STREAMBANK STABILITY IN OPEN CHANNEL DRAINAGE IN THE OTTAWA-ST. LAWRENCE LOWLANDS

© by
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SHORT TITLE

Streambank Stability in Open Channel Drainage

Abstract

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Interceptor tile drains were installed along streambanks on four experimental sites throughout the Ottawa - St-Lawrence Lowlands in the fall of 1983. They were subsequently analyzed for their effectiveness to control streambank erosion. Various seed mixtures were applied to the streambank and evaluated for their performance against soil erosion. The mixture containing both a legume and a grass appeared to provide the best protection against streambank erosion.

The watertable was monitored during the spring runoff, when streambanks are at the weakest state against erosion, for both tiled and untiled streambank sections.

It was calculated that basic force stability of the streambank increased by as much as 33% in tiled streambanks.

It was also discovered that seepage forces in certain localized areas along the streambank could be reduced by up to 60% during periods of high watertable.

Résumé

L'installation de drains souterrains parallèles aux cours d'eau à quatres differents tronçons experimentaux s'est effectuée en automne 1983. Ceux-ci furent analysés pour leur efficacité par rapport à la dégradation des cours d'eau.

Des divers mélanges de semences ont été appliqués au talus, puis une évaluation de leur performance contre l'érosion des sols a été entreprise. L'étude de l'ensemencement avec un mélange de plantes légumes et herbes apparaissait comme le meilleur contrôle d'érosion.

L'elévation de la nappe d'eau a été surveillée pendant les crues printanières, au moment que les talus sont particulièrement sensible à l'érosion, dans les sections de cours d'eau avec ou sans drains intercepteurs.

Il a été calculé que le facteur de stabilité peut accroître jusqu'à 33% après l'installation d'un drain intercepteur. Il a également été constaté que la force de la fuite déperdition dans certain endroits sensibles peut être diminuée jusqu'à 60%.

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INTRODUCTION

1.1 Nature and Scope of Problem

Current techniques in the design, construction and maintenance of open channel drains in the Ottawa - St-Lawrence Lowlands appear to be inadequate. Many recently constructed watercourses have been found to require extensive maintenance due to severe channel bank failure and subsequent erosion and sedimentation.

The designer of open channel drains should have more detailed information of the soils, in order to achieve the most cost effective and environmentally acceptable design, both in the short and the long term.

This research will first consider the methodology for field soil investigations in the more unstable soils and secondly, analyze remedial measures in the control of streambank erosion in the problem soil.

1.2 Demonstration Sites

Several demonstration sites were selected through contact with local consultants, government officials and from past experience with unstable soils. These demonstration sites are located in the eastern Ontario region, as shown in Figure 1, and have exhibited some form of streambank instability at some stage of their lifetime.

Visual identification and classification of the soil was carried on at all demonstrations sites. On selected locations, notably experimental sites, further monitoring and soil testing techniques were performed to aid in identifying unstable soils.

Generally, the experimental sites involved the installation of water table tubes to monitor the position of the water table during the spring freshet, detailed soil analyses to determine soil composition, hydraulic conductivities for soil permeability and soil strength measurement to determine the soil's weakest state.

Remedial measures were evaluated based upon their contribution against streambank failure and form part of the analysis within the experimental and observational sites. Interceptor tile drains parallel to ditch banks were installed along the ditch banks of the experimental sites to reduce seepage forces and weight of water in the streambank.

The observational sites provided further insight into some of the causes and actions of some of the pervasive erosion and sedimentation. Seeding of various vegetal mixtures was the only remedial measure applied to the observational site.

TABLE 1

DEMONSTRATION SITES

WATERCOURSE	MAP LOCATION WITH REFERENCE TO FIGURE 1	PREDOMINANT SOIL TYPE
	EXPERIMENTAL SITES	
North Morrow	1	Varved clay
Hammond	2	Fine sand over clay
Lower York	3	Silty loam
Vars	4	Fine sand
	OBSERVATIONAL SITES	
Cumming	5	Clay with sandy phases
Labreche	6	Fine sand
Millette	7	Fine sand

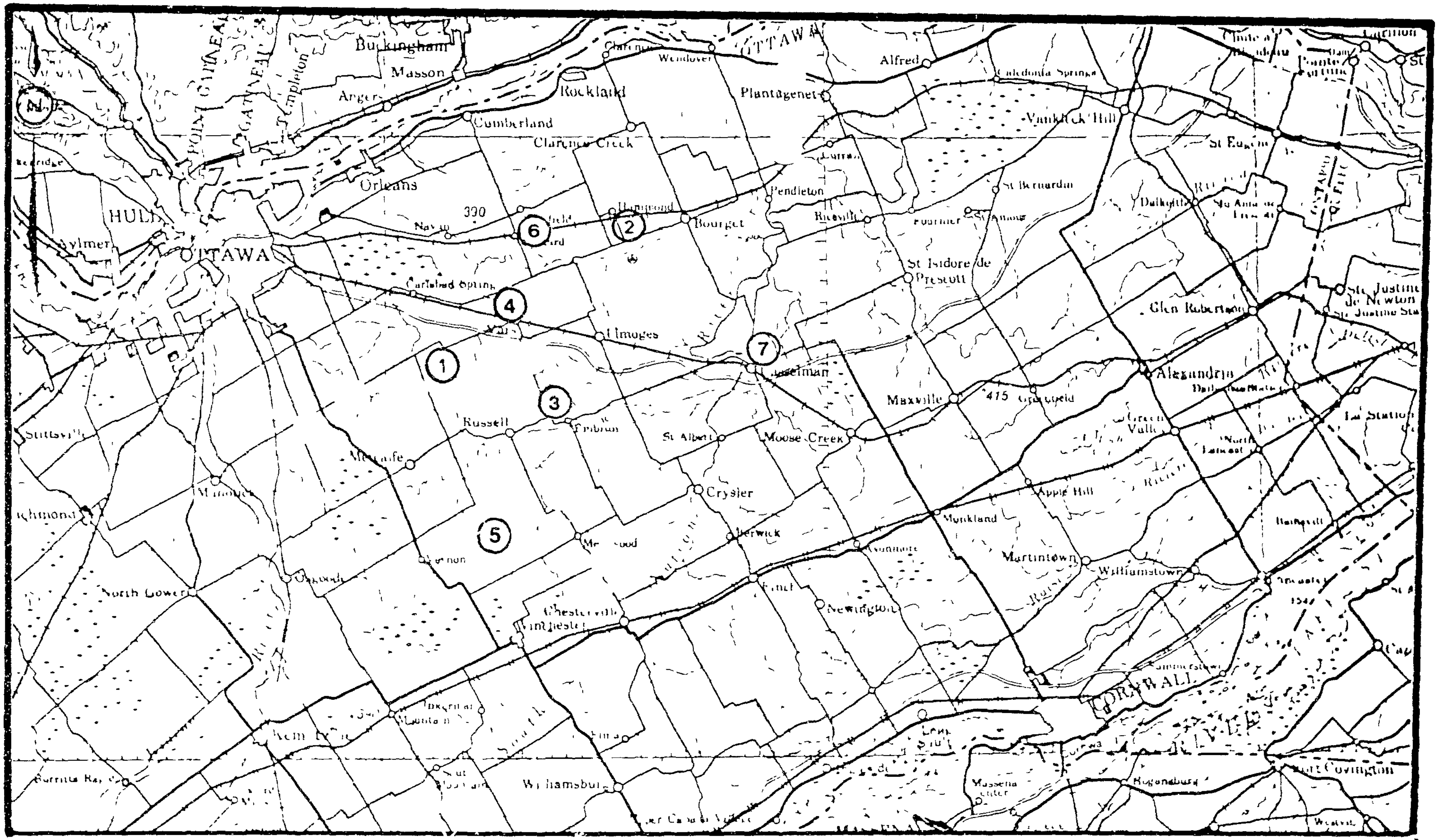


Figure 1 **DEMONSTRATION SITES**

1.3 Objectives

The objectives of this study were to :

i) determine the nature of the problem soil

ii) determine the causes of streambank failure

iii) observe the process of streambank erosion

iv) evaluate the effectiveness of applying remedial measures to the streambank vis-à-vis their contribution to streambank stability

CHAPTER II

REVIEW OF LITERATURE

2. STREAMBANK STABILITY

2.1 Pedology of Study Area

The predominant soils of the region in question are highly variable over relatively short distances. They range from a thin layer (about 1m) of stony glacial till over limestone and shale bedrock, to deep clays deposited in the marine environment of the Champlain inland sea during the retreat of glaciation. From the western part of the region eastward, materials ranging from coarse gravel to clay were deposited in fresh water over the marine clay during the retreat of the Champlain Sea. Many of the near-surface clay deposits are varved. This common occurrence in glacio-lacustrine deposits results in the laminae of alternating fine and coarse materials, each pair representing deposits of a single year. The double band is commonly only a couple of centimetres thick, but some pairs are several times this thickness.

Although surface relief is significant throughout the western part of the region, gradients in the rest of the plain are very low with water tables near the surface. It is in these areas of low gradients and high watertables that open channel outlet drains suffer most from severe erosion, rotational slumping and sedimentation, most of which occur in the period during and immediately following the annual spring snowmelt and runoff. A number of extensive outlet drainage channels have required reconstruction within three years of initial construction as a result of this erosion and sedimentation.

The glacial and recent geological history of the St-Lawrence Lowlands is quite well known. Three soils or terrain situations exist in which most open channel drainage is constructed. These are:

1) Till ground moraine deposits
2) Champlain Sea marine clay plains
3) Sand deposits of both alluvial and marine origin

1) Glacial Till: Glacial till is found in many areas of the St-Lawrence Lowlands, where it is usually exposed in upland areas, and it is sandy, permeable and easily eroded. In topographically lower areas, it is grey, cohesive, clayey soil and is usually covered by a clay or sand blanket.

2) Champlain Sea Clay: This is perhaps the most widespread soil type in the region and is found in lowland planar deposits. Two phases, a marine and an estuarine phase exist, which are geotechnically similar.

3) Sand: Both alluvial and marine sand occur in many places in the study area, often overlying clay till deposits. The sand varies greatly in stratification, type, grain size distribution and thickness.

2.2 Determination of Soil Particle Characterization

2.2.1 Grain-size Analysis

The size, character, and distribution of soil particles have a considerable influence on the permeability of a soil. The grain-size distribution test has been developed to determine the particle size gradation of a given soil, as well as the mass percent of the grains within a given size range of soil. These particle sizes are plotted on a logarithmic scale, revealing points which will show the total mass percent of the particles smaller than given particle diameters.

For the purpose of experimentation, the grain size distribution test, as outlined by Lambe (1951), was followed. Briefly, it involved sieving with a square mesh through which particles with a diameter greater than 0.1mm can fall. For particles less than 0.1mm, the grain size is determined by the use of a hydrometer which operates on the basis of the settling rate of the particles in a liquid. The relation between the diameter and the speed of settling of a sphere is expressed by Stokes' Law. Any further information concerning its application may be obtained in any standard soil physics textbook.

2.2.2 Atterberg Limits

The Atterberg Limits are generally referred to as the limits of consistency of fine grained soils on the basis of moisture content. These limits are the liquid limit, the plastic limit and the shrinkage limit.

The plasticity index, PI, is the difference between the liquid limit and the plastic limit of a soil. The plastic limit is defined as the moisture content of a soil which transforms it from a plastic to a semisolid state, and the liquid limit is the moisture content at which the soil changes from a liquid state to a plastic state.

The experimental procedure for the Atterberg limits was conducted according to Lambe (1951).

2.2.3 Hand Texture Test

As described above, the Grain Analysis is a reasonably accurate method for determining soil texture. However, as it requires extensive laboratory time and equipment, it is not a very practical, rapid field solution.

The hand test is a field method specifically designed to provide a quick on-site determination of soil texture. In addition, soil texture may be quite localized, therefore a visual check at random locations would be more desirable because many more samples can be characterized in a given time.

The analysis as outlined by the Ministry of Ontario, Transportation and Communication (1977) was followed, which involves selecting a small sample of soil and placing it in the palm of the hand. The sample is continually wetted until a ball 1-2cm in diameter is formed. Once a moist sphere has been reached and maintained throughout the test, the soil may be classified according to three basic textures

1) Sandy loam, loamy sand
2) Silt loam, loam
3) Silty clay loam, clay loam.

2.3 Causes of Streambank Failure

Before discussing the types of streambank failures which exist, the causes must be first identified. In general, most failures are caused by groundwater and seepage within the soil itself. These failures can be classified into one of two categories, as outlined by Cedergen (1967):

1. Failures which take place when soil particles migrate to an escape exit and cause piping or erosional failures;

2. Failures which are caused by uncontrolled seepage patterns that lead to saturation, internal flooding, excessive soil uplift, or excessive seepage forces.

In both cases, the causes of instability result in a decrease in soil strength or an increase in soil shear stress. Below is a modified table developed by Sowers and Sowers (1970) which lists the major contributions to increased stresses versus decreased strength in a streambank.

TABLE 2

Causes of Increased Stress	Causes of Decreased Strength
1. External loads such as water or snow.	1. Swelling of clays by absorption of water.
2. Increased unit weight by increased water content.	2. Pore water pressure (neutral stress).
3. Removal of part of mass by excavation.	3. Hairline cracking developed from alternate swelling and shrinking or from tension
4. Undermining caused by seepage erosions.	4. Strains, and progressive failure in sensitive soils.
5. Tension cracks.	5. Thawing of frozen soil or frost lenses.
6. Water pressure in cracks.	6. Deterioration of cementing material.
	7. Loss of capillary tension on drying.

2.4 Types of Streambank Failure

Gravity is the major force in producing failure while the shearing strength of the soil is the major force in resisting failure. There are three major forms of slope failure in cohesive soils, as described by Sowers & Sowers (1970) and illustrated in Figure 2. A base failure occurs in cohesive soils such as the soft clays with numerous soft seams. They are initiated by the formation of tension cracks at the top of the slope, and subsequently followed by a shear failure along a slip line, resulting in a slumping of soil mass at the foot of the slope. A toe failure is more common in a steeper cut with a higher degree of internal angle of friction. A special case which seems to represent most of the eastern Ontario failures is the slope or face failure. This form of failure exists when a hard strata limits the extent of the failure surface.

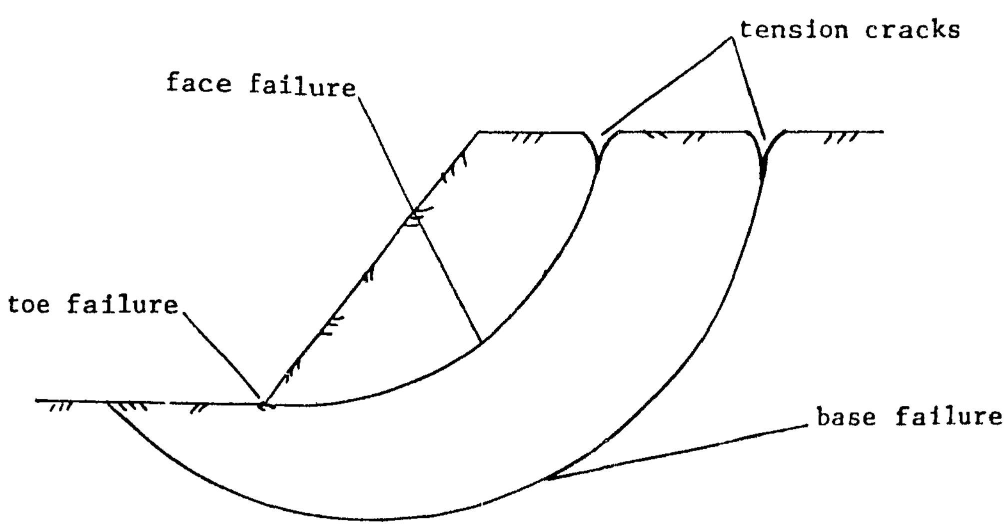


FIGURE 2

Lohnes & Handy (1968) investigated the relationship between the height, slope angle and strength properties of streambanks in loess material. Two main mechanisms of failure were identified: shear or face failure, and base or slab failure. The Culmann method was applied for a strength analyses to predict the maximum bank height for a non-vertical bank slope. Even with modifications of this method with the tension crack, Lohnes & Handy (1968) made clear their approach can only be approximate because the basic assumptions on which it is based are not quite satisfied in a real streambank. The Culmann method does not consider pore pressure and calculates only a total stress factor. For this reason, its application is limited to the sandy or highly permeable soils with a low degree of saturation. Further, soils are elasto-plastic, exhibiting strain prior to failure along a slip line progressively, as opposed to simultaneously at all points for a rigid-plastic material as suggested by the Culmann method.

Streambanks are loaded by three means as described by Bradford & Piest (1977); the weight of soil mass, the weight of water added to the soil mass, either by surface infiltration or rise in water table, and by seepage pressure in the soil mass. As the water content increases beyond some optimum moisture content, representative of that soil type, the shearing resistance of the soil decreases. The in-situ shearing strength is also influenced by freezing and thawing, or wetting and drying cycles.

Vertical tension cracks at the surface of the slope as shown in Figure 2, possibly occurring along natural cleavage planes, reduce the overall slope stability by decreasing the cohesion which can be mobilized along the upper part of the potential failure surface. Water infiltration from the soil surface into the surface crack also increases the pore water pressures acting on the failure surface.

In their analysis, Bradford & Piest (1977) further established a field network of tensiometers and piezometers in relation to a vertical wall and trench. The objective was to study the control of a water table within the soil mass and to observe failure as a function of pore pressure. Observations included a pop-out effect near the toe of the slope, leaving an overhang which eventually slumped into the trench. The application of Bishop's Method of Slices, (Fellenius, 1939) for a circular arc toe failure was inconclusive in calculating a factor of safety since the "pop-out" effect is best attributed to a high lateral seepage force due to an underlying impermeable layer.

A subsequent report by Bradford & Piest (1980) discussed further their observations of gully wall stability. The major failure modes are the deep seated circular arc toe failures which were found at the relatively low angle banks some distance downstream of a headcut. Slab failures were observed downstream from a headcut on steep banks. "Pop-out" failures were generated by steep banks with a high degree of saturation and subsequent washing out of soil at the toe of the slope.

There is evidence that ice formation at river banks and the break-up away of ice in the spring is a cause of bank erosion on medium and large rivers. For streams with drainage areas less than 2000 ha, ice generally is not the major cause of bank erosion. However, the ice effect is beyond the scope of this study.

2.5 Stability Analysis

In analyzing stability in cohesive soils, Bishop's Method of Slices is used to calculate a factor of safety. If a soil is to fail, then the sum of all the forces tending to produce failure, that is the weight of soil and weight of water, are greater than the sum total of the resisting forces, that is the shearing strength of the soil. The ratio of this resisting moment to the driving moment results in a safety factor. If the driving force is greater than the resisting force, then the ratio, or safety factor, would fall below unity, and the streambank would be expected to fail.

As described in Section 2.5.2 below, the Method of Slices takes into consideration the water pressure acting on each slice. However, for the purpose of calculating streambank stability in open channels, under certain circumstances generally referred to as worst case conditions, the Method of Slices may be reduced to a much more simplified calculation, known as the Circle Moment Method.

Instability results only when shear failure has occurred at enough points to define a surface along which the shear slip can take place. Specifically, the Circle Moment Method has its normal force and water pressure vector perpendicular to the slip line and parallel to the radial vector. This results in the elimination of the resisting moment forces of water pressure and friction along the slip line. However, when approaching saturated conditions, that is for worst case conditions, the soil begins to swell. Thus a shearing strength can be represented almost as closely by a constant value as it can by some form of Coulomb's law with strength varying linearly with normal intergranular soil pressure. This produces a negligible internal angle of friction, or $\phi=0^{\circ}$, and the frictional term of the Circle Moment Method becomes negligible.

2.5.1 Coulomb's Empirical Law

Given a strength envelope of allowable shear and normal stress for a given soil under certain conditions, Coulomb's law may be applied. As shown in Figure 3 below, the abscissa of this graph represents the effective normal pressure which is the intergranular pressure on the failure plane. For granular soils, the effective pressure is calculated as the total pressure minus pore water stress. The ordinate plot is a measure of the shear strength.

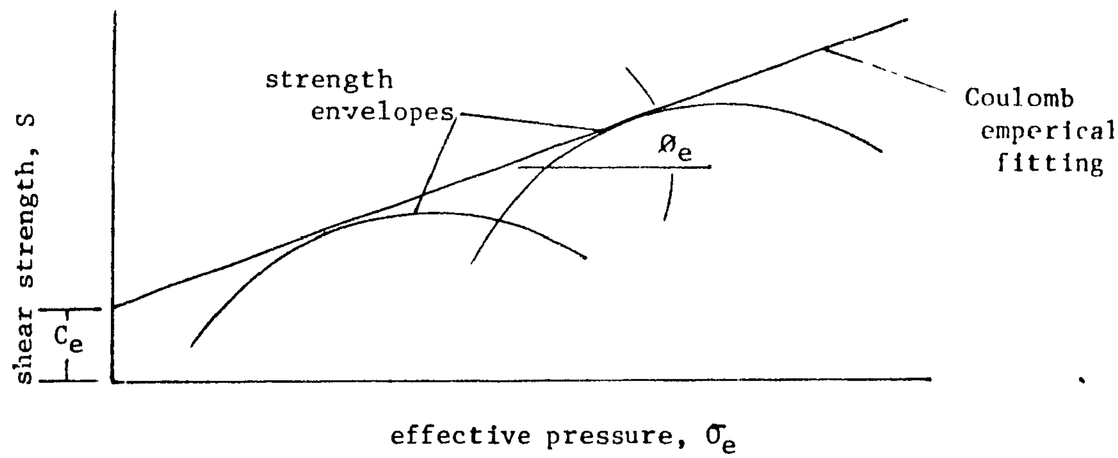


FIGURE 3

The straight line which can be produced from the given envelope may be represented by an empirical equation in the form

$$S = C_e + \sigma_e \tan\phi_e \qquad (1)$$

in which C_e and ϕ_e are the effective cohesion and the effective friction angle respectively. It should be noted that the effective pressure is greatly dependent on the conditions and thus Ce and ϕe are not constant properties for a soil. They may vary quite substantially with such factors as soil density and water content.

The diagram in the following section shows the application of the soil strength theory to the stability of a plane slope.

