. Animal burrows can aid this mechanism. These "pipes" can undermine the dyke, causing collapse and overtopping. Such piping of foundation soils was apparently responsible for several of the dyke failures which occurred during the 1948 flood.

Incompetency of Peat Dykes

Some low dyke sections in the downstream area of the Lower Fraser Valley are apparently composed predominantly of spongy brown peat. Such material is generally very weak and highly permeable and is essentially useless for embankment construction of any kind. The density of peat samples taken from the dykes is near that of water, indicating that during a time of flooding dyke sections composed mainly or wholly of peat could actually float and be carried away by the current.

Liquefaction of Loose Silts and Sands

The possibility of failure of sections of the dykes by liquefaction of loose cohesionless silts and sands has been investigated by the Department of Northern Affairs and National Resources. A report on this subject was submitted to the writer and to the Fraser River Board on June 7, 1963, by E. M. Clark who summarizes the results of the investigation by stating--"The results obtained do not prove that liquefaction will not occur. However, on the basis of this rather limited examination, it would appear that failure by liquefaction would not be anticipated." The writer has nothing to add to this concise report, and has no reason to add to or subtract from its contents.

Dyke Stabilization

In order to ensure that the dykes do not fail by any of the above discussed mechanisms during a time of flood, the stabilizing measures outlined below should be carried out.

(a) Sections of dyke presently below grade should be increased in elevation to allow 2 feet of freeboard above the maximum design flood level. The width of dyke crests should not be less than 10 feet and should be surfaced with sufficient gravel to allow easy movement of vehicles along them during even very wet weather.

(b) The land side slopes of the dykes should be stabilized by the addition of suitable material. If the local silts and sands are used, relatively flat slopes will be required to yield acceptable factors of safety. In this regard, it may be pointed out that in order to ensure safety against failure during times of flood, relatively homogeneous compacted silt dykes along the Mississippi River have been constructed with land side slopes of 6.5:1 and river side slopes of 3.5:1. If constructed of sand they are given land side slopes of 8:1 and river side slopes of 5:1 (Ref. 2). In order to stabilize the Fraser Valley dykes, free draining gravel should be added to any land side slopes presently steeper than 4:1. Stability analyses of certain dyke sections tested by drilling were carried out assuming that 2.5:1 gravel slopes were constructed on the existing land side slopes and that the average underlying silt and sand slopes were not allowed to be flatter than 1.75:1. Where the existing silt and sand slope is flatter than 1.75:1, the dyke toe should be excavated and backfilled with gravel as shown in Figure 1. Even in the few areas where the existing land side slopes are flatter than 4:1, some rock riprap or very coarse gravel should be placed against the land side slope along the toe areas to ensure stability. The effect of this type of dyke stabilization can be readily seen from Figure 1. The top flow line is considerably lowered, thus decreasing the neutral pressures and increasing the effective normal stresses at any point on a potential plane of failure under the slope. The results of the stability analyses indicate that safety factors of from 1.4 to 1.6 against failure of the land side slopes will result from the addition of gravel as outlined above. As a relatively small amount of seepage will pass through the dykes because of the short duration of a flood, graded filters between the existing dyke sediments and the gravels are not warranted. Consequently some plugging of the gravels by these fine-grained sediments will occur but this will not be serious. In the few areas where the dykes are constructed of generally dense, silty sandy gravel, considerable improvement to stability would be effected by the addition of very coarse gravel or rock riprap to the land side slopes.

The river side slopes of some sections of the dykes are relatively steep and when the water rises upon them during a flood, local failure could occur. Also, a rapid drop in the river level following a prolonged period of flooding could, especially in the finer-grained sections, create sufficiently high residual pore pressures under the river side slopes, to cause local failure. Consequently it is recommended that any river side slopes which are steeper than 3:1 should be reduced to this slope. This would generally be done by filling except where the present dykes directly border a river, in which case this slope would be trimmed and a relatively large quantity of material added to the land side slope to maintain the design outlined previously. In sections where filling of the river side slope to bring it to 3:1 will be done, all sod should first be removed from the existing slope. Finally, the completed river side slopes should be seeded with a hardy grass to help minimize the effects of erosion on these slopes.