2.5.2 Modified Bishop's Method of Slices

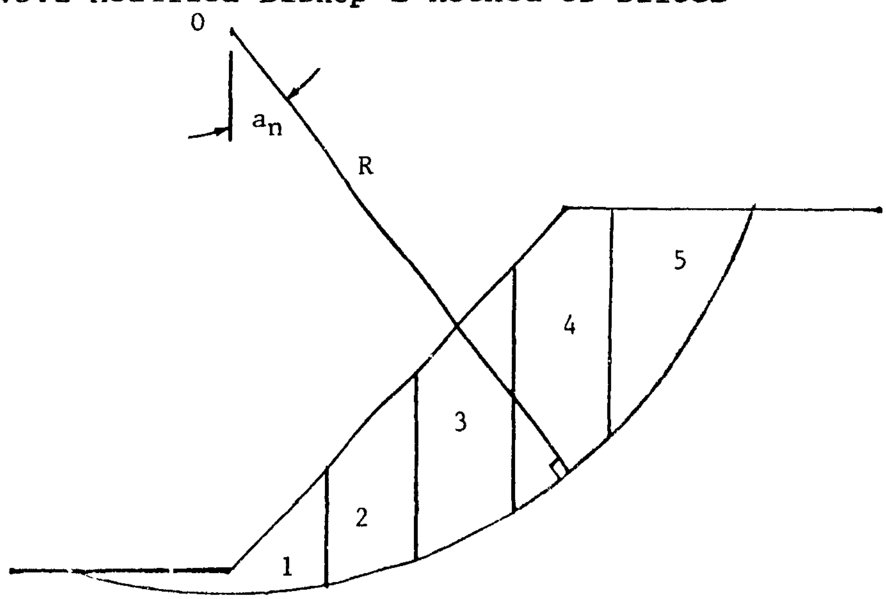


FIGURE 4a

A slip circle centered at 0 is assumed and the moving soil bank sectioned into slices 1, 2, 3, etc. The equilibrium of each slice of area A is examined as follows:

weight of slice $W = Y_t A$ (If the total density Y_t is not homogeneous, ie; sample slice is both unsaturated and saturated, then calculate areas using 1600kg/m^3 and 2000kg/m^3 respectively)

-Driving moment clockwise about point 0

$M_d = R\ W\sin a$

-Resisting moment taken counterclockwise

$M_r = R\ (\ C'L + (W\cos a - uL)\ \tan \phi')$

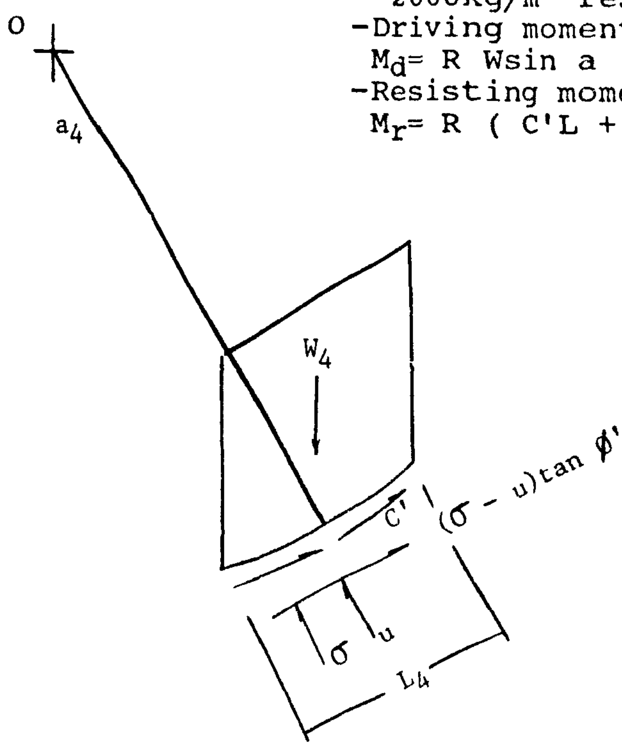


FIGURE 4b

When the driving and resisting moments are summed for all slices and divided to obtain the safety factor, then

$$S.F. = \frac{\Sigma M_r}{\Sigma M_d} = \Sigma \frac{C'L + (W\cos a - uL)\ \tan \phi'}{\Sigma (W\sin a)} \qquad (2)$$

where - L =length of arc segment

u =water pressure

$= Y_w hg$ and h=head of water measured from the top of the water table for that slice

Y_w=density of water

g=gravity

ϕ' =effective internal angle of friction

C' =effective cohesion

W =weight of soil slice

a =angle subtended by trial radius from the vertical to the arc centre.

2.5.3 Circle Moment Method

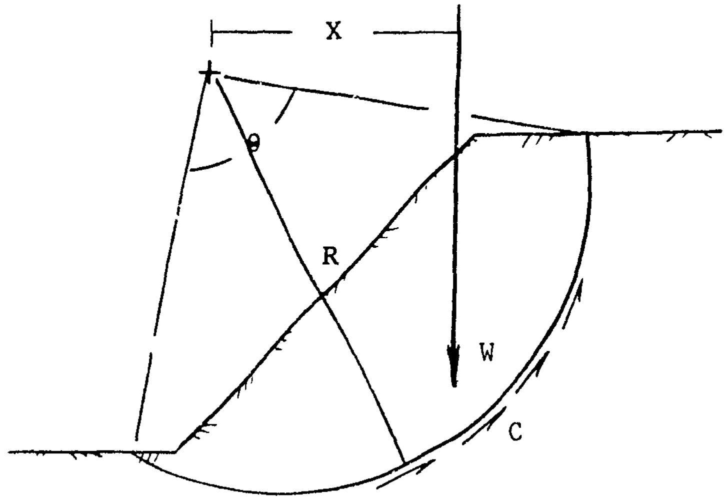


FIGURE 5

The resisting force is provided by the shear strength. If the undrained cohesive strength, C, acts along the arc segment, whose radius is R and arc angle θ, then the resisting moment is:

$$M_r = CR^2\theta \quad (3)$$

The driving moment is provided by the weight of the water and soil. If the total weight acting against the resisting force, is taken at its centre of gravity, as shown in the figure above, from the centre of the arc radius, then the driving moment is:

$$M_d = WX \quad (4)$$

The safety factor of the circular segment is then determined by

$$S.F. = \frac{\text{Resisting Moment}}{\text{Driving Moment}} = \frac{CR^2\theta}{WX} = \frac{N_s C}{YH} \quad (5)$$

Here, H is the slope height and N_s is called the stability factor. Graphs or tables of N_s have been calculated for the critical failure circle (smallest safety factor) for each slope angle, and are provided by Taylor (1948) or Scott (1963) and in other textbooks.

2.6 Methodology of Soil Strength Measurement

Soil strength parameters may be determined through laboratory or field tests. The former may be either by compression tests or direct shear tests. Testing procedures for both these methods may be found in standard civil engineering books. Turnbull (1948) performed direct shear tests on saturated and unsaturated samples of loess soil. His findings showed a large amount of consolidation and resulting settlement as the loess became saturated. This seems to be representative of the "pop-out" effect as observed by Bradford and Piest (1980).

Lutten (1969, 1974) used unconfined compression tests and both unconsolidated and consolidated undrained triaxial tests to obtain shear strength data for a Vicksburg loess. Tests produced varying results. The consolidated test revealed a breakdown of the soil structure resulting in loss of cohesion. A relationship between degree of saturation and shear strength was observed, with the increase in degree of saturation being accompanied by a gradual decrease in strength.

This theory may be further reinforced by Raghavan et al. (1975). Soil samples were tested for shear strength, and both the coefficient of cohesion and internal angle of friction were plotted versus moisture content. For various indices of plasticity, different relationships were observed. However, a similar pattern was noticed regardless of the index of plasticity in that at some moisture content there appeared to be an optimum point of shear strength. Beyond this point, however, soil shear strength decreased with the increase in moisture content.

There was one major problem which arose for all cases when solving for shear strength for high degrees of saturation. The curves developed by Raghavan et al. (1975) were obtained by the direct shear box test. When testing for shear strength of a saturated soil sample under any applied load, water is allowed to seep out of the sample, thus reducing its effective moisture content and not providing a true undrained shear strength. For this reason, it is preferable to test soil strength in the field, rather than in a laboratory, from the point of view of economy, time and preventing disturbance to in-situ soil structure.

Lohnes and Handy (1968) used a recently developed device, the Iowa Borehole Shear Tester, to determine in-situ drained direct shear strength in boreholes drilled into ditch banks. This borehole tester permits an on-site determination of cohesion and friction angle in the field.

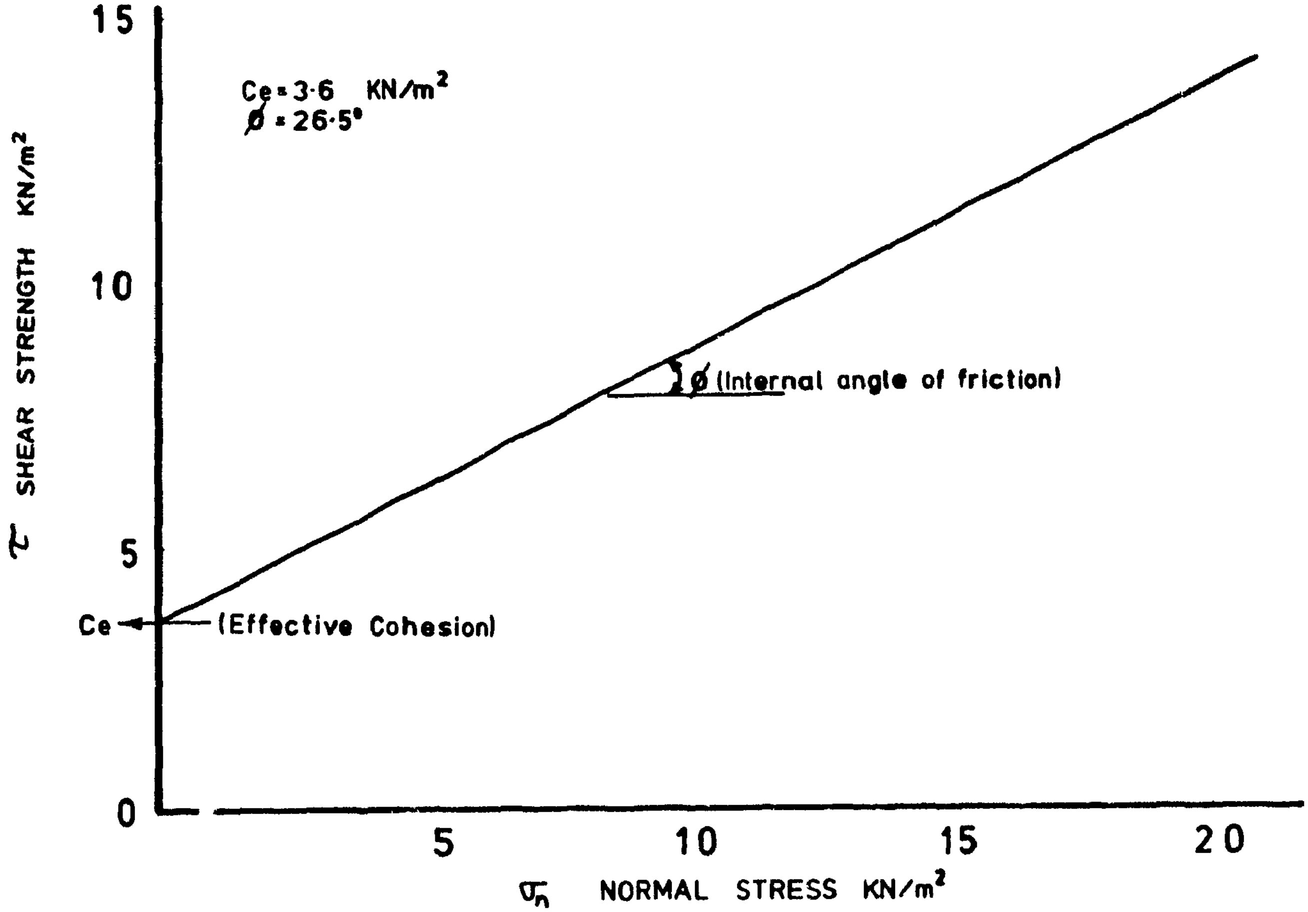
An even simpler instrument, although less accurate, is the Shear Graph. This device, which has been selected for soil testing in this study will be covered in more detail in the following section.

2.7 The Shear Graph and its Application

The Shear Graph is an instrument designed to determine a soil's in-situ effective cohesion and angle of internal friction. These two parameters are representative of the soil's shear strength. The operation consists of manually applying an axial load on the cylindrical device followed by a twisting motion. The torque is applied until a maximum resisting force, denoted by a shear failure of the soil, is achieved. Figure 6b illustrates the Shear Graph.

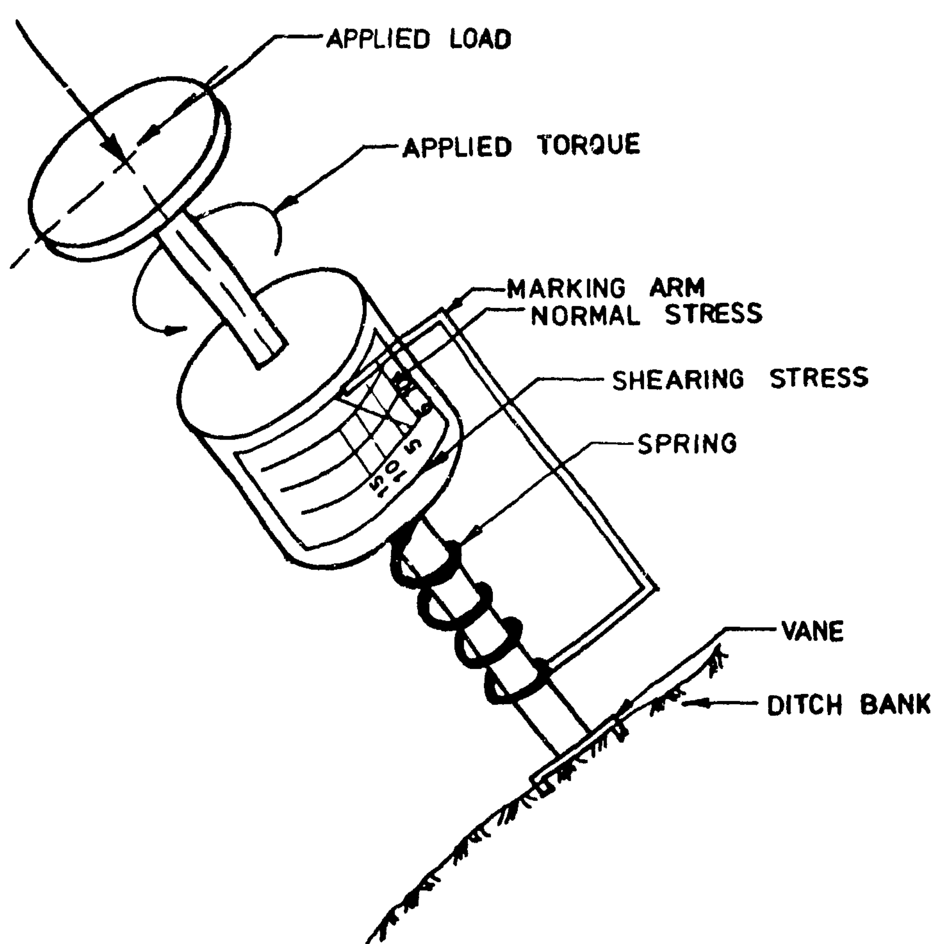
The shear strength of the soil and the corresponding normal strength are shown by a pen marking on a graph. This procedure is repeated for various applied loads until a number of points have been established to draw a straight line. Extrapolating this straight line to the Y-axis, or shear stress line, will give the soil's effective cohesion shear strength. The angle which it makes with the horizontal, or normal stress line, is taken as the internal angle of friction for that soil. A sample graph is illustrated in Figure 6a.

While the apparatus is effective in assessing the stability of an existing bank at a given time, it has limited uses in investigating soils for the purpose of bank design prior to ditch construction. Significant information, however, has been gained regarding bank failure through the use of this apparatus.



SHEAR GRAPH

FIGURE 6a



APPARATUS

FIGURE 6b

2.8 Groundwater Seepage

When an unbalanced water head occurs, as in the case of a loaded streambank with little stream flow, water is driven through the soil mass from a region of high pressure to one of low pressure. Energy, expressed as head, is described in three forms: position, pressure and velocity head. In soils, the resistance to flow is so great, that the velocity head is assumed to be negligible.

The flow of water through this variant of energy is best described by Darcy's law which expresses the seepage quantity as a function of the hydraulic gradient and the hydraulic conductivity over the total cross-sectional area normal to the direction of flow.

$$Q = KiA \qquad (6)$$

where Q = discharge m^3/sec

K = hydraulic conductivity m/sec

i = hydraulic gradient m/m

A = cross-sectional area m^2

The hydraulic conductivity, K, is defined as the water of flow per unit area of soil under a unit hydraulic gradient, and is expressed in units of velocity.

2.8.1 Flow net analysis

The flow of water through a saturated soil can be represented by paths of flow of particles of water. The different levels of energy or head can be represented on the same picture by equipotential lines which are lines indicating points of equal head. In support of Darcy's law, the flow net is firstly based on a saturated soil, secondly that the volume of water in the voids remain the same during seepage and lastly, that the coefficient of permeability remains constant at all points and in any direction at any point (i.e. a homogeneous isotropic material).

This criterion is derived from the equation of continuity which takes the following form for two dimensional flow. Here u and v are the horizontal and vertical components respectively of apparent water flow velocity.

$$\frac{du}{dx} + \frac{dv}{dy} = 0 \qquad (7)$$

Darcy's law indicates the components of discharge in a two dimensional flow to be:

$$u = -K \frac{dh}{dx} \quad \text{and} \quad v = -K \frac{dh}{dy}$$

$$\frac{d\left(-K \frac{dh}{dx}\right)}{dx} + \frac{d\left(-K \frac{dh}{dy}\right)}{dy} = 0 \qquad (8)$$

and, where K is the constant permeability for a homogeneous isotropic soil. The more common form known as the Laplace equation can be developed as shown below.

$$\frac{d^2h}{dx^2} + \frac{d^2h}{dy^2} = 0 \qquad (9)$$

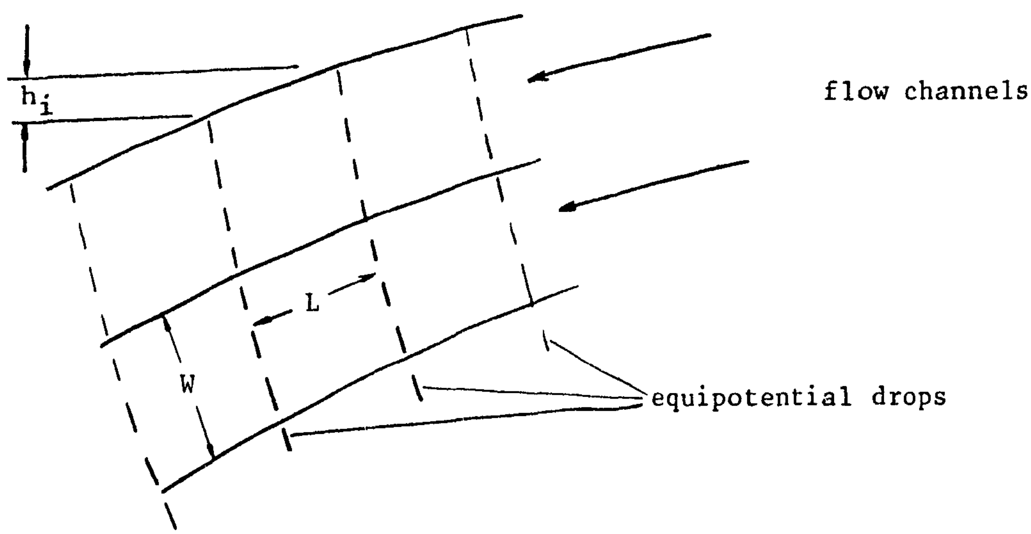


FIGURE 7

2.8.2 Flow Net Construction

Flow lines combined with equipotential lines form a pattern called a flow net. A number of flow channels, N_f, are selected so the flow through each, q, is the same. The total net head, h, is divided equally so that the equipotential drops, N_d, are the same when W = L for each flow compartment.

$$q = \frac{Q}{N_f} \qquad (10)$$

$$\text{and } h = N_d h_i \qquad (11)$$

The hydraulic gradient is given between successive potential drops by

$$i = \frac{h_i}{L} = \frac{h}{N_d L}$$

so

$$Q = qN_f$$

$$= KWN_f i$$

$$= Kh\frac{N_f}{N_d} \qquad (12)$$

which represents the basic equation for the computation of seepage quantities from flow nets having equal distance at each place between flow and equipotential lines.

2.8.3 Hydraulic Conductivity Measurement

To measure the hydraulic conductivity of soil below the water table the single auger hole method (Van Beers, 1958) is applied. Generally, the auger hole method is a field measurement which involves boring a hole to a certain depth from the soil surface and water table. Once the water in the bore hole has maintained an equilibrium with the surrounding water table, a certain amount of water is pumped out of the hole. The rate at which the water seeps back into the hole can be calculated to determine the soil's hydraulic conductivity. The procedure is carefully outlined in a bulletin released by the International Institute for Land Reclamation and Improvement, Netherlands (1979).

The auger hole method assumes the soil to be isotropic and determines the average permeability of the soil layers from the top of the water table to the bottom of the sample hole. Ideally, measurements should be conducted under high water table conditions such as in the spring or following prolonged rainy periods.

2.9 Effect of Vegetal Linings on Stability

Plant roots increase soil strength directly by mechanical reinforcement and, indirectly through water removal by evapotranspiration. The former effect is probably the more important because ditch bank slides occur most often during rainy periods when soil water potentials can rise to zero, regardless of vegetation.

When vegetal cover is present, the evapotranspiration is increased, thus reducing moisture content and placing the water molecules of the soil in tension. The direct effect of the tension of the moisture is to compact the soil and increase its shearing resistance.

From an hydraulic point of view, the vegetation protects the channel by reducing the flow velocity at the soil surface, hence minimizing the hydrostatic shear. Dense stands of long stemmed vegetation provide deep mats where velocity is at a minimum in these vegetal zones. It follows that a uniform dense cover with good stand and relatively deep penetrating root system will supply a high resistance to scouring. Grass roots help hold the whole bank surface together as a unit and thus distribute the forces and the tendency for erosion at the lower part of the bank. However, if the bank is too steep, a slip circle failure can occur to a depth below the root zone and cause a slumping of the whole bank. Thus, vegetation should be considered to prevent sheet erosion and to provide some additional insurance against bank failure, but it cannot be expected to hold a bank as originally designed if the bank is too steep for basic force stability.

CHAPTER III

OBSERVATIONS AND EXPERIMENTAL DESIGN

3.1 Observed Erosive Phases on a Streambank

Before test procedures were established, general observations were made of the nature and features of watercourse bank instability. Open channel drains constructed in unstable soils generally demonstrate a similar pattern of streambank erosion. An order of events is illustrated describing the transformation of a trapezoidal designed streambank into a stable parabolic shape. The following sequence of diagrams describes these patterns and subsequent action of an eroding streambank.

Figure 8a depicts the forces to which a typical trapezoidal ditch bank is subjected. A bare streambank would be susceptible to both rill and sheet erosion. Overland runoff from adjacent cropping land concentrates into rills which run down the ditch bank to the base of the surface drain. In sheet erosion, the impact of raindrops disturbs the structure of the surface layers of the ditch bank. Flowing water in the channel during a runoff event loosens and detaches soil particles along the wetted surface of the ditch bank when the resistance of the soil is exceeded by the hydrodynamic shearing force of the water.

All these erosive forces are conducive to streambank instability and produce a loss of streambank material. Figure 8b illustrates the effects of these erosive forces. In sections of open channel drains where water velocities are high, scouring at the base of the drain results. This renders the streambank less stable in basic force stability as it increases the effective local bank slope angle. In addition, overland runoff can create concentrated rills along the surface of the ditch bank which will eventually develop into larger gullies.

Overall intergranular strength of surface layers of the streambank are reduced due to the impact of the raindrops. This loosening of the surface structure will result in soil particle detachment during a runoff event.

Some of the most destructive forces which can produce bank instability are those associated with internal soil seepage. In homogeneous clays, these forces are contained by the internal strength of the soil, known as cohesion, and hence lead to no unstable effect. In sands, however, a high water pressure creates a boiling or quicksand effect, and the soil loses its strength and sloughs into the channel. The more common type of failure in the sands is this piping effect due to seepage.

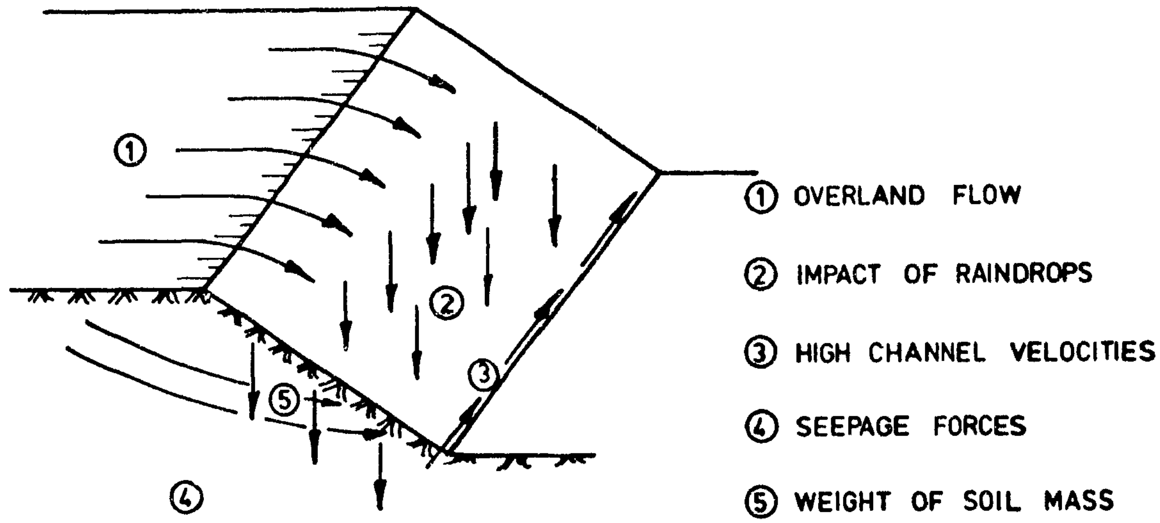
In some cases, slumped ditchbank material will remain as an obstruction in the open channel when the sediment load exceeds the transport capacity of the channel flow. This condition usually gives rise to continual problems as the slumped material acts as an obstruction and deflects channel flow into the side of the ditch bank, further scouring and eroding the streambank.

Tension cracks develop in a cohesive soil and are usually initiated during wetting and drying cycles upon the swelling and shrinking of the soil. Once these openings have been established at the berm of the ditchbank, subsequent failure may follow during wet periods when the weight of the ditchbank increases due to water infiltration and internal soil strength decreases. Failure of the ditchbank will act along a slip line of least resistance, and will be compounded by the tension cracks which provide an escape route for surface runoff.

In times of heavy precipitation, water runs down the tension crack further reducing the resistance of the soil. This process will lead to slumping as shown in Figure 8c. A combination of bed scouring and saturated weight of the soil mass results in slumping of the ditch bank material along a slip line which runs from the berm of the watercourse to the base of the ditch. This slumped material will remain in the ditch bottom until another runoff event takes place.

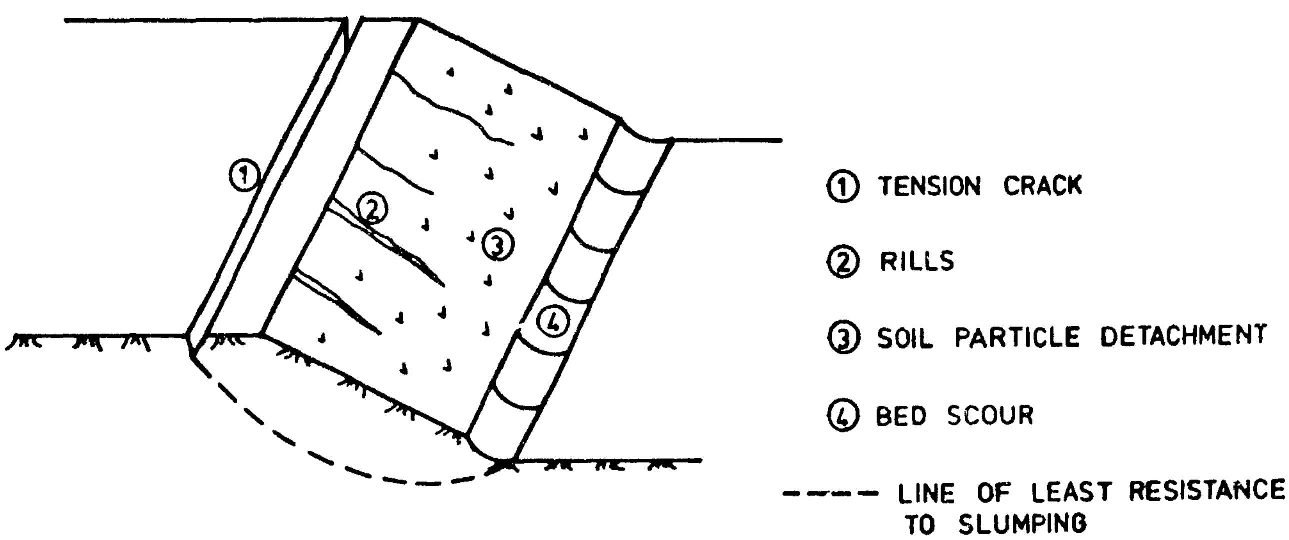
The continual effects of seepage forces, repeated saturation/unsaturated cycles of the soil and high channel velocities, will result in further scouring of the soil material, as shown in Figure 8d, allowing for repeated varieties of ditch bank slumping. The displaced bank material will eventually wash away following a series of rainfall events until the channel has stabilized into a parabolic shape, and is diplayed in Figure 8e. At this point, the erosion and sedimentation processes will have decreased considerably, but large quantities of soil have already been lost and may be deposited somewhere downstream.

TRANSITION OF A TRAPEZOIDAL TO A PARABOLIC SHAPE OF A STREAMBANK THROUGH EROSION



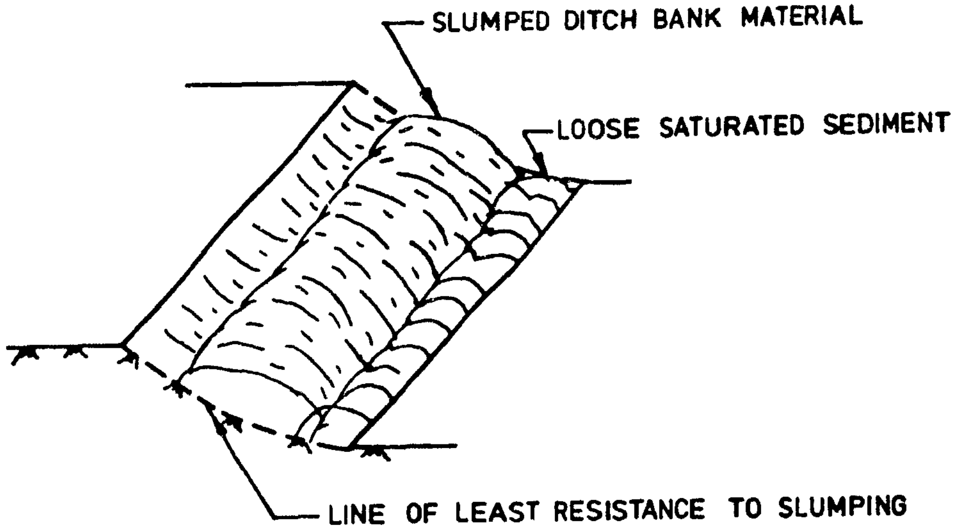
EROSIVE FORCES ON AN UNPROTECTED STREAMBANK

FIGURE 8a



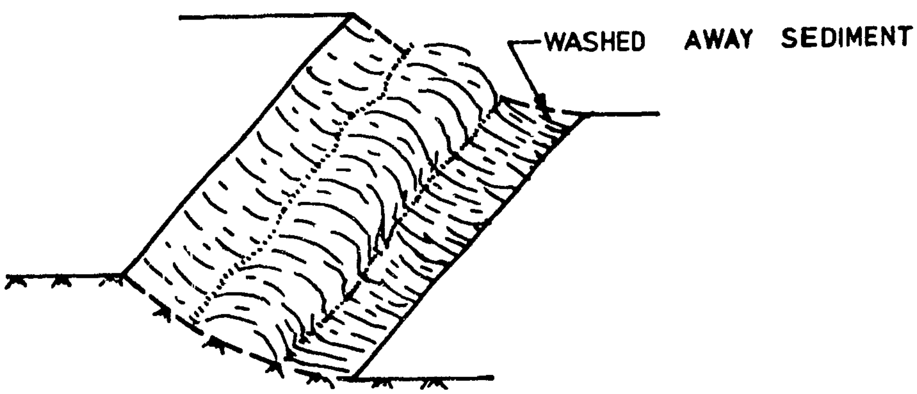
PRIMARY EFFECTS OF EROSIVE FORCES

FIGURE 8b



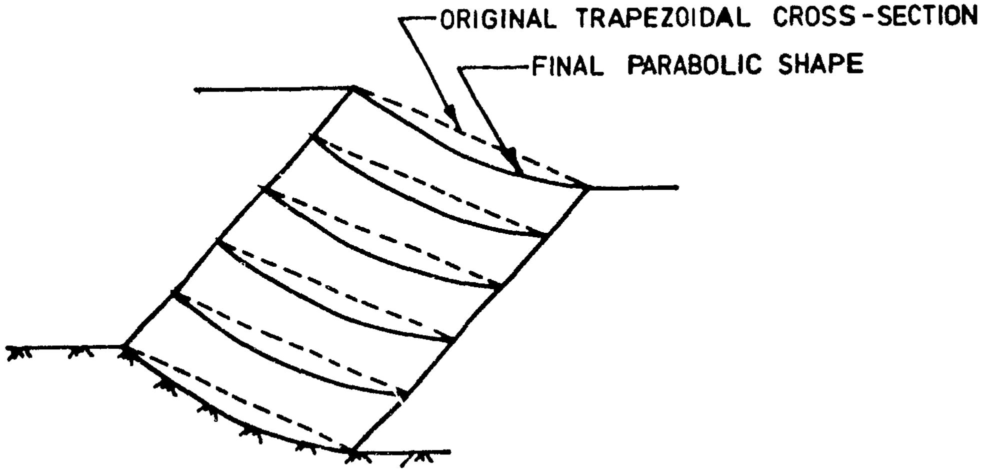
PRIMARY STAGE OF STREAMBANK EROSION

FIGURE 8c



SECONDARY STAGE OF STREAMBANK EROSION

FIGURE 8d



FINAL ERODED SHAPE OF DITCH BANK

FIGURE 8e

3.2 Experimental Sites

The experimental sites, through contact with consultants, government officials and from experience were selected based on their history of having exhibited streambank instability at some stage in their lifetimes. In particular, four experimental sites were chosen for closer monitoring and soil testing to further characterize the unstable soil. These sites had demonstrated, and were still demonstrating severe streambank slumping and erosive conditions, indicative of the modes of erosion discussed in section 3.1. As such, they provided an excellent resource in establishing an unstable soil analysis. The experimental sites are all located in the eastern Ontario - St-Lawrence Lowlands and are representative of the major soil groups in the eastern Ontario and western Quebec region.

Of the four experimental sites, three are municipal drains and one a sewer outlet, each with similar streambank instability conditions. Particularly, the North Morrow, Lower York and Hammond municipal drains, and the Vars sewer outlet all underwent extensive soil testing to determine their instability characteristics. The experimental sites are described in Appendix "A", and the field investigations to which were applied are detailed in the following section.

3.3 Field Investigations and Soil Tests

3.3.1 Water table readings

Throughout the course of the year, the water table can fluctuate from the soil surface during a state of saturation, as in the spring thaw, to severe dry conditions when the water table may be located far below the ditch bottom.

The former case is of interest from the points of view of groundwater seepage and the gravitational force of water in the streambank. As discussed in Section 2.3 it has been observed that streambank failures occur predominantly following prolonged wet periods where matric potentials, or suction pressure, in the soil can rise to zero. In this instance, all the major forces acting towards streambank failure are at an optimum since the weight of the soil mass increases, as do the seepage forces. Accordingly, measurements were conducted during the spring thaw to ensure that readings for worst case conditions could be obtained. More precisely, each of the experimental sites was monitored over a 20 day period during the spring to best determine the streambank's weakest state. The readings are tabled in Appendix "B".

In determining unstable conditions relative to water levels in the soil, all of the experimental sites were fitted with water table tubes which would permit the measuring of these fluctuating water levels.

In each case, three or four water level tubes were installed in a line perpendicular to the watercourse at a distance of up to 15m from the edge of the streambank. In this manner, enough points could be generated to interpolate a phreatic line from a distance where the water table is level to a point where the water table intersects the face of the streambank.

The installation and exact location of the water table tubes were dependant on the depth and location of the interceptor tile drain as described in section 3.3.5, and varied from site to site. In general, however, the water level tubes were installed in the following manner. The first water level tube was placed on the face of the streambank to establish the intercepting water table surface line with the streambank. The second tube was located between the first tube and the interceptor drain. The third tube was placed beyond the interceptor tile drain away from the streambank and the last up to 15m into the field to establish the average field water table depth.

Each tube measured 1.5m in length with an inside diameter of 19mm. All tubes were perforated with 6.5mm diameter holes punched at regular intervals along the length of the tube, enabling the soil water to enter freely. A filter cloth wrapped around the tube prevented the entry of sand particles and a removable cap was placed on the top of each tube to prevent surface water entering the tube and impairing actual watertable measurements.

3.3.2 Hydraulic conductivity of soil

To measure the hydraulic conductivity of a given soil, the single auger hole method as discussed in section 2.8.3, was followed. To determine whether a soil is potentially unstable, hydraulic conductivity measurements were conducted using boreholes at different depths. For instance, a layered soil with two distinct soil layers would result in two different magnitudes of hydraulic conductivity. If the hydraulic conductivity of the deeper borehole is in the range of a few cm/day while the hydraulic conductivity of the shallower borehole is in the magnitude of a few metres per day, then there is a potential for the formation of a pressure head over the deeper layer. This produces high seepage forces acting along the layer interface and groundwater flow towards the outlet channel will occur, especially under spring conditions when adjoining lands remain saturated for an extended period of time. Often it will be evident that the hydraulic conductivities are significantly different and thus high lateral flows will occur through the channel banks.

In determining the soil's hydraulic conductivity, the single auger hole method, as described in section 2.8.3, was followed. In each case, three bore holes were dug for each experimental site in close proximity to the streambank to best determine actual seepage conditions. Borehole depths were dependent on the textural change of soil as the case may be, but in general they were 1.5m deep to determine the upper hydraulic conductivity, K_1, and 2.4m deep to determine the lower hydraulic conductivity, K_2. The soil type and variability of hydraulic conductivities determined the spacing of test holes. Measurements are to be conducted under conditions of high water table. In this case measurements were taken in the spring. The results are included in Appendix "A" under each watercourse description.

3.3.4 Soil property identification

3.3.4.1 Soil survey maps

Soil survey maps have been developed by the Department of Soils, Ontario Agricultural College, Guelph and the Research Branch, Canadian Department of Agriculture. From an agrarian point of view, they provide a detailed analysis of the surface soil texture on a large scale. The maps are primarily used as a quick reference guide in aiding the designer as to what kind of soils to expect. However, information is inadequate for the design of open channel drains. A good description of each soil series and type is given for all regions in eastern Ontario and western Quebec, but is limited to the top metre or so of depth. As an unstable soil may be a result of multi-layered soils, further field investigation is required to properly assess the underlying soils.

A separate description of the soil type, as described by the soils map, is supplied for each of the four experimental sites listed in appendix "H".

3.3.4.2 Bore hole test

Many drains are designed on a visual examination of the soil surface. However, in many of the problem soils, further investigations below the soil surface would reveal underlying soil material of a different nature.

To this effect, test holes were bored to below the depth of the watercourse at various depths in order to produce a soil profile. The test holes were dug at standard spacings and specifically in areas adjacent to localized slumping and erosion to determine if indeed a change in soil texture exists.

As the holes were being bored, soil was removed from the tip of the auger and examined. At this point, the soil sample is carefully inspected. Thin layers of soil may be extremely difficult to detect but could be quite significant from the point of view of instability.

To develop a soil profile, soil samples were extracted from the auger tip, measured from the soil surface, tagged for location, placed in plastic bags and returned to the laboratory for further testing.

3.3.4.3 Grain-size analysis

The soil samples obtained from the field for classification were analyzed using the grain size analysis as outlined in Section 2.2.1.

A soil profile could then be developed for each watercourse. Depending on the variability of the soil, a minimum of two samples were taken at different depths.

Percentage of fineness curves were developed from the grain size analyses and classified according to the USDA Soil Conservation Service classification system. Each soil sample for each experimental site which was analyzed for a particle size distribution, is graphed and displayed in Appendix "C".

3.3.4.4 Atterberg limits

Two of the four experimental sites, namely the Lower York and North Morrow municipal drains were classified as cohesive soils and subsequently tested for basic force stability using the axial shear graph.

Soil samples were extracted from the same location as the axial shear vane measurements were performed, then tested in the laboratory for their plasticity index.

Atterberg limits were tested as described in Section 2.2.2.

3.3.5 Installation of interceptor tile drain

Interceptor tile drains were installed along the ditchbanks in all the experimental sites. These parallel interceptor drains were installed to intercept the seepage water which tends to move soil in the ditchbank towards the toe of the slope, as soil water flows from a zone of higher pressure towards that of a lower pressure, located at the ditch bottom. The subsurface drains were installed as close as possible to the open drain yet at a depth slightly above that of the drain bottom. They were installed low enough to maximize the lowering of the water table adjacent to the slope but high enough to ensure adequate outlet conditions. In all cases, the tile drains were placed at a slope equivalent to that of the average slope of the watercourse, along the experimental section.

The depth of the interceptor drains was limited to the installation capacity of the trenchless plough which allowed a maximum depth of 2.0m from the surface. The interceptor tiles were 100mm diameter perforated corrugated tube wrapped with filter material, with lengths of up to 175m depending on the experimental site. Average distance of the tile drain from the ditch bank was approximately 2m. This distance is governed by the track width of the trenchless plough.

Water table measurement tubes, as described in section 3.3.1, placed perpendicular to the ditch, made possible an evaluation of the water table during critical periods of the year. By closely monitoring the fluctations of the water table along the ditchbank, the effectiveness of the tile drain in reducing seepage and water content in the streambank could be evaluated.

3.3.6 Shear Strength Measurement

The presence of water in the soil has a distinct effect on the behaviour of a fine grained soil. In addition to the water content, it is necessary to compare this behaviour to a set standard. The Atterberg limits define these critical limits between water contents for cohesive or fine grained soil, when soil may bahave in a different manner.

To obtain the soil's shear strength, the soil's cohesion and intergranular angle of friction were determined as a function of moisture content using the shear graph. These strength measurements were used in correlation with the ditchbank shape and depth of drain to determine the safety factor of a ditchbank. Two of the four watercourses selected for testing from the experimental sites are fine textured and thus may be tested for slope stability using the Method of Slices or the Circle Moment Method. The latter method, as described in Section 2.5.3, assumes a saturated fine textured soil to have a negligible internal angle of friction.

Raghavan, et al. (1975), developed cohesion curves as a function of moisture content for various soil types. Clearly, these curves indicate that beyond some optimal moisture content, the soil's shear strength decreases to a point of least resistance.

To determine this minimal shear strength and thus greater likelihood of ditch bank slumping, measurements were conducted in the spring during the runoff period to obtain saturated or near saturated conditions. Because of the great fluctuation of the soil's shearing strength versus its water content, soil samples were collected at the same location where the axial shear graph tests were made to determine the moisture content.

Shear graph measurements were conducted at random locations along and up and down the streambank and at various depths. Due to the nature of the device, accurate measurements of soil shear were only made to a depth of 25cm. Beyond this depth, manipulation of the instrument became very unorthodox and produced inadequate results.

3.3.7 Vegetal Lining

All of the experimental sites were seeded with various seed mixtures. Many factors influence the selection of a vegetal lining to maximize growth, early root establishment and the ability of the vegetal lining to germinate rapidly and continuously.

Seeding generally will be most successful if carried out in late spring or late summer/early fall. Specifications normally recommend seeding within a 24 hour period after excavation of ditchbanks, to obtain desirable moisture conditions.

All the experimental sites were hand broadcasted in the late summer of 1983, with the exception of the Vars Storm Sewer outlet which was hydroseeded immediately following excavation of the watercourse.

The following mixtures were applied at different locations along the streambank within a single experimental site. The concentration of the seed mixture applied to the streambank was 60kg/ha as recommended by the Ontario Ministry of Agriculture and Food Field and Crop Guide, 1984. A heavier concentration was applied to the Millette Municipal Drain due to a change of seeding equipment. It was calculated after the seed was broadcasted on the streambank that 70kg/ha had been applied.

Ease of establishment and time required to develop a protective cover are extremely important considerations in selecting the correct mixture and vegetal type. Annual grasses are preferred for rapid growth, mixed in with the more native and hardy sod-forming grasses which require a longer period of time for establishment.

Climate and soil are the principal factors which determine the specie or preferable mixture. However, there are local conditions which contribute to the selection of a vegetal lining for a ditch bank. In this particular region of study, the single most important factor is the timing of seeding.

TABLE 3

SEEDING

Drain	Date of Seeding	Concentration of Seed	Seed Mixture applied on random locations		
NORTH MORROW	May 1984	60 kg/ha	0-100 m 100-200 m 200-300 m	'A' 'C' 'D'	Both sides
LOWER YORK	May 1984	60 kg/ha	0-100 m 100-200 m 200-300 m	'C' 'E' 'A'	Both sides
HAMMOND	May 1984	60 kg/ha	0-250 m 0-250 m	'D' 'A'	East Bank West Bank
LABRECHE	May 1984	60 kg/ha	Entire Length all branches	'C' 'D'	Both sides
CUMMING	May 1984	60 kg/ha	0-200 m 200-275 m 275-500 m	'B' 'A' 'C'	Both sides
MILLETTE	May 1984	70 kg/ha	0-1000 m	'D'	Both sides

SEED MIXTURES **TABLE 4**

Forage Mixture	Component	%Seed
'A'	FAIRWAY [1]	40
	TIMOTHY	40
	WHITE CLOVER (DUTCH)	10
	ALSIKE	10
'B'	FAIRWAY [1]	75
	ALSIKE	12.5
	WHITE CLOVER (DUTCH)	12.5
'C' [2]	BIRDSFOOT TREFOIL [3]	40
	RED FESCUE	60
'D'	FAIRWAY [1]	100
'E'	FAIRWAY [1]	65
	BIRDSFOOT TREFOIL [3]	15
	ALSIKE	10
	WHITE CLOVER (DUTCH)	10

OTHER MIXTURES

Recommended by the Ministry of Transport and Communications (1977)

CREEPING RED FESCUE	54%
KENTUCKY BLUEGRASS	22%
WHITE CLOVER (DUTCH)	07%
RED TOP	05%
COMPANION CROP	12%

Recommended in the Design Manual for Open Channel Drainage (1980)

CREEPING RED FESCUE	55%
PERENNIAL RYE GRASS	28%
WILD WHITE CLOVER	09%
BIRDSFOOT TREFOIL	08%

NOTES

1. "Fairway" is a commercial mix at 40% Kentucky, 35% Red Fescue, 25% Rye Grass.

2. Mixture "C" recommended for erosion control by the O.M.A.F. Field and Crop Guide, 1984.

3. For a legume such as Birdsfoot Trefoil, an innoculant at 227g per 25 kg of mixture is required.

CHAPTER IV

RESULTS AND DISCUSSION

4.0 Streambank stability

A soil may not necessarily be categorized as unstable based on a textural classification. In many of the sideslopes designed as safe, streambank failures result due to a condition of the soil. This implies that this condition be reached at some critical point when the shearing or resisting strength of the soil is exceeded by the driving or gravitational force of the soil mass of the ditchbank. However, it would appear that a number of soil types are more prone to these critical conditions which lead to streambank collapse.

Streambanks are at this weakest point during periods of prolonged wetness, when matric potentials in the soil can rise to zero. Accordingly, each of the four experimental sites were monitored over a period of 20 days during the spring thaw of 1984. A total of 9 readings were obtained and are tabled in appendix "B".

To determine which of these readings represent the worst case conditions, there must a point during a runoff event when the differential head between the watertable in the streambank and stage height in the ditch is at a maximum. During a runoff event, the watertable in the streambank will rise with the water level in the watercourse. At the peak stage height, the water level in the ditch will equal the watertable in the streambank. Under these conditions, with channels flowing at near full capacity, the effective safety factor in the submerged slope usually has a higher safety factor than the same slope unsubmerged. This stands to reason due to the pressure of water acting against the ditchbank, preventing the loaded streambank from collapsing.