(c) The present practice of dumping coarse gravel or rock into "boils" located near the inside toes of the dykes or into the river opposite "boils" is of practically no use in stabilizing this condition. Instead, any "boil" and a considerable area surrounding it should be covered by properly graded filter layers. The filter layers have the effect of increasing the effective weight over the "boil" thus holding the fine-grained light soil down while allowing the upward percolating water to escape harmlessly. If new "boils" develop during a time of flood, they should be immediately treated in this manner before piping under the dyke has a chance to proceed. Consequently, properly graded filter material should be stored at strategic points within the dyked areas for use in emergencies.

(d) Any of the dykes composed entirely or very largely of peat should be reconstructed to an acceptable design.

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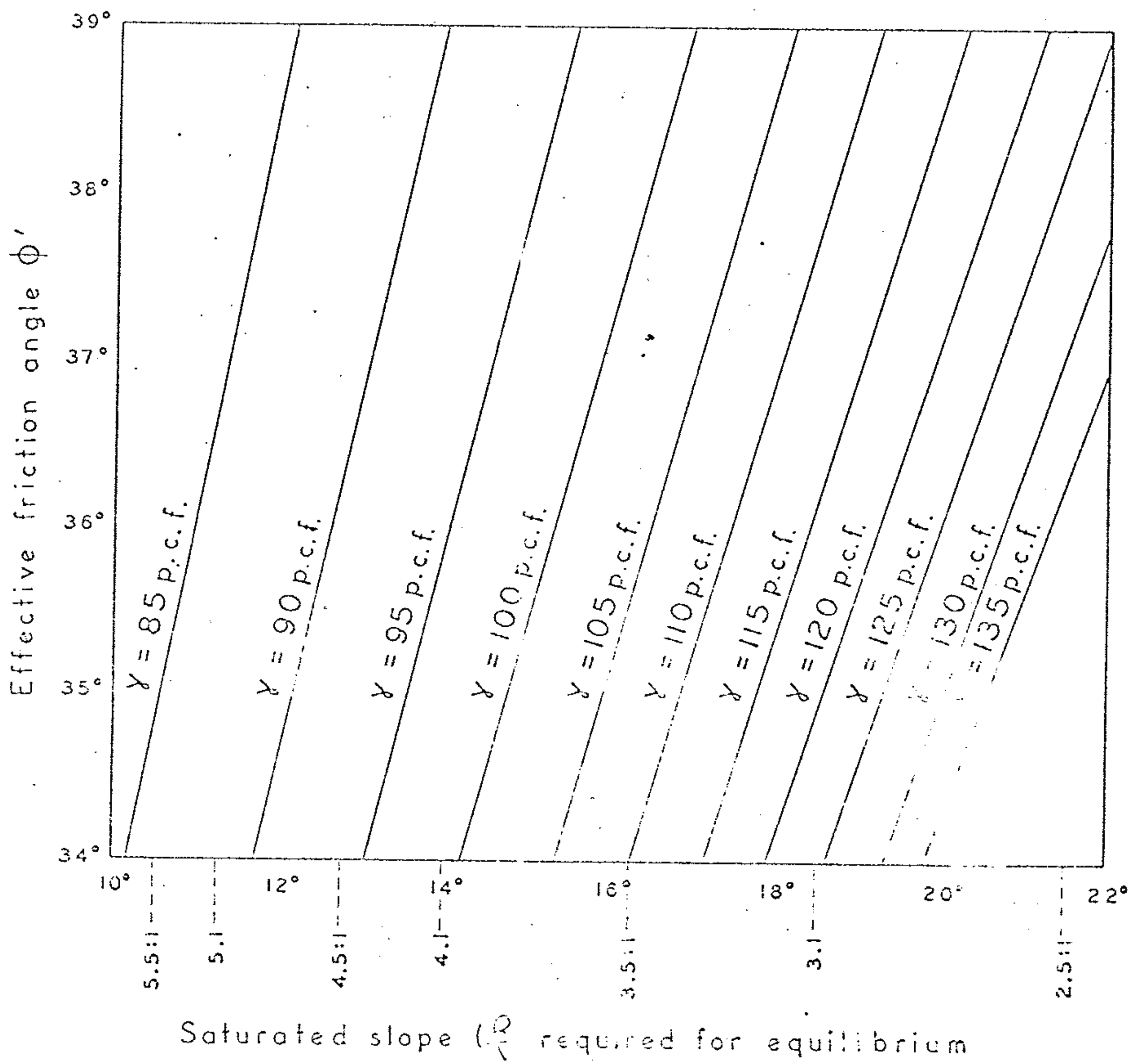
Effective friction angle φ'
39°
38°
37°
36°
35°
34°
γ = 85 p.c.f.
γ = 90 p.c.f.
γ = 95 p.c.f.
γ = 100 p.c.f.
γ = 105 p.c.f.
γ = 110 p.c.f.
γ = 115 p.c.f.
γ = 120 p.c.f.
γ = 125 p.c.f.
130 p.c.f.
= 135 p.c.f.
10°
12°
14°
16°
18°
20°
22°
5.5:1
5:1
4.5:1
4:1
3.5:1
3:1
2.5:1
Saturated slope (β) required for equilibrium