In most cases, however, once the runoff event has peaked, the stage height in the watercourse will drop much faster than the water level in the streambank. The result is that the neutral stress within the slope cannot adjust itself to the new water level in the ditch, thus creating an imbalance of pressure on the streambank. This condition is termed drawdown and is common in a watershed with short times of concentration, when the stage height in the streambank fluctuates rapidly. It is at this point that calculations were made for worst case conditions.

In the following section each experimental site is discussed separately on the effects of lowering the streambank's water table with the introduction of an interceptor tile drain. By developing flow nets for both tiled and untiled sections as described in section 2.8.2, seepage forces acting against the streambank can be calculated. To compare both tiled and untiled cases under similar conditions, a slip line was first developed for the worst case condition in the untiled streambank. Generally, the procedure was followed as outlined in section 2.5.2 where an arc or slip line was drawn from the intersection of the water table in the streambank, to the water level in the ditch. This circular arc was then superimposed onto the tiled streambank section. At this point, both seepage forces as described in section 2.8, could then be calculated.

The following flow net diagrams for each experimental site, both tiled and untiled sections, illustrate the flow channels in blue with arrows pointing in the direction of flow, and the slip line and water pressure distribution, drawn in red.

4.1 Flow Net Analysis

4.1.1 Lower York Municipal Drain

The study section within the Lower York was reconstructed through previously undisturbed soil for realignment purposes. The site was chosen for remedial testing as it had demonstrated frequent sloughing of streambank material into the watercourse. Two drop structures were installed along the test site prior to this analysis due to the high channel velocities. There was one significant bend in the watercourse resulting in channel flow being redirected into the ditchbank causing unusual scouring at the toe of the slope.

The modes of erosion discussed in Section 3.1 exemplify the Lower York's streambank erosion. The high spring flows, undercut the toe of the slope and steepened the streambank. High surface runoff had created rills and gullies down the streambank face. High water tables coupled with freezing and thawing, wetting and drying cycles, led to tension cracking at the berm of the streambank.

An auger hole test revealed a contributing factor in the streambank's instability. Approximately 1 meter below the soil's surface, a clay layer was discovered 0.40 meters thick in an otherwise silty loam soil. The hydraulic conductivity was difficult to determine due to the thickness of the clay lens, but was nevertheless sufficiently low to create a perched water table above the clay lens.

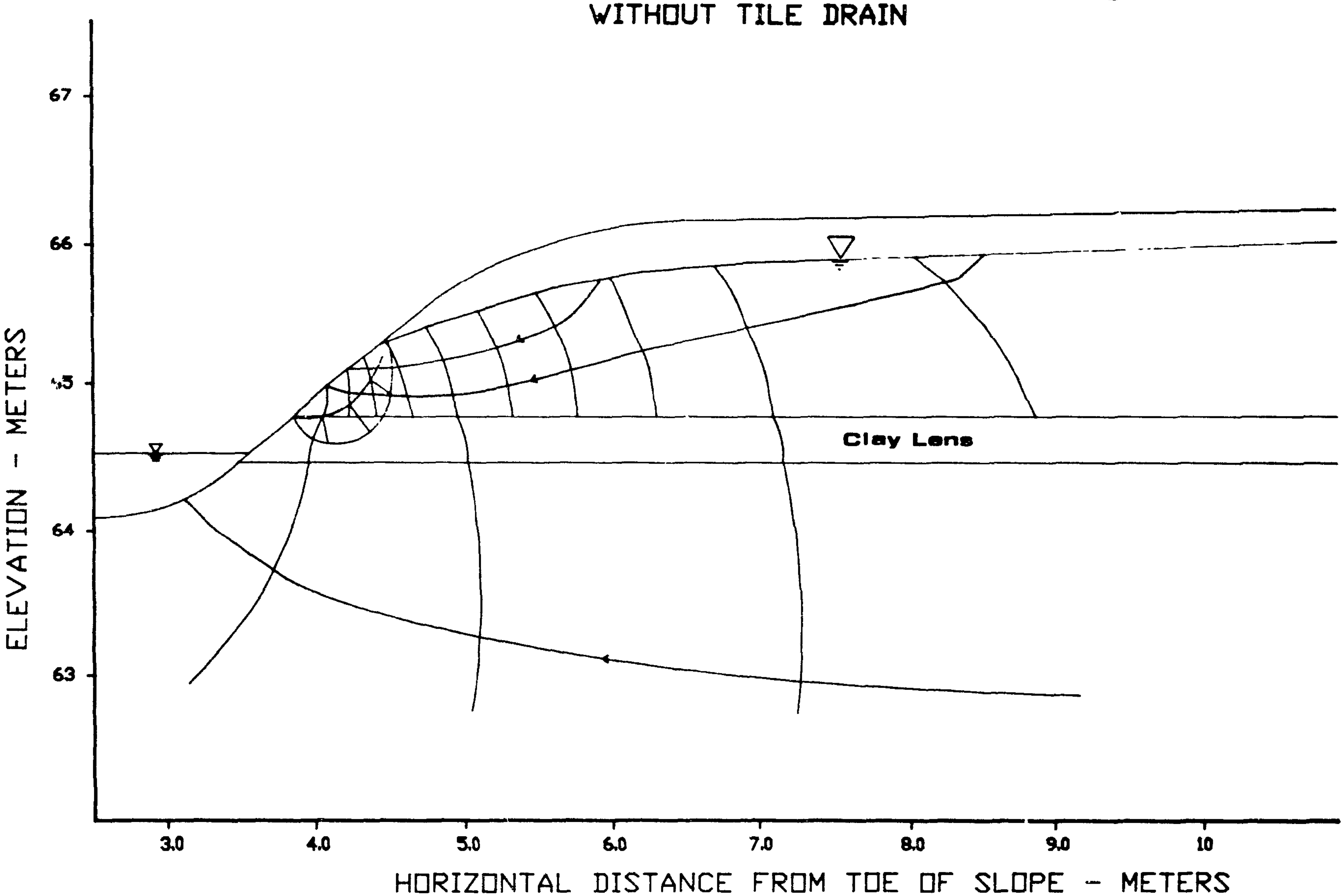
The adjacent topography surrounding the watercourse was steep resulting in increased surface runoff and greater seepage forces along the streambank. Evidence of these high seepage forces were the fine soil particles observed seeping out above the clay lens along the face of the streambank.

To reduce the seepage forces and thus lower the water table, a tile interceptor drain was installed along the ditchbank. Due to the variation of the depth of the clay lens, the interceptor tile was installed in and out of the clay lens. Figures 9a and 9b illustrate the location of the tile drain as at the bottom of the lens, with a wedge cut above the tile drain as created by the trenchless plough. It was assumed that the silty loam under saturated conditions would flow into the wedge developed by the cutting action of the plough, thus ensuring a porous medium for the overlying perched water table to drain through to the interceptor tile.

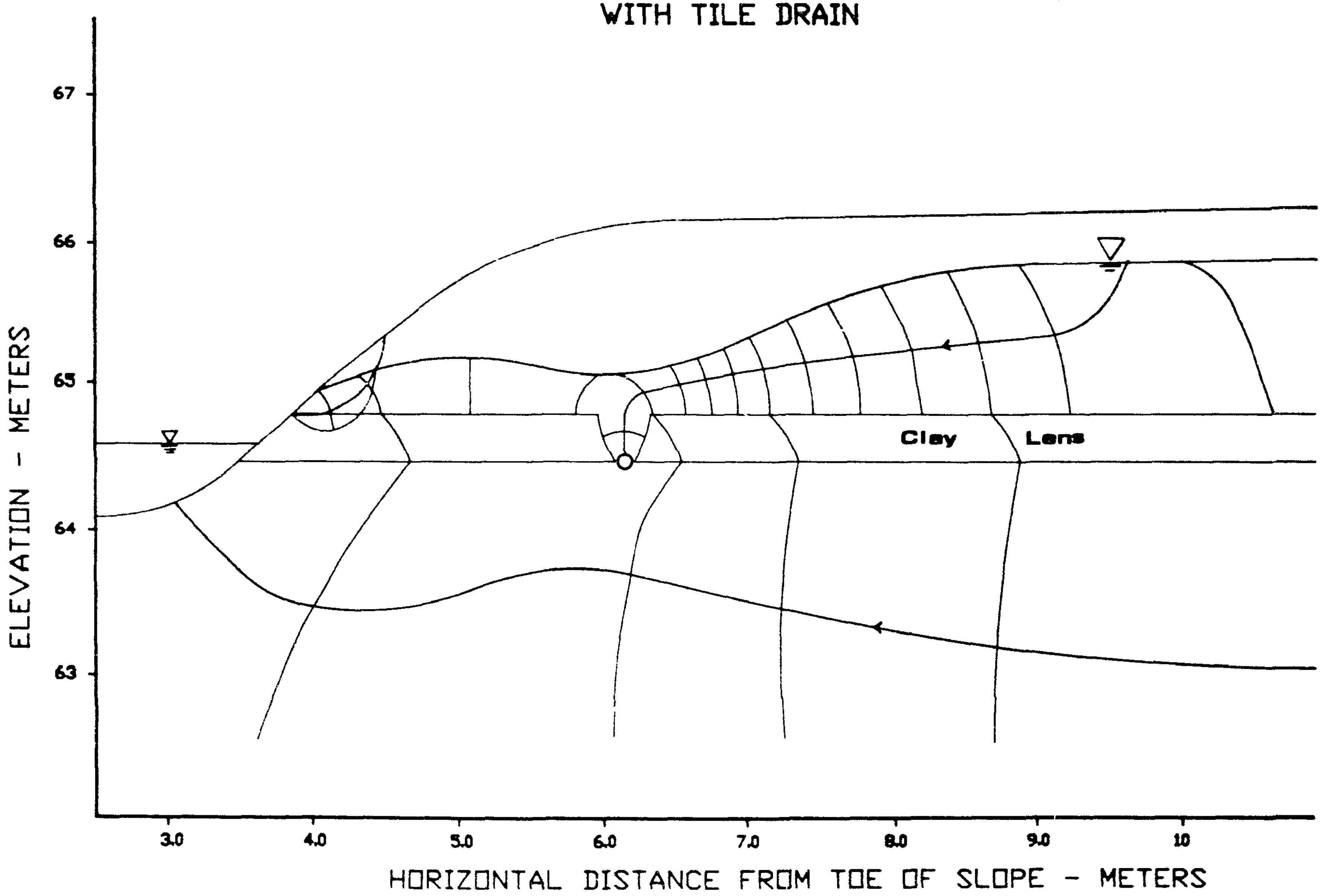
As shown in Table 5, the seepage forces are much less than the other watercourse. This is due to the confined slip circle taken to be above the clay lens where the fines were observed to be seeping out. However, the installation of an interceptor tile drain appears most effective in reducing the water table, as seepage forces under worst case conditions were calculated to be 60% less than for an untiled section.

LOWER YORK MUNICIPAL DRAIN
WITHOUT TILE DRAIN

Figure 9a



LOWER YORK MUNICIPAL DRAIN Figure 9b
WITH TILE DRAIN



4.1.2 Vars Storm Sewer Outlet

The Vars Storm Sewer outlet was constructed during the summer of 1982. Initial construction was delayed due to unusually high water levels in the soil. The fine sand and high water table resulted in sloughing of the newly excavated banks as construction proceeded. To correct this problem, interceptor tile drains were installed on each side of the proposed ditchbank up to 2.0m below the soil surface and up to 0.25m below the ditch invert. This removed the excess water in the soil and permitted a normal construction.

Many of the watercourses on the St. Lawrence lowlands which have exhibited streambank instabilities in one form or another reveal similar soil make up. For this reason, further investigations were made on this watercourse to determine the soil's profile and permeability. Results of the Auger hole tests on this site confirmed the relatively high permeability in a homogeneous fine sand. In frictional soils, particularly the medium to fine sands, sideslopes are constructed at 3h:1v (3 horizontal : 1 vertical) due to high lateral seepage forces along the streambank. The installation of an interceptor tile drain permitted sideslopes to be constructed at 2h:1v due to the reduction of water pressure caused by a high water table in the streambank. Throughout the spring runoff, water table tube #2 located above the interceptor tile and shown in appendix

"D", only registered readings on four occasions signifying that the tile drains were quite effective in lowering the water table.

As illustrated in Figures 10a and 10b for worst case conditions, four flow channels in an untiled section were subsequently reduced to two flow channels in a tiled section. As indicated in Table 5, the seepage force was calculated to be 42% less in the latter case, clearly reducing the incidence of slumping. The benefits of installing a tile drain as deep as possible is illustrated by the hump in the watertable between the tile drain and the ditch bottom, indicating that there is flow towards the tile drain and therefore away from the watercourse.

There is a definite advantage in installing the tile drain below the ditch bottom but in many situations outlet conditions do not permit such a design. In this case however, the Vars watercourse had an average slope of 0.3%, permitting the installation of tile drains below the ditch bottom at the minimum 0.1% slope.

Figure 10a

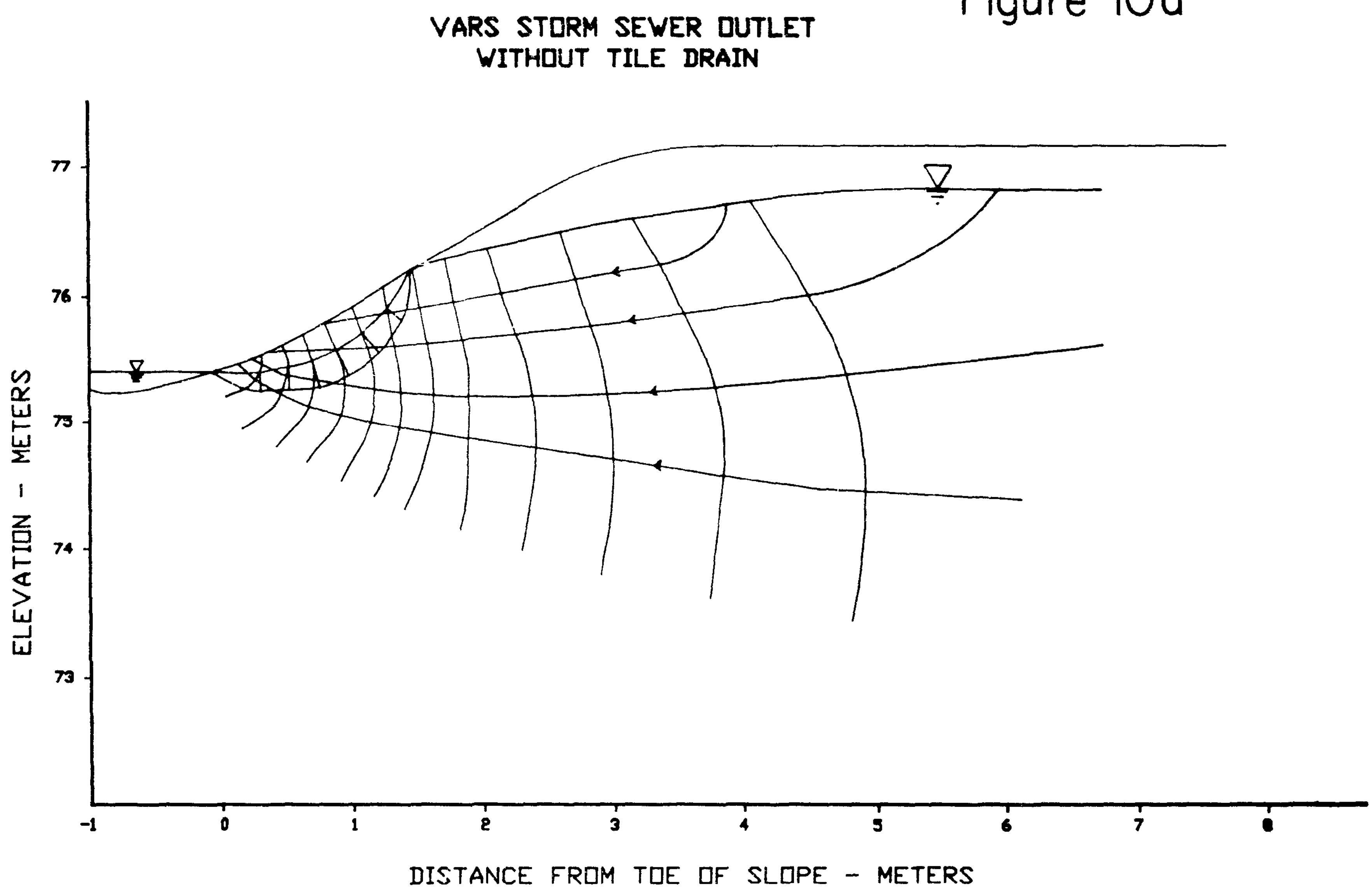
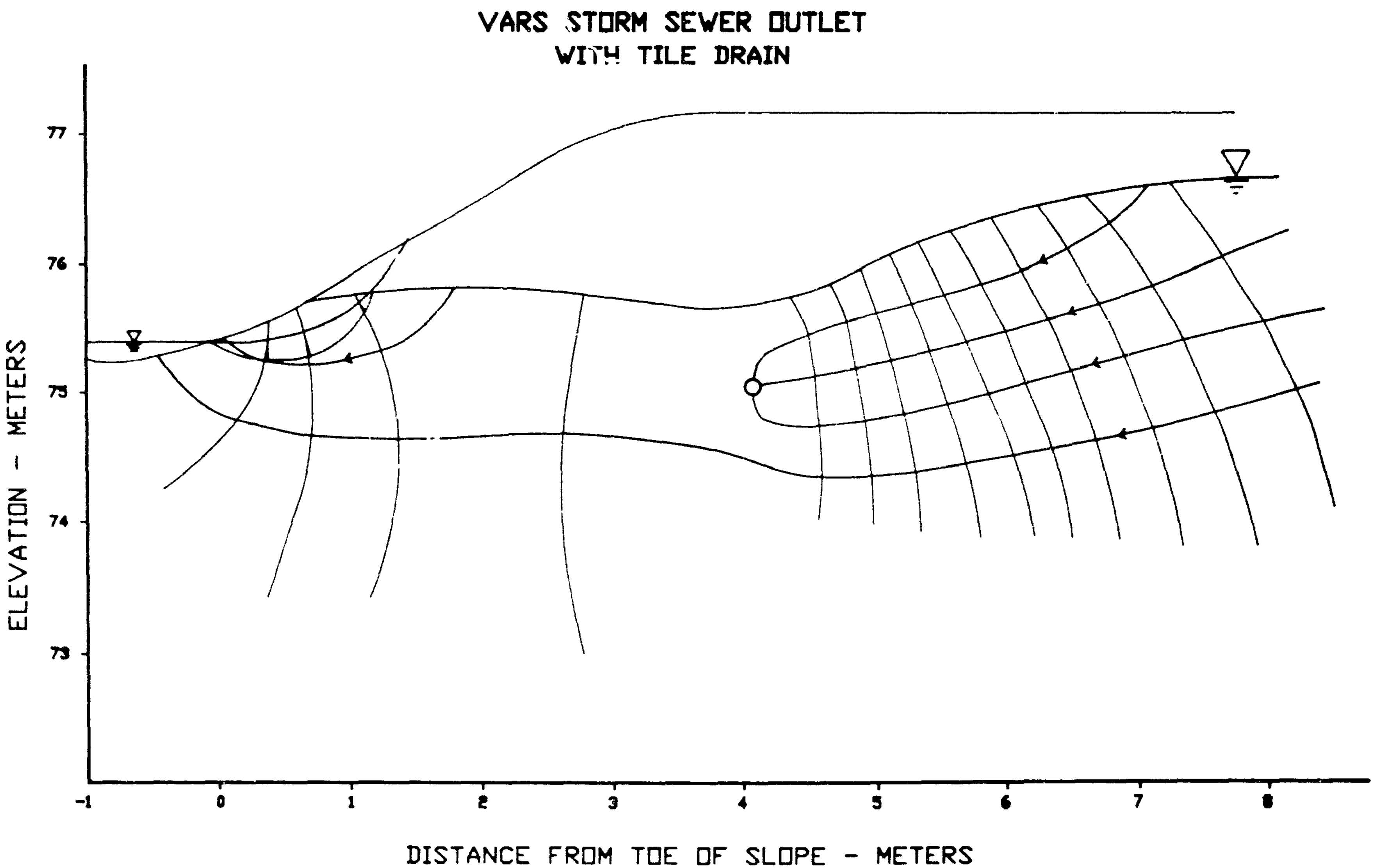

VARS STORM SEWER OUTLET
WITHOUT TILE DRAIN
ELEVATION - METERS
77
76
75
74
73
-1
0
1
2
3
4
5
6
7
8
DISTANCE FROM TOE OF SLOPE - METERS

Figure 10b


VARS STORM SEWER OUTLET
WITH TILE DRAIN
ELEVATION - METERS
77
76
75
74
73
-1
0
1
2
3
4
5
6
7
8
DISTANCE FROM TOE OF SLOPE - METERS

4.1.3 Hammond Municipal Drain

The Hammond Municipal Drain demonstrates a typical problem soil encountered in open channel drainage construction. It would appear at the outset that the open channel is constructed in a homogeneous fine sand with recommended 3h:1v sideslopes for this class of soil.

Along the bottom of the streambank above the lower base flow stage height, severe washing out of the sands was observed. This phenomenon known as soil piping takes place when the soil particles migrate to an exit point, in this case along the streambank face.

The adjacent land surrounding the watercourse is relatively flat with no noticeable regions supplying a higher than normal recharge area. Upon closer examination of the soil profile, it was discovered that a clay layer was approximately 1.5 m below the surface, underlying the fine sand. It is along this fine sand/clay interface that soil piping was observed. Permeability tests were carried out on site, and determined the hydraulic conductivity of the upperlying fine sand to be 2.5 m/day, and the underlying clay layer to be 0.3 m/day. This larger differential of permeability between the two soil materials appeared to be the source of the problem; a significant build up of water pressure in the sand above the clay resulted in a high lateral flow through the streambank face.

To reduce these high seepage forces and the extent of soil piping, an interceptor tile drain was installed. The tile drain was laid as deep as possible but above the clay layer so as to intercept the horizontal seepage forces in the sand.

The adjoining Figures 11a and 11b describe the success in the interceptor tile reducing the seepage forces. Four flow channels in the untiled section were reduced to two thus reducing the water pressure acting along the streambank face by 40%. As in the cases of both the Vars and Lower York, some subsurface flow was measured to be flowing away from the open channel to the tile drain as indicated by the rise in the watertable between the tile drain and the watercourse.

Figure 11a

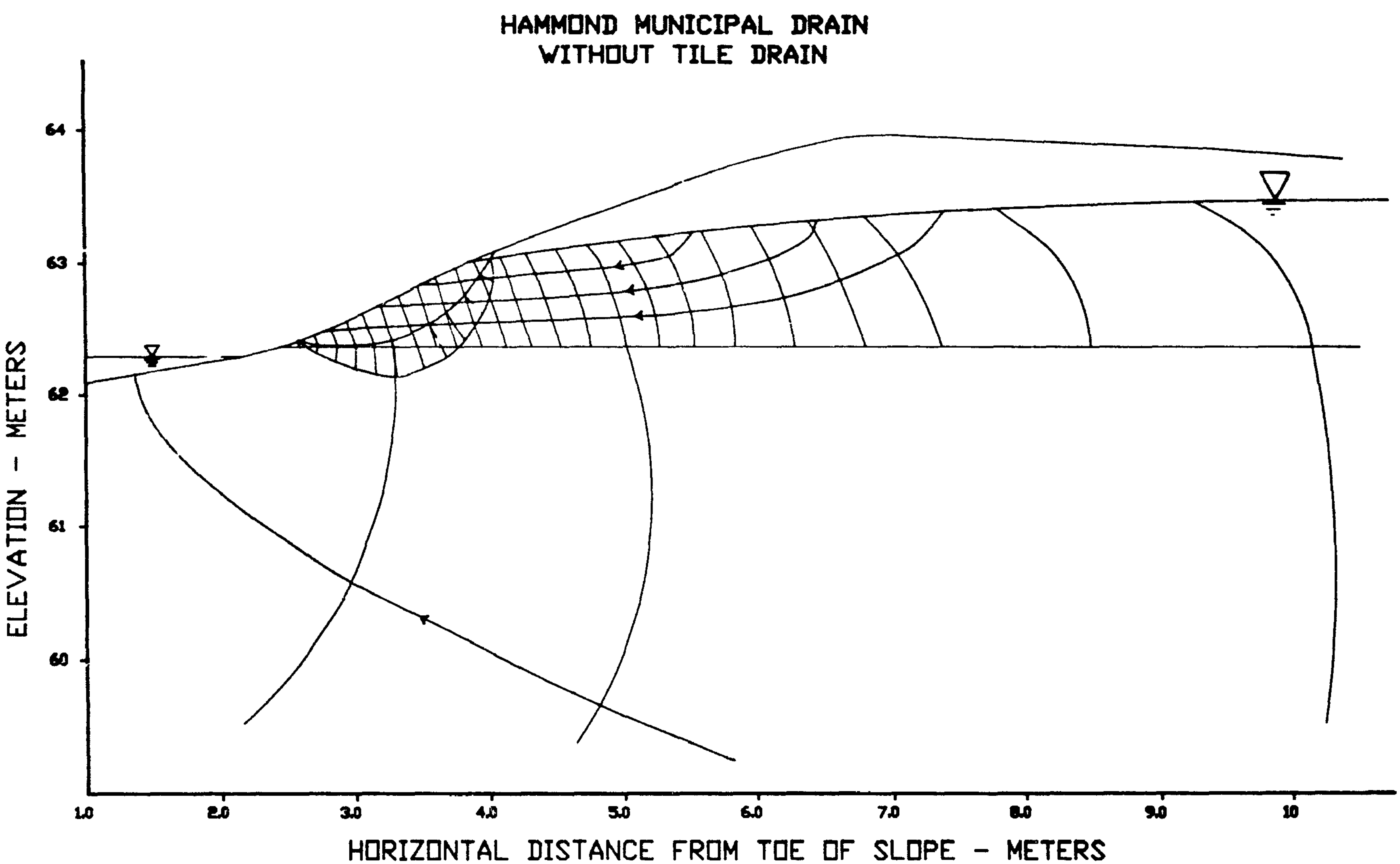
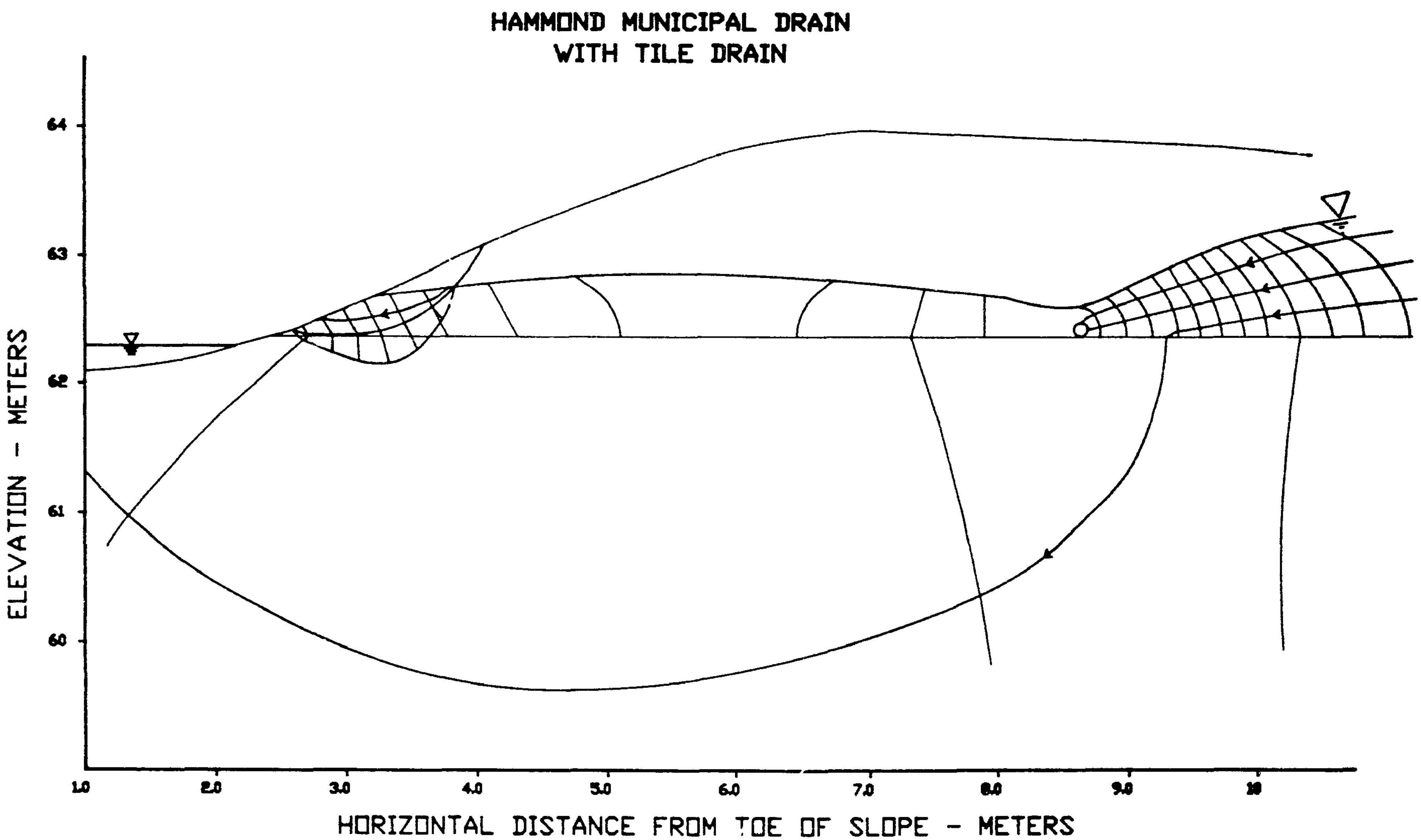

HAMMOND MUNICIPAL DRAIN
WITHOUT TILE DRAIN
64
63
62
61
60
ELEVATION - METERS
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

Figure 11b


HAMMOND MUNICIPAL DRAIN
WITH TILE DRAIN
ELEVATION - METERS
64
63
62
61
60
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

4.1.4 North Morrow Municipal Drain

Historically, the North Morrow along the test site has always proven troublesome. Continual slumping on both sides of the watercourse has generated large amounts of soil material to be washed away, resulting in a larger than designed channel capacity. The average grade of the channel within the test section is 0.1%, a little steeper than the average slope of the entire drain nevertheless quite flat. Despite the enlarged capacity of the watercourse within the test section, a high water level, approximately 0.75 m, would remain throughout most of the spring. At this water level, flows would remain negligible, indicating a back water problem. Scouring was not observed and is unlikely in the process of rendering the streambank unstable.

Slumping, however, was observed at different stages of development along the test site indicating high levels of water table and seepage forces. An on-site investigation of the soil profile would reveal something much more complicated than a Rubicon Fine Sand as identified by soil maps for this region.

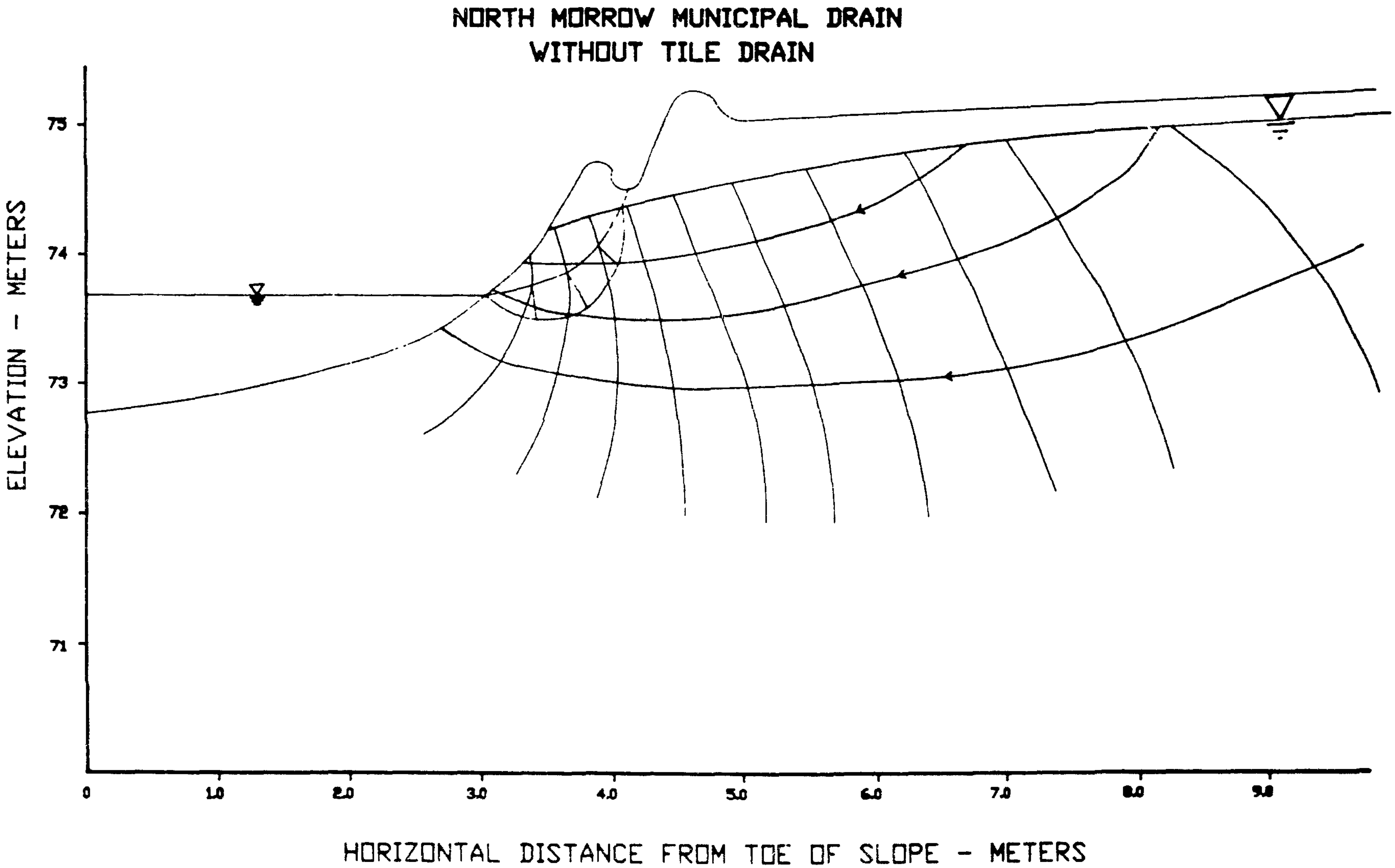
A close inspection of the soil removed from the auger during digging revealed a soil make-up highly susceptible to ditchbank slumping. Alternate this layers of clay, silt, and sand, known as a varved clay, make up the profile of this ditchbank. These thin laminae exit at the ditchbank face permitting the water contained in the coarser grained material, sandwiched within the finer grained material, to seep out. In this case, the water pressure in the sand is too large to be maintained by its internal strength resulting in washing out of both soil material and water through the ditchbank. The soil loss in the sand and silt layers results in a collapse of the clay layers above. This process usually initiates higher forms of slumping such as slab failure.

The prescribed auger hole method was only useful in determining the hydraulic conductivity of the upper soil horizon relative to that of the lower. It is not possible to determine on site, the hydraulic conductivity of each alternate layer which measures 2 to 5 cm in thickness. It is understood that following a rise in the water table, the flow lines are refracted at angles that vary according to the hydraulic conductivity of the soil layer. However, for the purpose of comparing the effectiveness an interceptor tile, the flow nets were developed on the basis of a homogeneous soil.

The results of an interceptor tile were the least successful of all the watercourses under examination as some slumping did continue after installation. There was, however, a noticeable decrease in the water table. From Figures 12a and 12b, it appears that the interceptor tile drain removed water immediately around the drain but was limited due to the relatively impermeable layers of clay surrounding the tile drain and thus impeding the passage of water to the tile drain. However, it is felt that the installation of the tile drain by the cutting action of the trenchless plough, broke up the varves and provided a pathway for the water held above in the confined silt and sand layers, to reach the tile drain.

In measuring the seepage forces for this particular watercourse, a slip line was drawn from a tension crack on a previously slumped soil mass and not from the intersection of the water table and the ditchbank. The flow net diagrams for North Morrow illustrate that tile drainage eliminates one flow channel to the streambank. However, the installation of a tile drain for this watercouse was the least successful of all the four watercourses monitored as the reduction of seepage forces was calculated to be 33% less for the tiled streambank.

Figure 12a



NORTH MORROW MUNICIPAL DRAIN
WITHOUT TILE DRAIN
ELEVATION - METERS
75
74
73
72
71
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

Figure 12b

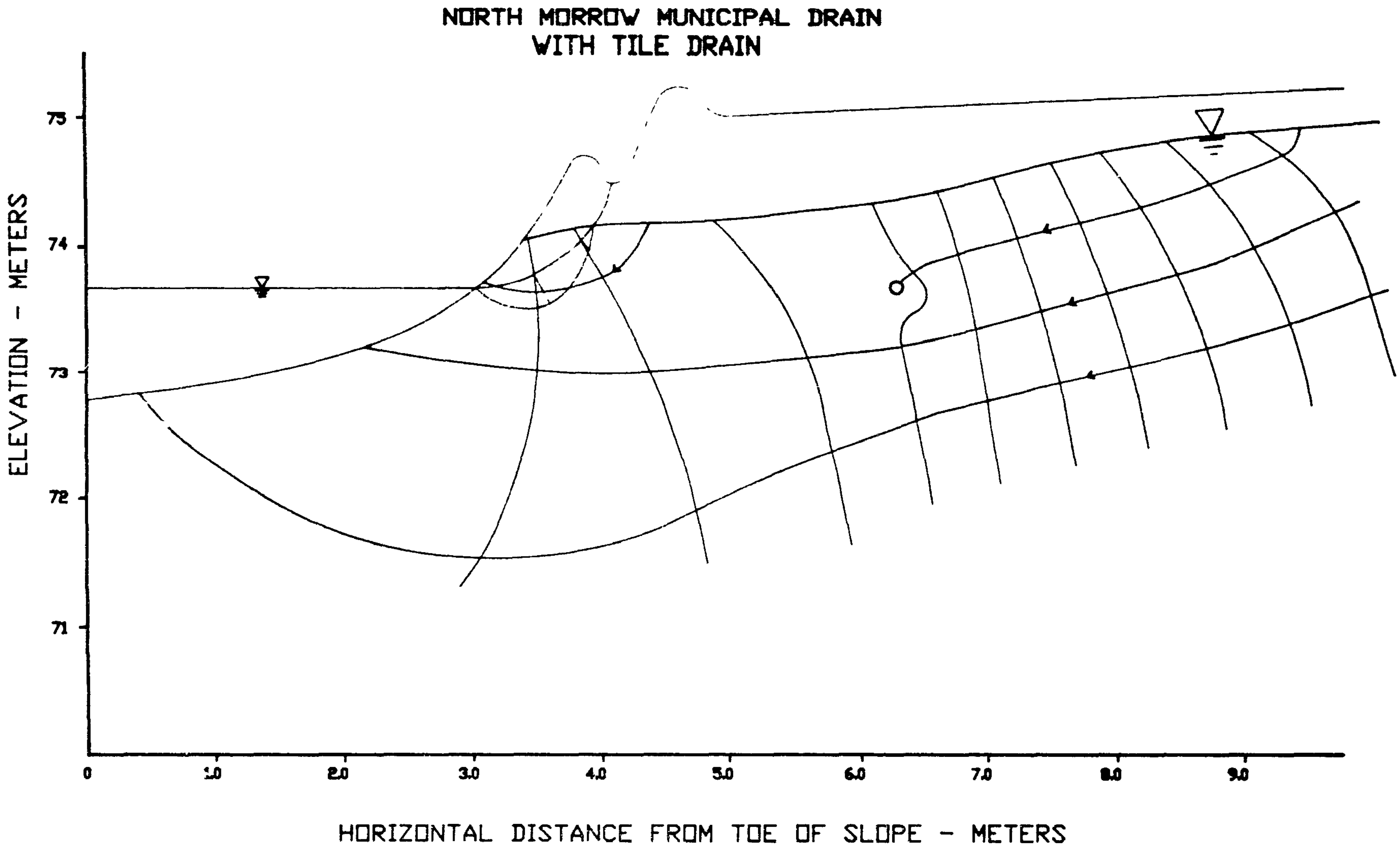

NORTH MORROW MUNICIPAL DRAIN
WITH TILE DRAIN
ELEVATION - METERS
75
74
73
72
71
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

TABLE 5

Watercourse	Seepage Pressure by Flow Net		
	Without tile drain kPa	With tile drain kPa	% reduction
North Morrow Municipal Drain	1.58	0.63	60%
Vars Storm Sewer Outlet	2.93	1.72	42%
Hammond Municipal Drain	3.79	2.29	40%
North Morrow Municipal Drain	2.83	1.90	33%

4.2 Slip Circle Analysis

The previous section involved the calculation of seepage forces through the development of flow nets. In this section, a more detailed analysis using the method of slices can calculate a safety factor based on parameters such as weight of soil, water pressure and the soil's cohesion and internal angle of friction.

The North Morrow and Hammond Municipal drains were both selected for sample calculations as the former demonstrates typical slab failure representative of cohesional soils and the latter, localized sloughing indicative of frictional soils.

For worst case conditions, when the ditchbank is most prone to failure, the soil strength's parameters are minimal. The term, worst case condition, is used in this context as the actual physical measurements obtained on site during prolonged wetness which revealed soil strength parameters at their lowest. It is quite likely that when on site measurements were performed, the streambank may not have been at its weakest. From Table 6, it can been seen that for the cohesional soil, the weakest state which was observed with the axial shear graph for the North Morrow was determined to have a shearing force of 10 kPa with a corresponding internal angle of friction of 8° at a moisture content of 38%. The Hammond drain, a frictional soil, the worst case condition was determined to have a shearing force of 3 kPa with a corresponding internal angle of friction of 30^0 at a moisture content of 25%.

For the North Morrow, the internal angle of friction could be interpreted as negligible and the Circle Moment Method chosen to calculate the streambank's effective safety factor. However, to properly gauge the effectiveness of the interceptor tile reducing the streambank's water table and seepage forces, the Method of Slices is used as the method of calculation.

To determine the effectiveness of a tile drain under worst case conditions a safety factor is calculated for both an untiled and tiled cross-section. The same slip circle is used for the purpose of calculating a safety factor as was used to calculate the seepage force by flow net. In both cases, they represent the most likely regions along the streambank where failure would occur, should the soil's strength parameters be at their weakest. Using equation 2 developed in section 2.5.2, a safety factor is calculated by summing the resisting moment of each slice and dividing by the summation of the driving moment.

$$S.F. = \frac{\Sigma M_r}{\Sigma M_d} = \frac{\Sigma}{\Sigma} \frac{C'L + (W\cos a - uL) \tan \phi'}{(W \sin a)}$$

As expected the driving moment will be greater for the untiled section due to the larger amount of water in the streambank, as indicated in Tables F-1 and F-2 in Appendix "F".

The tile drain reduced the streambank weight for the North Morrow drain by 0.335 Kg-m and the Hammond by 0.383 Kg-m. This difference in weight is quite small and would appear insignificant when substituted in the driving moment formula in equation 2. However, this reduction in water content increases the unsaturated zone in the overall streambank and as a result, increases the shearing resistance in a cohesional soil, and reduces the pore pressure in a frictional soil.

In particular, the increase in stability for the North Morrow is greatly attributed to the increase of cohesion in slice #5, as shown in Figures 12a and 12b. This slice has now become unsaturated, resulting in an increase in the shearing strength along the slip line from 10 kPa to 18 kPa. To some degree, the pore pressure has been reduced but is relatively insignificant when compared to the shearing force in a cohesional soil. Here it can been seen why in most cohesional soils when calculating a factor of safety under saturated conditions, the term " (Wcos a - uL) tan ϕ'" of the resisting moment becomes negligible for small internal angles of friction, when compared to C'L. In this instance, the much simpler form of calculating a factor of safety using the Circle Moment Method as shown by equation 3 and described in Section 2.5.3., may be applied.

Alternatively, as shown in Figures 14a and 14b, for a frictional soil such as the Hammond drain, the shearing force is relatively small, consequently a reduction in pore pressure has a much larger effect on the overall stability equation. It can be seen that the resisting moment is increased as the internal angle of friction is larger. Accordingly, the term " (Wcos a - uL) tan ϕ' " becomes much more significant when compared to " C'L ".

Appendix "F" outlines the detailed calculations to determine the safety factor using equation 2. Table F-1 illustrates an increase of 33% in stability for the North Morrow between a tiled and untiled streambank. Table F-2 shows an increase of 28% in stability after the installation of an interceptor tile in the Hammond.