FIG. 2

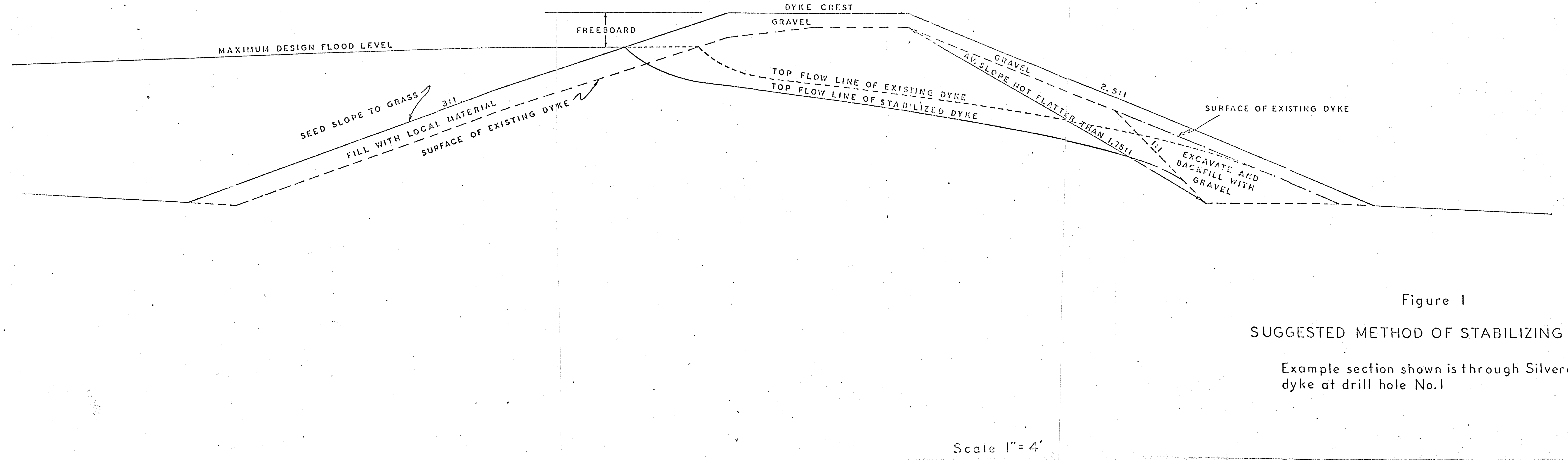

Figure 1

SUGGESTED METHOD OF STABILIZING DYKES

Example section shown is through Silverdale dyke at drill hole No. 1