NORTH MORROW M.D. METHOD OF SLICES
WITHOUT TILE DRAIN

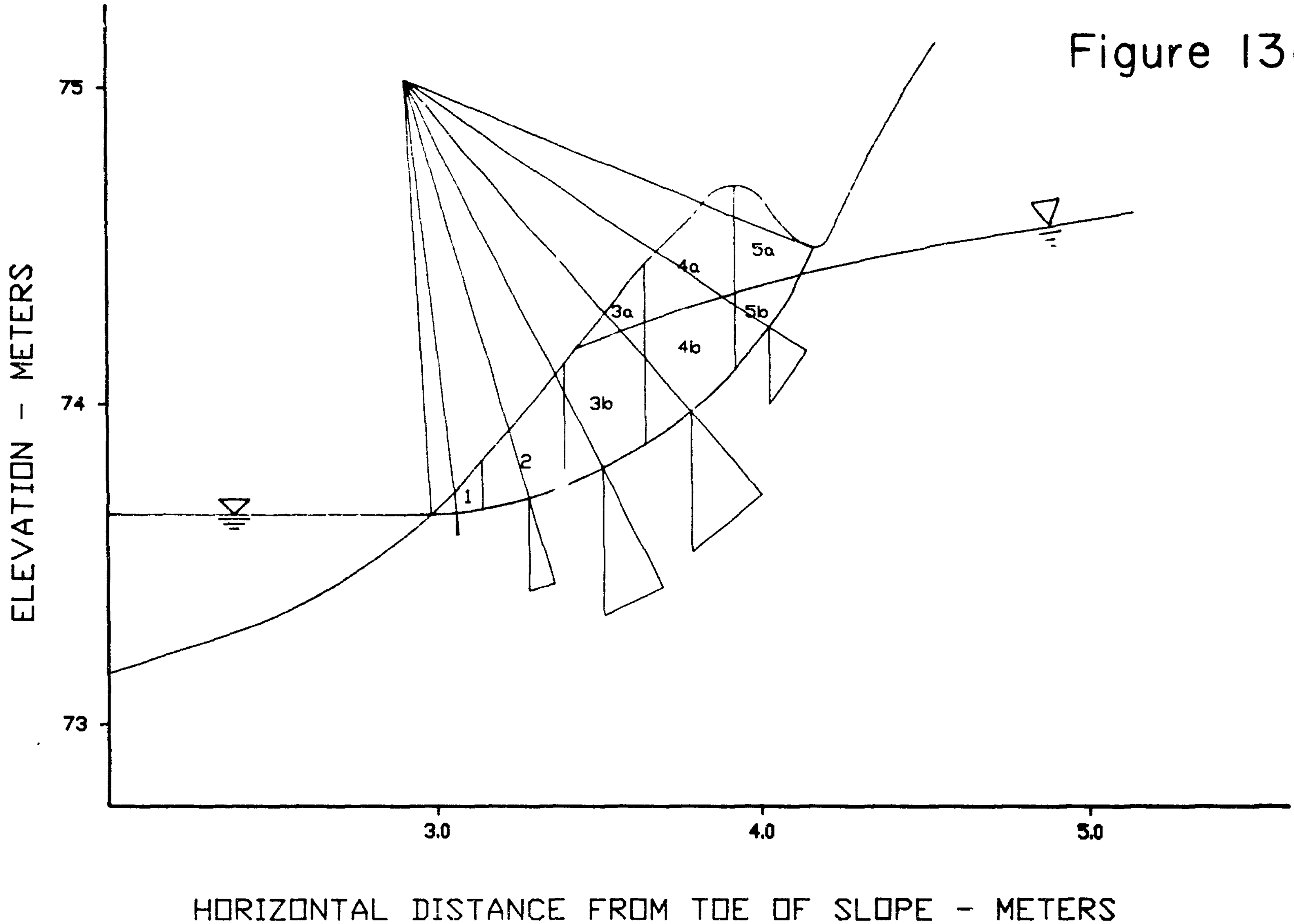
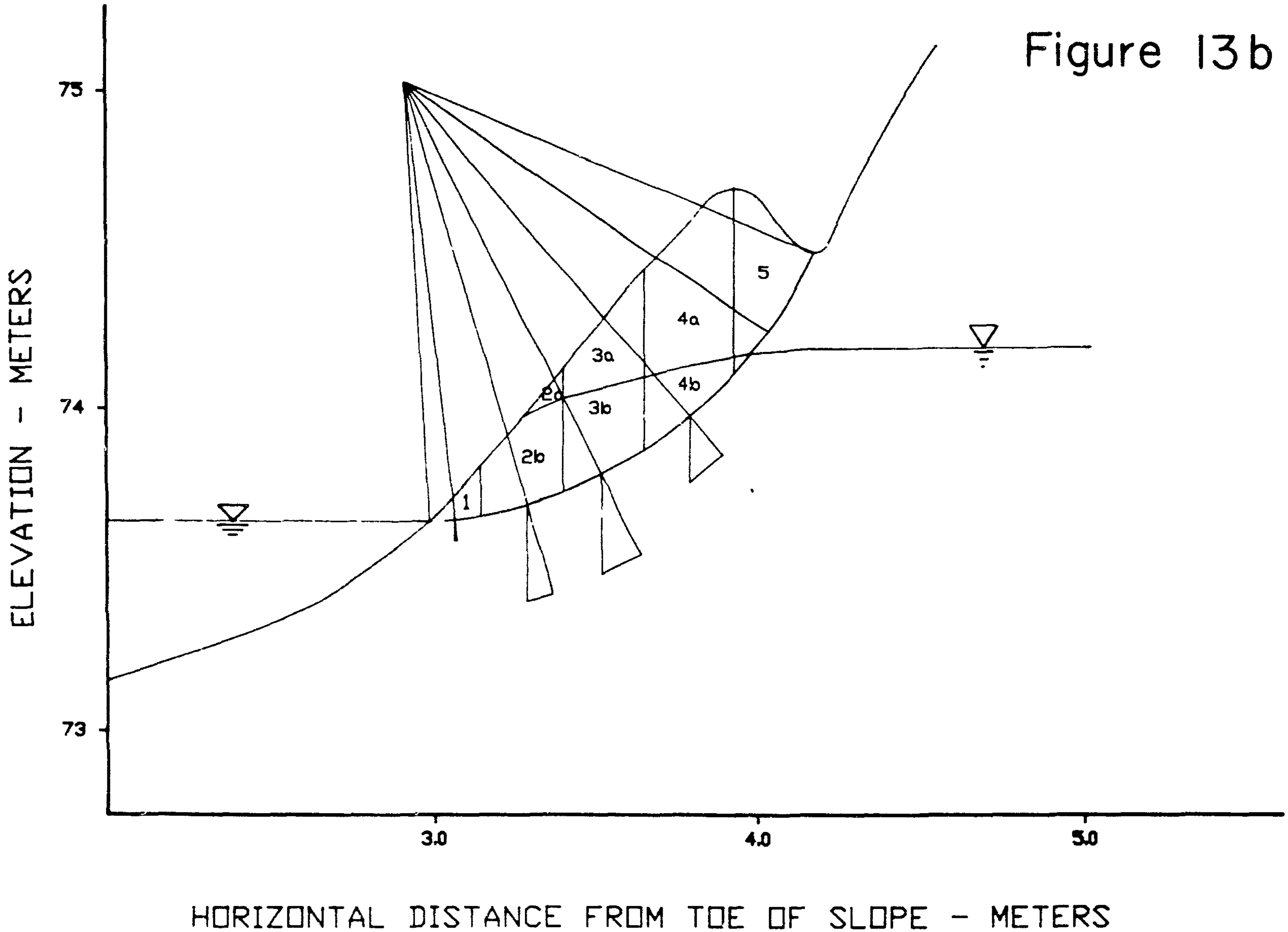
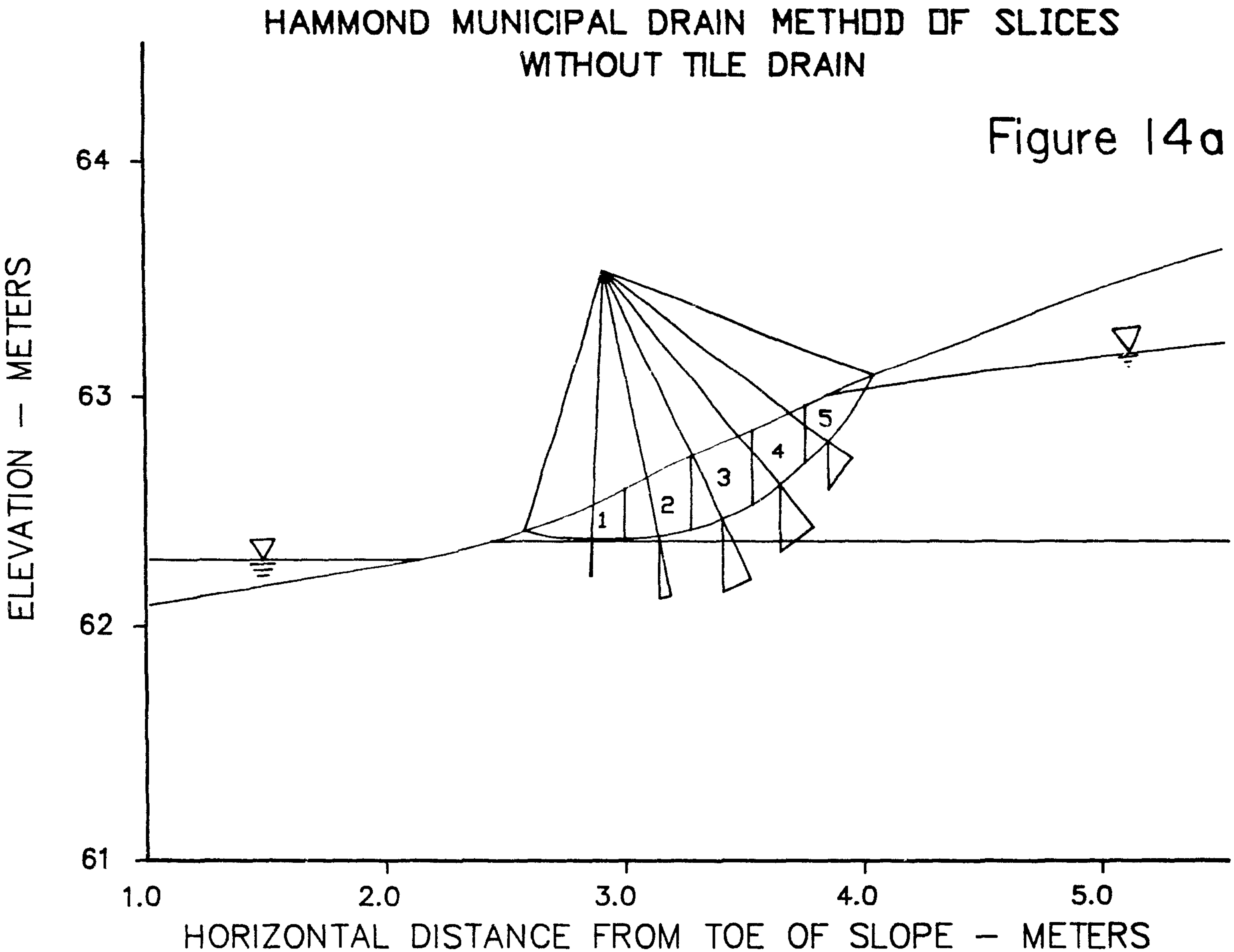


Figure 13a

NORTH MORROW M.D. METHOD OF SLICES
WITH TILE DRAIN

Figure 13b




HAMMOND MUNICIPAL DRAIN METHOD OF SLICES
WITHOUT TILE DRAIN
Figure 14a
64
63
62
61
ELEVATION - METERS
1
2
3
4
5
1.0
2.0
3.0
4.0
5.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

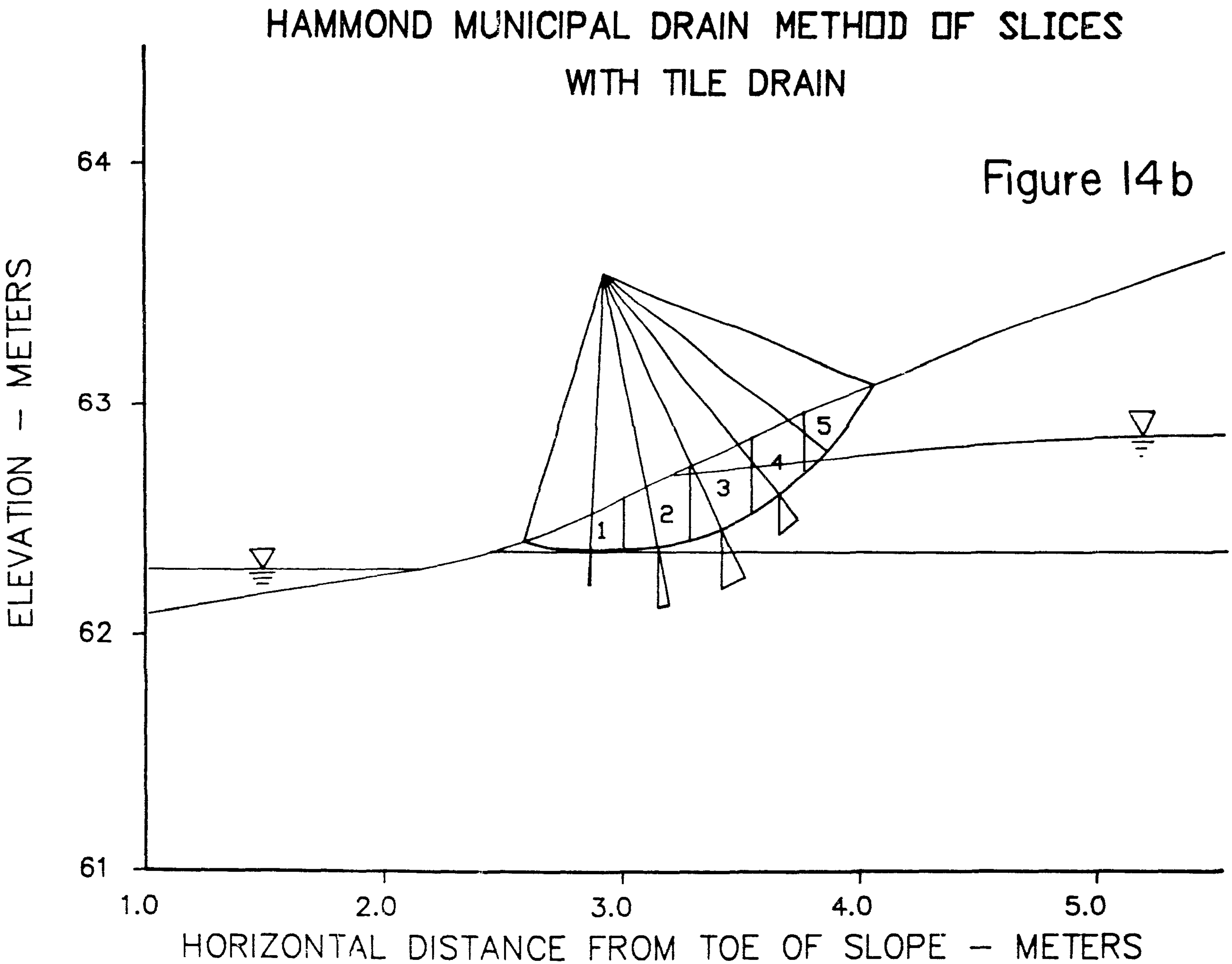

HAMMOND MUNICIPAL DRAIN METHOD OF SLICES
WITH TILE DRAIN
Figure 14b
ELEVATION – METERS
64
63
62
61
1.0
2.0
3.0
4.0
5.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE – METERS
1
2
3
4
5

TABLE 6

SELECTED DATA FROM

AXIAL SHEAR GRAPH

TEST LOCATION & SOIL TYPE	MOISTURE CONTENT (%dw)	COHESIVE STRENGTH	INTERNAL ANGLE OF FRICTION	PLASTICITY INDEX
NORTH MORROW	10%	42 kPa	20°	30-41
	25%	36 kPa	16°	29-40
VARVED CLAY	33%	18 kPa	13°	32-41
	38%	10 kPa	8°	29-40
LOWER YORK	15%	35 kPa	30°	24-31
	22%	28 kPa	20°	24-30
SILTY LOAM	25%	21 kPa	21°	24-32
	33%	10 kPa	15°	22-30
HAMMOND	25%	3 kPa	30°	--
FINE SAND	23%	5 kPa	30°	--
	20%	6 kPa	30°	--
	15%	8 kPa	34°	--

4.3 Vegetal Lining Results

The stages of development for a good vegetal cover are generally recognized as planting and germination, growth, development, and maturity. The time required to complete this cycle greatly depends on the species and growth environment.

Given the proper moisture conditions and nutrient availability in the soil, at least two growing seasons are required to establish a dense vegetal lining. The sample mixtures were applied during spring 1984 and were evaluated in the fall of 1984. Many factors influence the selection of vegetal linings. An evaluation of uniformity and density of cover, maintenance of stand, climate and antecedent moisture conditions will assist in the choice of mixture. Forage crops used for erosion control may be categorized into two groups; sod-forming grasses and bunchgrasses. Sod-forming grasses, also known as turf grasses develop a dense root system and spread rapidly by shooting off new rooting systems from the surface plant material. They usually develop a short vegetal cover with a thick but short rooting system; the bunchgrasses develop a deep but thin root system. Instead of the spreading characteristics of the sod-forming grasses, the surface plant material shoots and branches upward.

The selection of the various types of grasses is limited due to climate. Some of the more ideal and highly acclaimed grasses for erosion control measures such as Bermudagrass which, as a fast-spreading, sod-forming grass that produces more wear-resistant turf than any other, is suited only for warm season growing. The combination of the species which form the test mixtures, are all proven cool season resistant varieties.

The Trepp method (1950) was used to evaluate degree of cover provided by each mixture at each specific site and is shown in Appendix "E". Table 7 describes each specie and their characteristics which make them attractive components in an erosion control forage mixture. It is the combination of these which provide the make-up of the test mixtures. Table 8 describes the outcome of each of the seed mixtures.

Seeding operations were conducted in late May following the spring freshet. The high moisture condition allowed the bunchgrasses, like Timothy and Annual Ryegrass, to establish rapidly and develop a base for the sod-forming grasses which generally took longer to germinate. The best conclusions can be made from the North Morrow and Hammond Municipal Drain. Both mixtures "C" and "A" exhibited better growth characteristics than the conventional mixture "D".

The Hammond drain provided ideal conditions for good vegetal growth; a sandy surface and flat side slopes. The most prominent specie of this mixture was the clover which developed a solid root system and lush vegetal cover. The clovers are known to develop a dense rooting system but require longer time to germinate. The rapid germination of the Timothy and the Annual Rye grass provided this base. Other noticeable species in this mixture were the Kentucky Bluegrass and Creeping Red Fescue which exhibited good growth characteristics but, like the clovers, require a nurse crop to aid in establishment. The adjacent side slope seeded with mixture "D" exhibited poor growth and cover. The North Morrow drain revealed mixture "C" to be the most productive. The clovers were observed to be the predominant specie developed from mixture "A" but was not as lush as the Birdsfoot Trefoil which had established very well further downstream as part of mixture "C". Since both were seeded at the same time, the best explanation for a better growth of mixture "C", may be that Birdsfoot Trefoil grows well in poorly drained soils.

Mixture "D" was notably successful on the Millette drain where with an increase of concentration to 70 kg/ha, a reasonably good growth took place. However, in addition to the sandy soil and flat side slopes which make for good growth, ideal conditions and gentle precipitation periods kept the soil moisture conditions favorable.

The seeding of the Labreche drain was the least successful. The sideslopes are quite steep making establishment of the seed quite difficult. It was observed when applying the seed to the bank that the Birdsfoot Trefoil, due to the spherical nature of the seed, had a tendency to roll down the ditchbank into the bottom of the drain. Most of the seed had germinated and was developing a good vegetal cover at the bottom of the drain on the flatter sideslopes.

Despite the success of mixture "C", it is felt that it could be improved further with the addition of a nurse crop. Table 7 indicates a germination period of up to four weeks for both Red Fescue and Birdsfoot Trefoil. With the inclusion of an annual grass as a starter, the mixture should be the ideal combination for establishing a good vegetal cover.

Except for the Millette drain, all test mixtures were seeded at the rate of 60 kg/ha. It is felt that this increase in concentration of approximately 15% from current recommendations is necessary to cover seed lost through application and natural erosion processes during early seed growth periods.

TABLE 7

	Char. Class.	Growth Habit	Germination Time (days)	Drainage Class Dry	Drainage Class Well Drained	Drainage Class Poorly Drained	Root System	Vegetal Lining	Remarks
Kentucky Bluegrass	S	P	21-28		X	X	Dense but shallow	Lush long blades	Creeps and fills where others thin out.
Creeping Red Fescue	S	P	21-28	X	X	X	Dense but shallow	Thick fine leafy growth	Requires well drained soils. Produces complete cover in one year.
Annual Rye Grass	B	A	7		X	X	Deep but thin	Long narrow leaves extend from base of plant	Excellent starter used as nurse crop.
Timothy	B	A	7	X	X	X	Deep but thin	Long thin strands	Excellent starter uses as nurse crop. Grows in widest range of environments.
Birdsfoot Trefoil	S	P	14-21		X	X	Dense but shallow	Forms dense flowery surface cover	Grows well on poorly drained soils but not very drought resistant. Expensive.
White Clover	S	P	7-14		X	X	Dense but shallow	Flat leafy structure spreads on surface	Like Alsike requires a nurse crop to give an early maturing mixture.
Alsike (Clover)	S	P	7-14		X	X	Dense but shallow	Flat leafy structure spreads on surface	More tolerant in acid soils and poor drainage than White Clover but more expensive.

B - Bunchgrass A - Annual
S - Sod-forming P - Perennial

TABLE 8

SEED MIXTURE RESULTS

DRAIN	MIXTURE	VEGETAL* COVER	RESULTS
North Morrow	'A' 'C' 'D'	3-4 4-5 1	Mixture 'C' exhibited excellent growth. Some flowering observed in a lush combination mixture Birdsfoot Trefoil and Creeping Red Fescue. Only scattered clumps of vegetation were produced from Mixture 'D'
Lower York	'C' 'E' 'A'	2-3 3 3-4	Most of the growth was found to be at the bottom of the ditch where side slopes were flatter. Some of the annual grasses established at the top of the drain.
Hammond	'D' 'A'	1 5	Mixture 'A' exhibited excellent growth. Clover spread rapidly, had good root system.
Labreche	'C' 'D'	1 +	Sideslopes are 1.5h:1v making seed establishment very difficult. Most of the seed was lost.
Cumming	'B' 'A' 'C'	2 2 2	Most of the seed was lost following a rainstorm after the seeding. The Clovers and Birdsfoot Trefoil show some signs of spreading.
Millette	'D'	4	In addition to sandy soils and flat sideslopes, ideal conditions prevailed following seeding operations.

*Estimation by degrees of cover. See Appendix "E"

CHAPTER V

CONCLUSION

5.0 Summary

To determine if a streambank will be unstable, an on-site examination of the soil should be conducted. It involves a bore hole test to determine the soil profile up to 1m below the proposed ditch bottom, and possibly an auger hole test to determine the hydraulic conductivity. Soil maps have been proven to be inadequate for the drainage engineer as they do not identify local soil conditions nor do they describe the soil conditions beyond 1 meter below the soil's surface.

Soils which have been identified as unstable are the fine sands and the layered soils. In sandy soils, horizontal permeabilities are very high and result in a pressure head along the interface, especially after rainy periods. The sand transmits the water more freely and water flows along the interface to the ditchbank where piping or washout occurs.

5.1 Interceptor tile drain

The results of this study have proven that the installation of subsurface drains parallel to open channel watercourses have been successful in reducing the incidence of streambank failure.

The parallel subsurface drains are installed to intercept water which in turn reduces seepage forces that tend to move soil towards the toe of the slope as water flows towards the ditch. Bank stability is greatly reduced during snowmelt when banks are prone to failure due to extended periods of high water table. The removal of excess water behind the ditchbank, in effect increases the streambank's water holding capacity during this critical period of the year. Subsurface interceptor drains may not eliminate the slumping, however, the number and severity of occurrences will be greatly reduced.

Insertion of a subsurface interceptor drain has been shown to be very effective in reducing lateral seepage forces in sandy soils as shown in both the Vars and Hammond. Installation in a varved clay such as the North Morrow has been effective in streambank stability, however, the efficiency of removing water in the streambank could be improved by constructing a granular trench above the tile. In this manner, each individual horizontal water carrying layer may be intercepted and water transported down to the subsurface drain, relieving the seepage forces at the streambank.

5.2 Vegetal Cover

In the selection of seeding mixtures, there are several factors to consider. In addition to soil and climate, the following considerations are important.

Seeding generally will be most successful if carried out from May 1st to June 15th and from August 20th to September 20th. Specifications normally recommend seeding within a 24 hour period after excavation of ditchbanks to obtain desirable moisture conditions. This guideline should not be enforced before May 1st or after October 15th. The demonstration sites were all seeded in the fall based on optimum soil conditions as the combination of a decrease in evapotranspiration, raising the water table, and the peak in soil temperature. These conditions make for a rapid seed germination. In this manner the seed will germinate enough to establish a root system to carry it through the spring runoff. In the spring, seeding should be done following the spring runoff or the seed may be washed away. Moisture conditions are normally favorable during this period and are complemented by warm weather. Should seeding be unsuccessful, re-seeding should be carried out when conditions are most favorable, that is in the summer during or following rainy periods.

Ease of establishment and time required to develop a protective cover are extremely important considerations in selecting the correct mixture and vegetal type. Annual grasses are preferred for rapid growth mixed in with the more native and hardy sod-forming grasses which require a longer period of time for establishment.

High flows in the channel may remove seed or shoots before the grasses have sufficient roots for anchoring. In the same manner, bank seepage can wash out seeds or new growth.

It is recommended that the seeding rate be no less than 60 kg/ha. Reseeding may be necessary and should be carried out at the earliest opportunity when soil conditions are favorable.

Deposition may be controlled to some extent by the selection of a vegetation. Low, shallow flows undergo very high retardance when flowing through sod covers. A dense cover keeps the flow from channeling. However, the end result of low velocities is excessive settling of the suspended sediment. The bunch grasses and open covers offer less resistance to shallow flows than sod-forming grasses, but provide less protection against erosion.

Hydro seeding is a very successful operation but due to its cost it should be reserved for highly erosive areas.

RECOMMENDATIONS FOR FUTURE STUDY

With reference to the experiments undertaken in this study, it is suggested that the following be considered to further evaluate methods to increase streambank stability.

- Due to the cost associated with the installation of subsurface filtered drainage tube, evaluate the effectiveness of installing a mole drain as an interceptor tile
- Install a depth recorder instead of watertable tube to determine the fluctuating watertable in the streambank relative to the water level in the watercourse. In this manner the maximum head differential could be calculated for actual case conditions
- The effect of ice lenses acting as impermeable layers during the Spring runoff should be evaluated
- A more thorough seeding schedule should be developed to determine various seed mixtures vis-a-vis soil type, soil conditions, local climate, application and seed costs
- Laboratory tests should be conducted to confirm isotropy in the study soils to properly evaluate subsurface flow by flow net

REFERENCES

BOWIE, A.J. Investigations of Vegetation for Stabilizing Eroding Streambanks, accepted for publication as Paper No. 81-2104, TRANSACTIONS of the ASAE, 1981.

BRADFORD, J.M. and PIEST, R.F. Gully Wall Stability in Loess-Derived Alluvium, Soil Science Society of American Journal, 41, No.1, pp.115-122, 1977.

BRADFORD, J.M. AND PIEST, R.F. Erosional Development of Valley-Bottom Gullies in the Upper Midwestern United States, In, Thresholds in Geomorphology, D.R. Coates adn J.D. Vitak (Editors), pp.75-101, 1980.

BROWN, B. Methods of Surveying and Measuring Vegetation, Commonwealth Bureau of Pastures and Field Crops, Chapter 7, Bulletin 42, 1954.

CALDWELL, L.W. Alternatives in the Design and Maintenance of Stable Channels, accepted for publication as Paper No. 81-2102, TRANSACTIONS of the ASAE, 1981.

CEDERGEN, H.R. Seepage, Drainage, and Flow Nets, New York, John Wiley & Sons, 489 pp, 1968.

GIBBONS, J.S. Design Manual for Open Channel Drainage in the Ottawa-St.Lawrence Lowlands, Agriculture Canada, 62pp, 1982.

GRISSINGER, E.H., LITTLE, W.C., and MURPHEY, J.B. Erodibility of Streambank materials of Low Cohesion, TRANSACTIONS of the ASAE, pp.524-630, 1980.

HOLTZ, R.D. and KOVACS, W.D. An Introduction to Geotechnical Engineering, Englewood Cliffs, New Jersey, Prentice Hall, 733pp, 1981.

LAMBE, W.T. Soil Testing for Engineers, New York, John Wiley & Sons, pp.22-42, 1951.

LITTLE, W.C., PIEST, R.F. and ROBINSON, A.R. SEA research program for channel stability and gully contral, accepted for publication as Paper No. SW-319, TRANSACTIONS of the ASAE, 1980.

LITTLE, W.C., THORNE, C.R. and MURPHEY, J.B. Mass Bank Failure Analysis of Selected Yazoo Basin Streams, TRANSACTIONS of the ASAE, pp.1321-1328, 1982.

LOHNES, R.A. and HANDY, R.L. Slope Angles in Friable Loess, Journal of Geology, 76, pp.247-258, 1968.

LUTTEN, R.J. Fractures and Failure Mechanics in Loess and Applications to Rock Mechanics, Report S-69-1, U.S. Army Engineers, WES, Vicksburg, Mississippi, 53pp, 1969.

LUTTEN, R.J. Uses of Loess Soil for Modeling Rock Mechanics, Miscellaneous Report S-74-28, U.S. Army Engineers, WES, Vicksburg, Mississippi, 163pp, 1974.

MANBEIAN, TAGHI The influence of Soil Moisture Suction, Cyclic Wetting and Drying, and Plant Roots on the Shear Strength of a Cohesive Soil, Ph.D. Thesis. University of California, Berkeley, pp.207-215, 1972.

MTC, Guide for Soil Field Inspectors, Highway Engineering Division, Downsview, Ontario, 1977.

OMAF, 1984 Field Crop Recommendations, Publication 296, 1984.

PALMER, L.A., THOMPSON, J.B. and YEOMANS, C.M. The Control of a Landslide by Subsurface Drainage, Highway Research Board Proceedings, Vol. 40, pp.503-508, 1950.

RAGHAVAN, McKYES, BEAULIEU, MARINEAU and AMIR Traction and Compaction in Eastern Canada, Volume II, Engineering Research Services for Agriculture Canada, 1975.

RAGHAVAN, G.S.V., McKYES, E. and CHASSE, M. Prediction Techniques for Traction Using Field and Laboratory Data, TRANSACTION of the ASAE, pp.405-408, 1976.

RICE, R.M., CORBETT, E.S. and BAILEY, R.G. Soil Slopes Related to Vegetation, Topography and Soil in Southern California, Water Resources Research, pp.647-659, 1969.

SOWERS, G.B. and SOWERS, G.F. Introductory Soil Mechanics and Foundations, New York, Macmillan, 556pp, 1948.

TAYLOR, D.W. Fundamentals of Soil Mechanics, New York, John Wiley & Sons, 700pp, 1963.

TERZAGHI, I.K. and PECK, R.B. Soil Mechanics and Engineering Practice, New York, John Wiley & Sons, 566pp, 1948.

THORNE, C.R., MURPHEY, J.B., and LITTLE, W.C. Bank Stability and Bank Material properties in the Bluffline Streams of Northwest Mississippi, Appendix D, Report prepared for U.S. Army Corps of Engineers, Vicksburg District, on Stream Channel Stability, 258pp, 1981.

TURNBULL, W.J. Utility of Loess as a Construction Material, Proceedings of the International Conference of Soil Mechanics, pp.97-103, 1978.

USDA, SCS Engineering Division, Design of Open Channels, Technical Release No.25, Washington, D.C., 261pp, 1977.

VAN BEERS, W.F.K. The Auger Hole Method, Netherlands, International Institute for Land Reclamation and Improvements, Bulletin 1, 23pp, 1979.

Appendix "A"

GENERAL WATERCOURSE DATA

Experimental Site : North Morrow Municipal Drain

Drain location

This drain is located in Concession X and XI of the Township of Osgoode, Concession XI and X of the Township of Cumberland and Conc. I, II and III of the Township of Russell in the Province of Ontario. The section considered herein extends for 200m south of the culvert bridge on the boundary road.

Drainage area up to experimental section: 382 ha

Watercourse length up to and including experimental section: 2500m

Length of experimental section: 200m

Average channel grade over experimental section: 0.1%

Soil identification

Soil map description within experimental section: Rfs

Rubicon fine sand; light grey depressions and reddish brown hummocks of sandy soils, with sorted sand parent material.

An on-site investigation of the soil reveals a varved clay with layers (1-2mm) of silt.

Adjacent topography and land use

Flat lands border each side of the drain with hay grown as the principle crop.

Channel Geometry

Initial construction of side slopes: 2h:1v trapezoidal

Condition of side slopes before maintenance: Severely eroded, continuous slumping along drain. Section developing into a U-shape. Tension cracks present and further degrees of failure evident, i.e. slab failure, resulting in loose sediment at bottom of drain.

Auger hole test

Depth From Surface	Grain Analysis	Soil Classification
0-0.1m	Top Soil	
0.1m-0.9m	41% Sand 43% Silt 16% Clay	Loam
0.9m-1.2m	57% Sand 23% Silt 20% Clay	Sandy Loam
1.2m-1.7m	15% Sand 41% Silt 44% Clay	Silty Clay Loam
1.7m+	11% Sand 39% Silt 50% Clay	Clay

Hydraulic Conductivities

k1, for hole at depth 1.5m: 0.49 m/day

k2, for hole at depth 2.4m: 0.17 m/day

Remedial Measures

-100mm filtered P.V.C. tile drain was installed on both sides of test section for approximately 150 m.

-Hand broadcast seeding of ditchbanks

Experimental Site : Hammond Municipal Drain

Drain Location

The existing Hommond Drain is located in Lots 16 and 17 of Concession VIII and IX in the Township of Clarence. The study section runs from County Road No. 21 to the Canadian Pacific Railway.

Drainage area up to experimental section: 1200 ha

Watercourse length up to and including experimental section: 4055m

Length of experimental section: 400m

Average channel grade over experimental section: 0.06% with a drop structure within study section.

Soil identification

Soil map description within experimental section: Rfs-Sfs

Rfs-Sfs combination

Rubicon fine sand; Light grey depressions and reddish brown hummocks of sandy soils with sorted sand parent materials	St. Samual fine sand; Grey mottled sandy soils with sorted non-calcareous fine sand parent material

An on-site investigation reveals 1.6m deep from surface underlain with clay.

Adjacent topography and land use

Flat lands border both sides with a high recharge area to the east, wooded area to the west and residential to the east.

Channel Geometry

Initial construction of side slopes: 2h:1v trapezoidal

Condition of side slopes before maintenance:
Considerable erosion on both sides due to piping of soil. Sediment collected at base of drain. Rip-rap of side slopes in adjacent locations has maintained original condition. Grassed sideslopes downstream have also resulted in stable sideslopes.

Side slopes after maintenance: 3h:1v trapezoidal

Auger hole test

Depth From Surface	Grain Analysis	Soil Classification
0-0.1m	Top Soil	
0.1m-0.9m	96% Sand 2% Silt 2% Clay	Fine Sand
0.9m-1.6m	70% Sand 26% Silt 20% Clay	Sandy Loam
1.6m+	12% Sand 32% Silt 56% Clay	Clay

Hydraulic Conductivities

k1, for hole at depth 1.5m: 2.5m/day

k2, for hole at depth 2.4m: 0.35m/day

Remedial Measures

-100 mm filt'red P.V.C. tile was installed along both sides of test section

rip-rap on stream bends

flatter slide slopes

hand broadcast seeds

Experimental Site : Vars Storm Sewer Outlet

Drain Location

The study section of this watercourse runs in a westerly direction west of Regional Road 33 and empties into Shaws Creek.

Drainage area up to experimental section: 20 ha

Watercourse length up to and including experimental section: 1500m

Length of experimental section: 750m

Average channel grade over experimental section: 0.3%

Soil identification

Soil map description within experimental section: Ufs Rfs Sfs

Upland fine sand; Reddish brown loose, fine sandy soil with sorted non-calcareous fine parent material	Rubicon fine sand; light grey depressions and reddish brown hummocks of sandy soil with sorted sand parent material	St. Samuel fine sand; grey mottled sandy soil with sorted non-calcareous fine sand parent material

Adjacent topography and land use

Residental land north of the open drain and a wooded area to the south. Flat lands on both sides of drain.

Channel Geometry

Initial construction of side slopes: 2h:1v

Condition of side slopes before maintenance: Newly excavated

Auger hole test

Depth From Surface	Grain Analysis	Soil Classification
0-1.0m	2% Med.Sand 82% Fine Sand 14% V.Fine Sand 2% Silt	Fine Sand
1.0m-1.8m	17% Med.Sand 74% Fine Sand 7% V.Fine Sand 2% Silt	Fine Sand

Hydraulic Conductivities

k1, for hole at depth 1.5m: 2.3m/day

k2, for hole at depth 2.4m: 2.1m/day

Remedial Measures

-100mm filtered P.V.C. tile drain installed below ditch bottom on both sides of drain

-Ditchbanks were hydroseeded

-Outlet of ditch bottom rip-rapped due to steep gradient

Experimental Site : Lower York Municipal Drain

Drain location

The Lower York Municipal Drain is located in Concession V and VI of the Township of Russell and is an outlet for the York, Eadie Snyder Municipal Drains. The section considered under this contract extends from the County Road No. 17, Lot 11, Concession VI flowing in a southerly direction to the Township Road, Concession VI.

Drainage area up to experimental section: 1400 ha

Watercourse length up to and including experimental section: 1500m

Length of experimental section: 400m

Average channel grade over experimental section: 0.04% but with two drop structures along this section.

Soil identification

Soil map description within experimental section: Cfsl Castor fine sandy loam; Dark grey, fine sandy soils with layered silt and fine sand parent materials.

On-site investigation revealed a clay lens 0.3m thick, 1.3m below surface overlain and underlain by silty loam.

Adjacent topography and land use

Land rises sharply on both sides of drain. North of drain, corn is the predominant crop and to the south general pasture land with wooded area.

Channel geometry

Initial construction of side slopes: 2h:1v

Condition of side slopes before maintenance: Cross section developed into a U-shape. Ditch slopes were exhibiting circular arc type failure. High seepage forces were evident at base of drain with localized slumping. Rill and sheet erosion were also observed. Problems of erosion and slumping occuring predominantly on south side.

Side slopes after maintenance:

North Streambank; Compound trapezoidal 3h:1v bottom
1 1/2:1v top

South Streambank; Parabolic

Auger hole test

Depth From Surface	Grain Analysis	Soil Classification
0-0.1m	Top Soil	
0.1m-1.35m	30% Sand 53% Silt 17% Clay	Silt Loam
1.35m-1.65m	14% Sand 26% Silt 60% Clay	Clay
1.65m+	33% Sand 51% Silt 16% Clay	Silt Loam

Hydraulic Conductivities

k1, for hole at depth 1.5m: 0.77m/day

k2, for hole at depth 2.4m: 1.2m/day

Remedial measures

-100mm filtered P.V.C. tile installed along southern bank for 300m

-Parabolic/Compound trapezoidal cross section constructed

-Hand broadcast seeding of ditchbanks

-2 drop structures installed along experimental section

Appendix "B"

WATER LEVEL READINGS IN THE STREAMBANK

NORTH MORROW M.D.

EVENT	WATER LEVEL MEASUREMENT TUBE 1	2	3	4	Ditch W.L.	h
1	------No readings------					
2	74.09	74.20	74.60	75.02	73.75	.34
3	74.21	74.33	74.80	75.18	73.95	.26
4	74.45	74.55	74.96	75.36	74.25	.20
5	74.32	74.40	74.80	75.20	74.00	.32
6	74.15	74.25	74.73	75.13	73.70	.45
7	74.05	74.15	74.52	74.96	73.65	.40
8	73.96	74.08	74.45	74.86	73.60	.36
9	74.10	74.22	74.55	77.94	73.75	.35

W.L. = Water Level

h = W.L. measurement tube #1 - W.L. in ditch

All readings in meters above sea level

Events recorded in April 1983

WATER LEVEL READINGS IN THE STREAMBANK

HAMMOND M.D.

EVENT	WATER LEVEL MEASUREMENT TUBE 1	2	3	Ditch W.L.	h
1	62.74	62.65	63.08	62.33	.41
2	62.70	62.63	62.91	62.31	.39
3	62.62	62.58	62.80	62.24	.38
4	62.55	62.52	62.70	62.20	.35
5	63.12	62.96	63.45	62.64	.48
6	62.84	62.68	63.25	62.24	.60
7	62.74	62.64	63.10	62.20	.54
8	62.70	62.58	62.82	62.20	.52
9	62.60	62.55	62.75	62.20	.40

W.L. = Water Level

h = W.L. Measurement tube #1 - W.L. in ditch

All readings in meters above sea level

Events recorded in April 1983

WATER LEVEL READINGS IN THE STREAMBANK

VARS S.S.

EVENT	WATER LEVEL MEASUREMENT TUBE 1	2	3	Ditch W.L.	h
1	75.35	n.r.	76.13	75.40	-.05
2	75.35	n.r.	76.12	75.40	-.05
3	75.40	n.r.	76.21	75.42	-.02
4	75.94	76.15	76.96	75.85	.30
5	75.85	76.10	76.90	75.50	.35
6	75.80	76.01	76.75	75.40	.40
7	75.45	75.80	76.30	75.40	-.05
8	75.36	n.r.	76.20	75.38	-.03
9	75.30	n.r.	76.10	75.38	-.08

n.r. = no readings

W.L. = Water Level

h = W.L. Measurement tube #1 - W.L. in ditch

All readings in meters above sea level

Events recorded in April 1983

WATER LEVEL READINGS IN THE STREAMBANK

LOWER YORK M.D.

EVENT	WATER LEVEL MEASUREMENT TUBE 1	2	3	4*	Ditch W.L	h
1	65.08	ice	65.76	n.r.	64.57	.51
2	64.94	ice	65.43	n.r.	64.50	.44
3	64.86	ice	65.20	n.r.	64.48	.38
4	64.82	64.88	65.08	n.r.	64.36	.36
5	65.25	65.48	65.98	n.r.	64.70	.45
6	65.10	65.32	65.85	n.r.	64.55	.55
7	65.03	65.20	65.60	n.r.	64.52	.51
8	64.96	65.07	65.48	n.r.	64.50	.46
9	64.92	65.01	65.41	n.r.	64.49	.43

* Watertable tube was placed in a depression and was submerged throughout trial period.

n.r. = no reading

W.L. = Water level

h = W.L. measurement tube #1 - W.L. in ditch

All readings in meters above sea level

Events recorded in April 1983

Appendix "C"

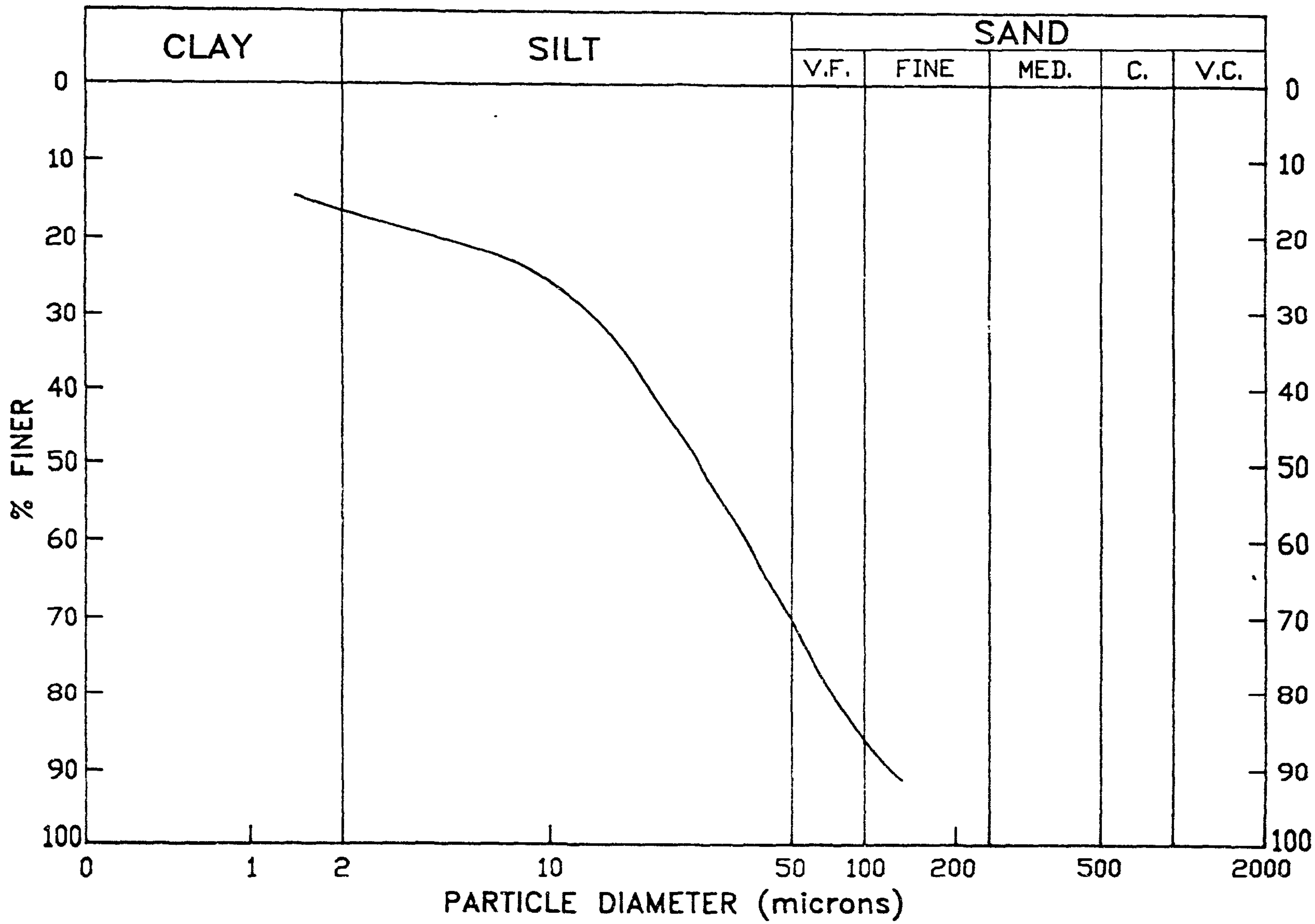


Figure C1: Lower York – Sample Depth 0.6m

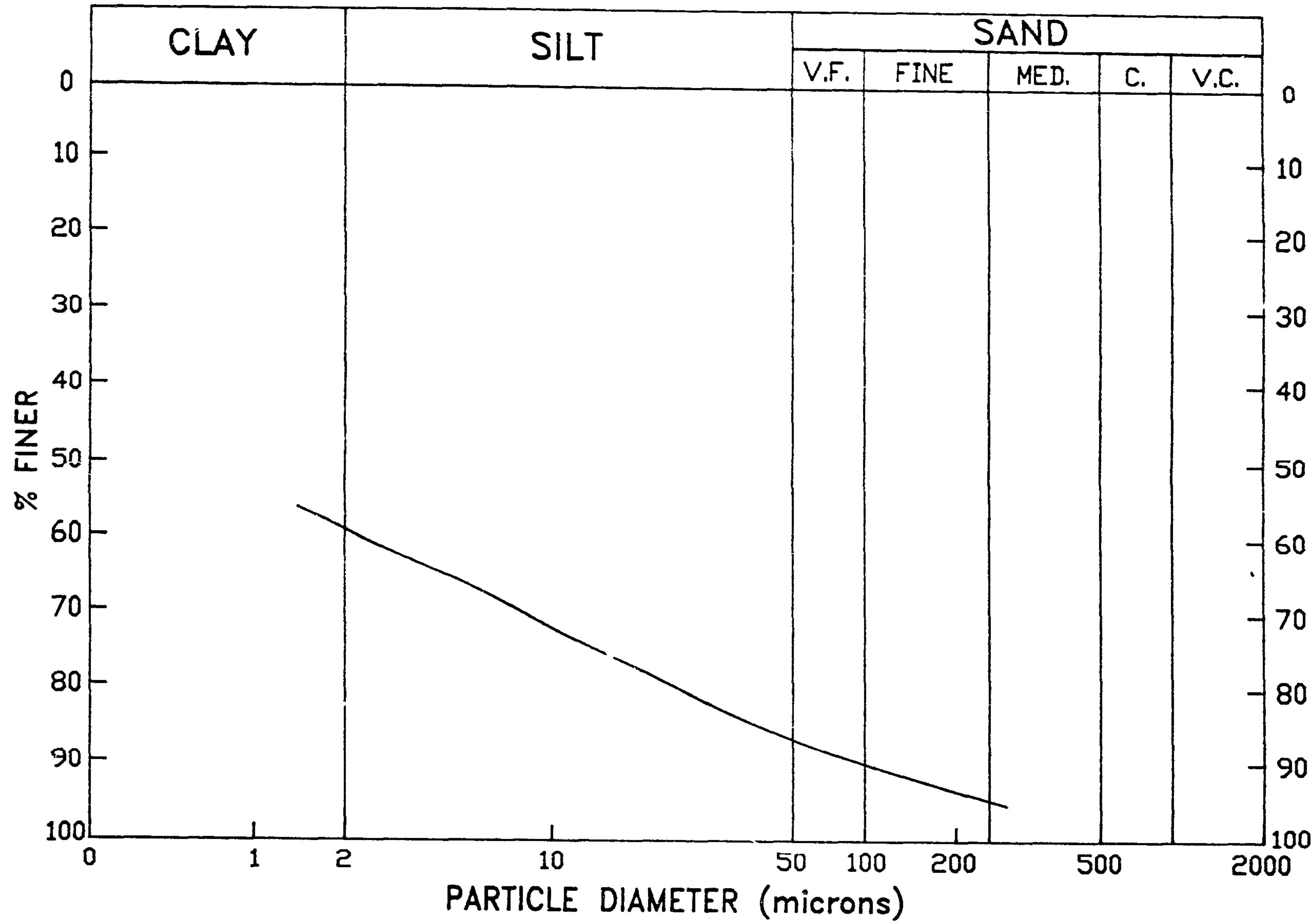


Figure C2: Lower York – Sample Depth 1.2m

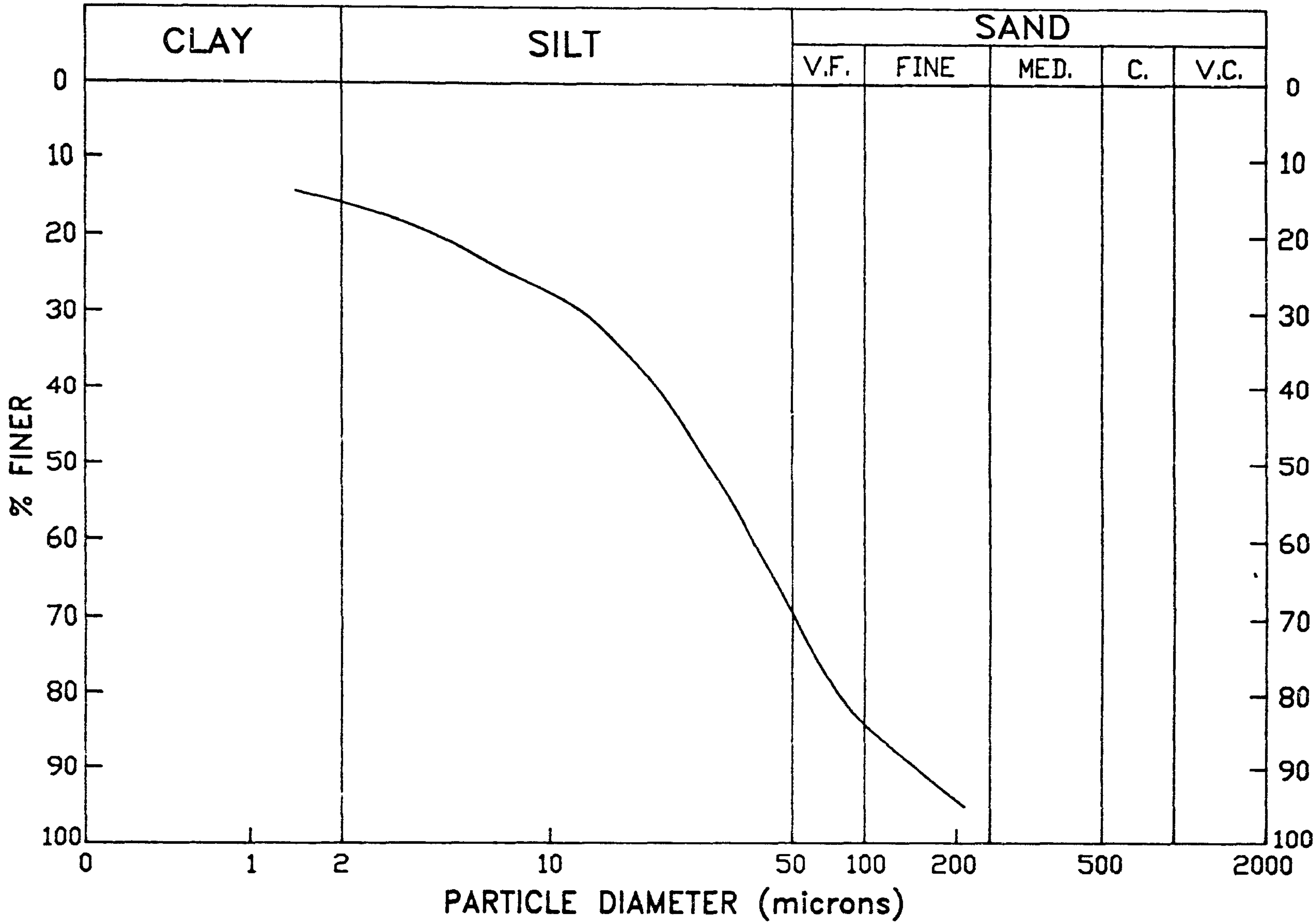


Figure C3: Lower York – Sample Depth 1.8m

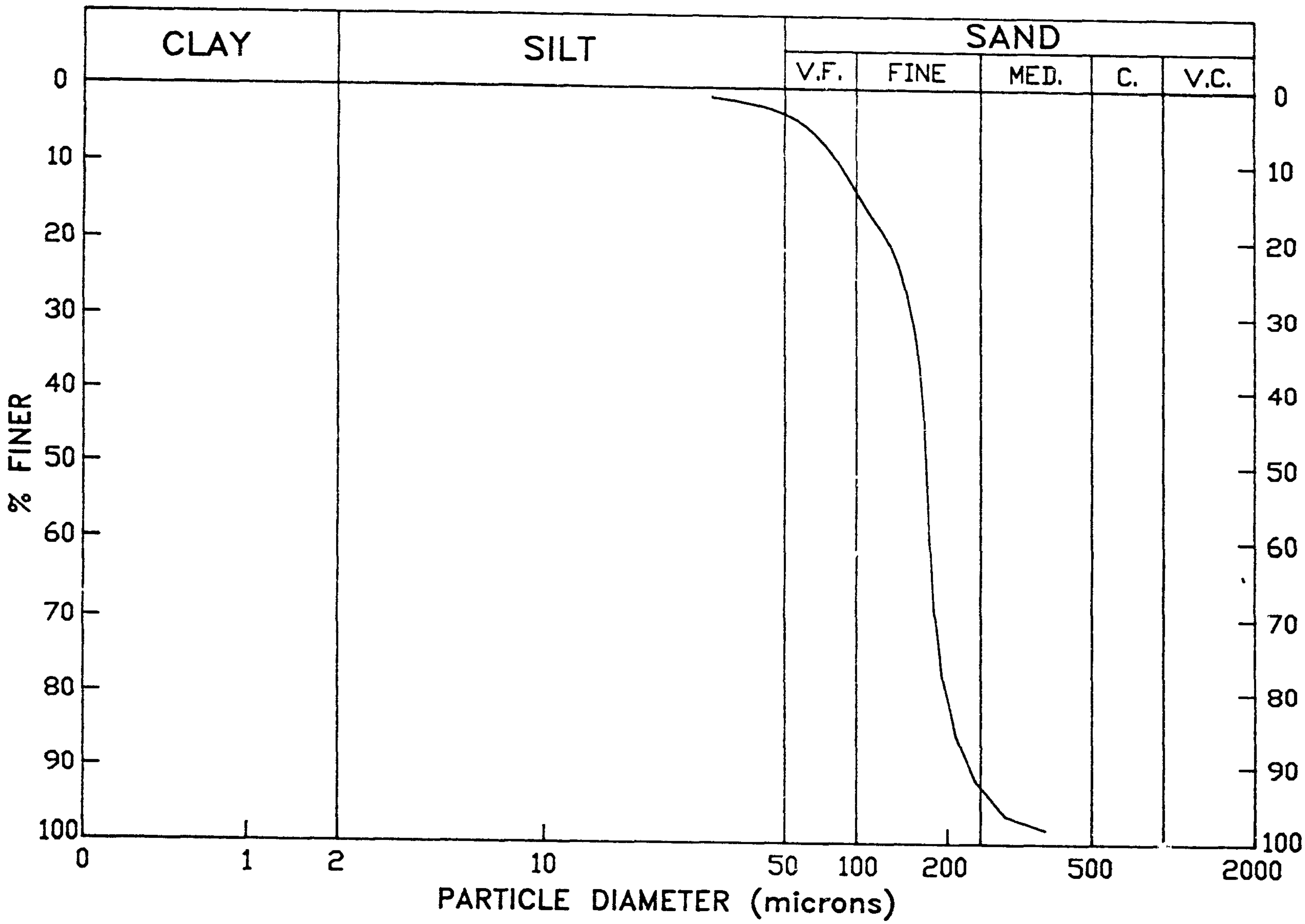


Figure C4: Vars – Sample Depth 0.6m

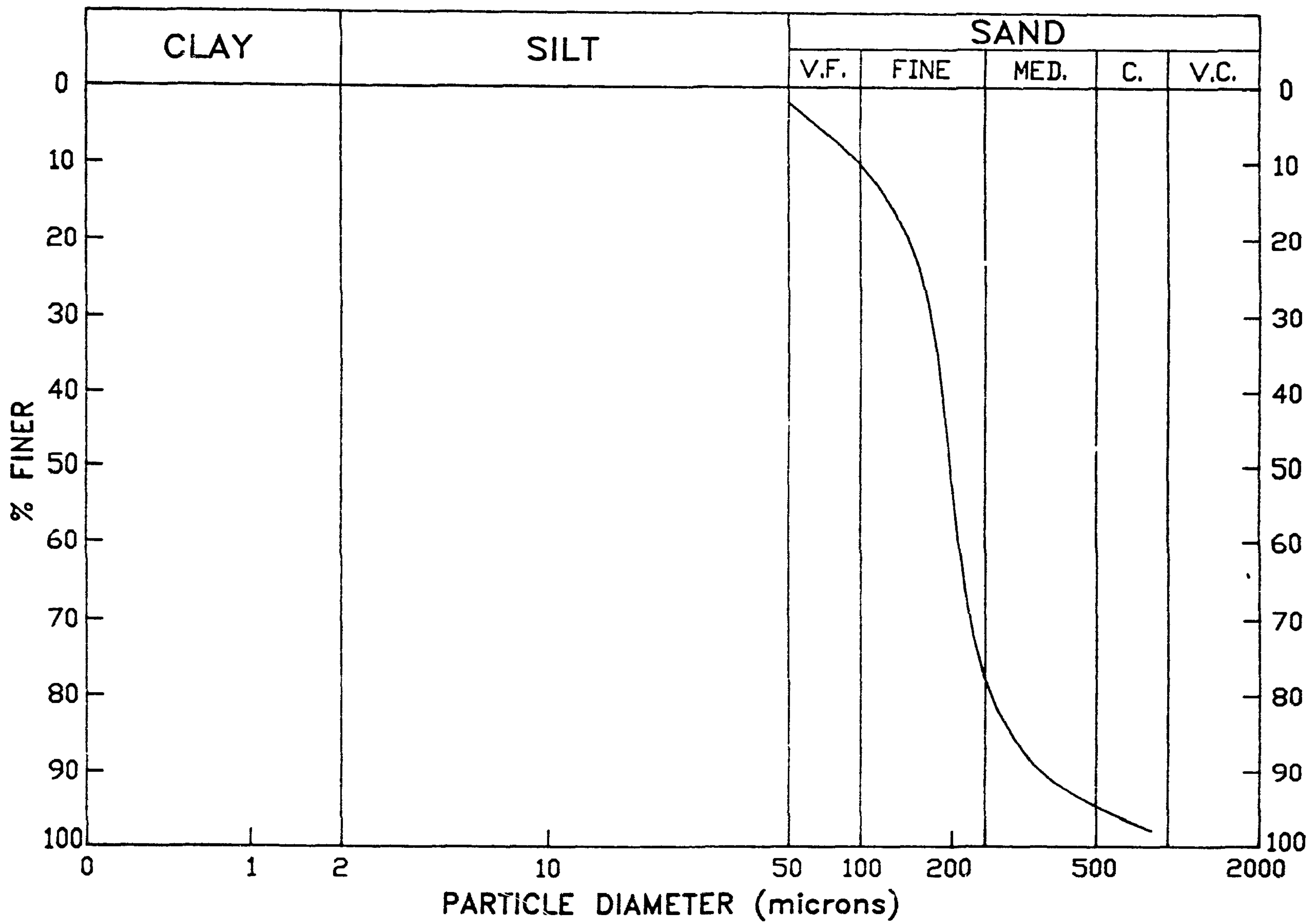


Figure C5: Vars – Sample depth 1.4m

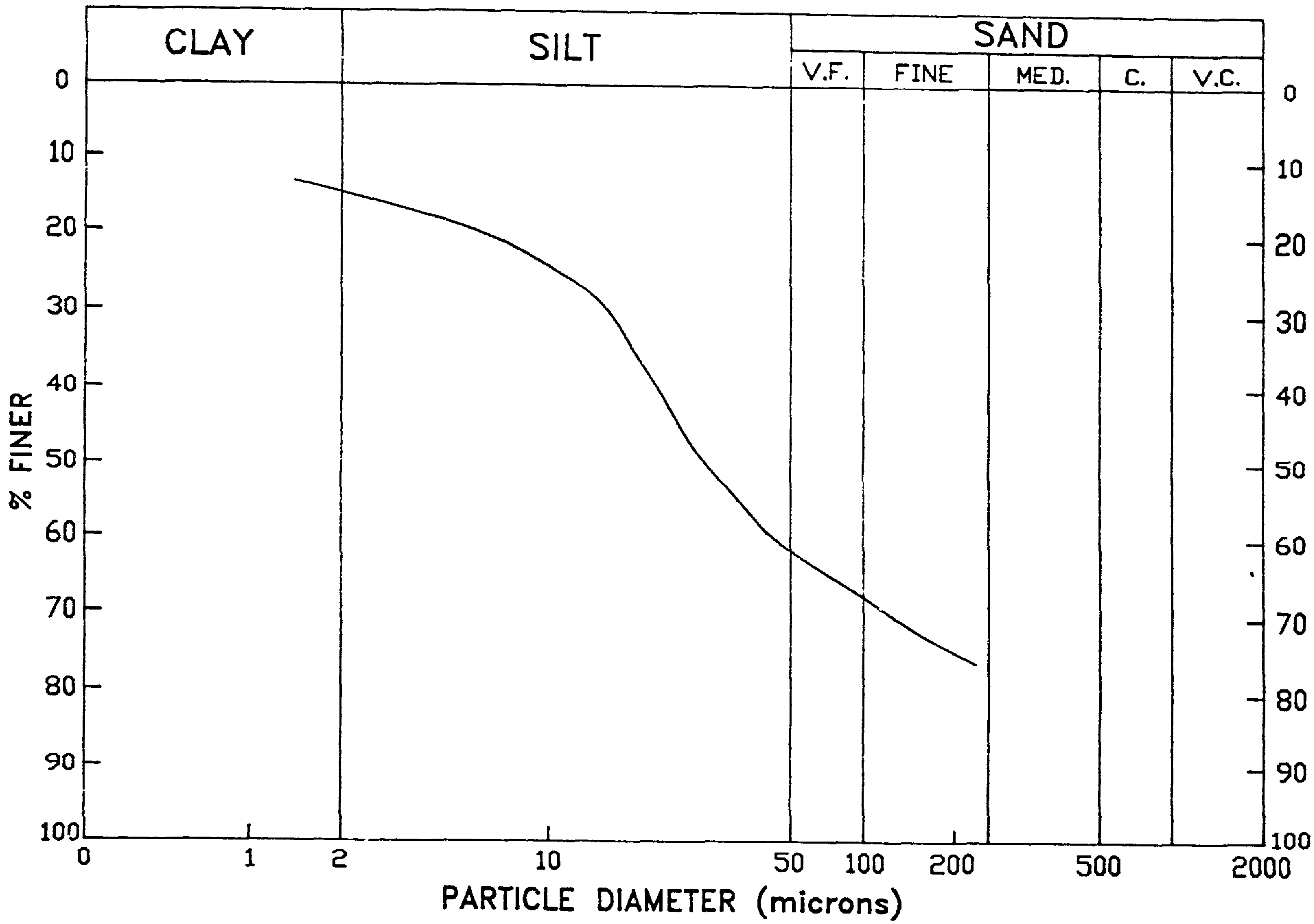


Figure C6: North Morrow – Sample Depth 0.6m

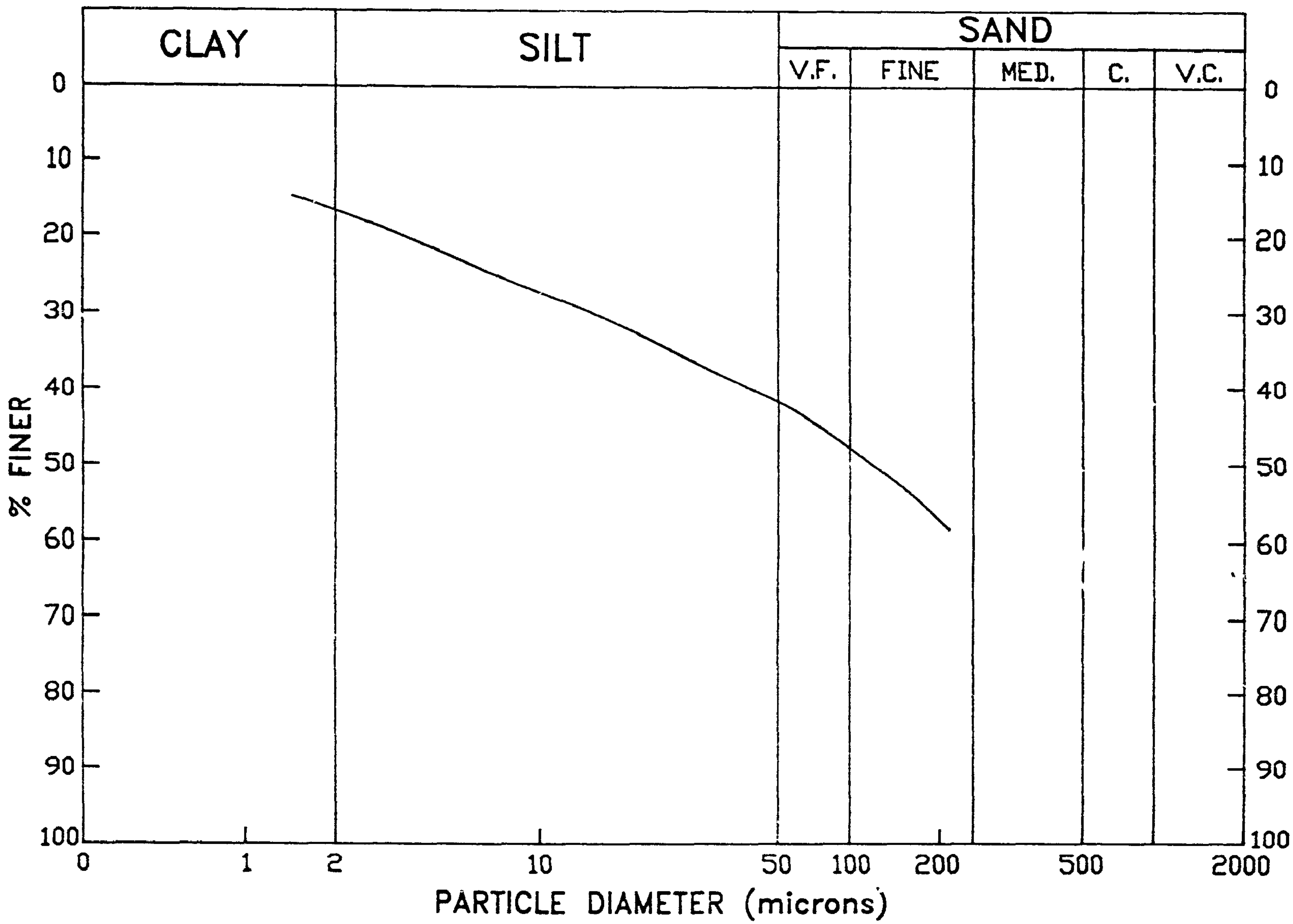


Figure C7: North Morrow – Sample Depth 1.1m

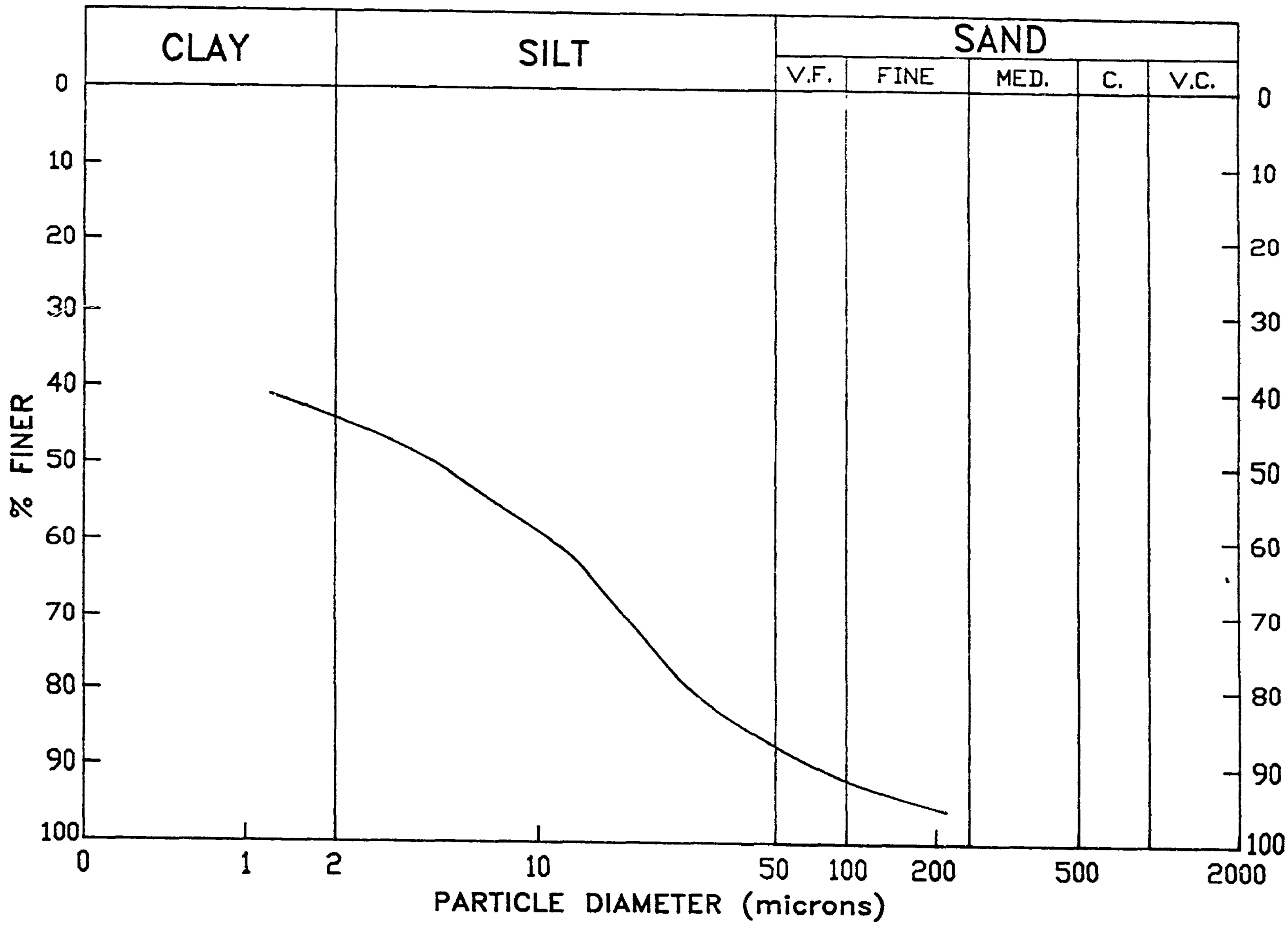


Figure C8: North Morrow – Sample Depth 1.4m

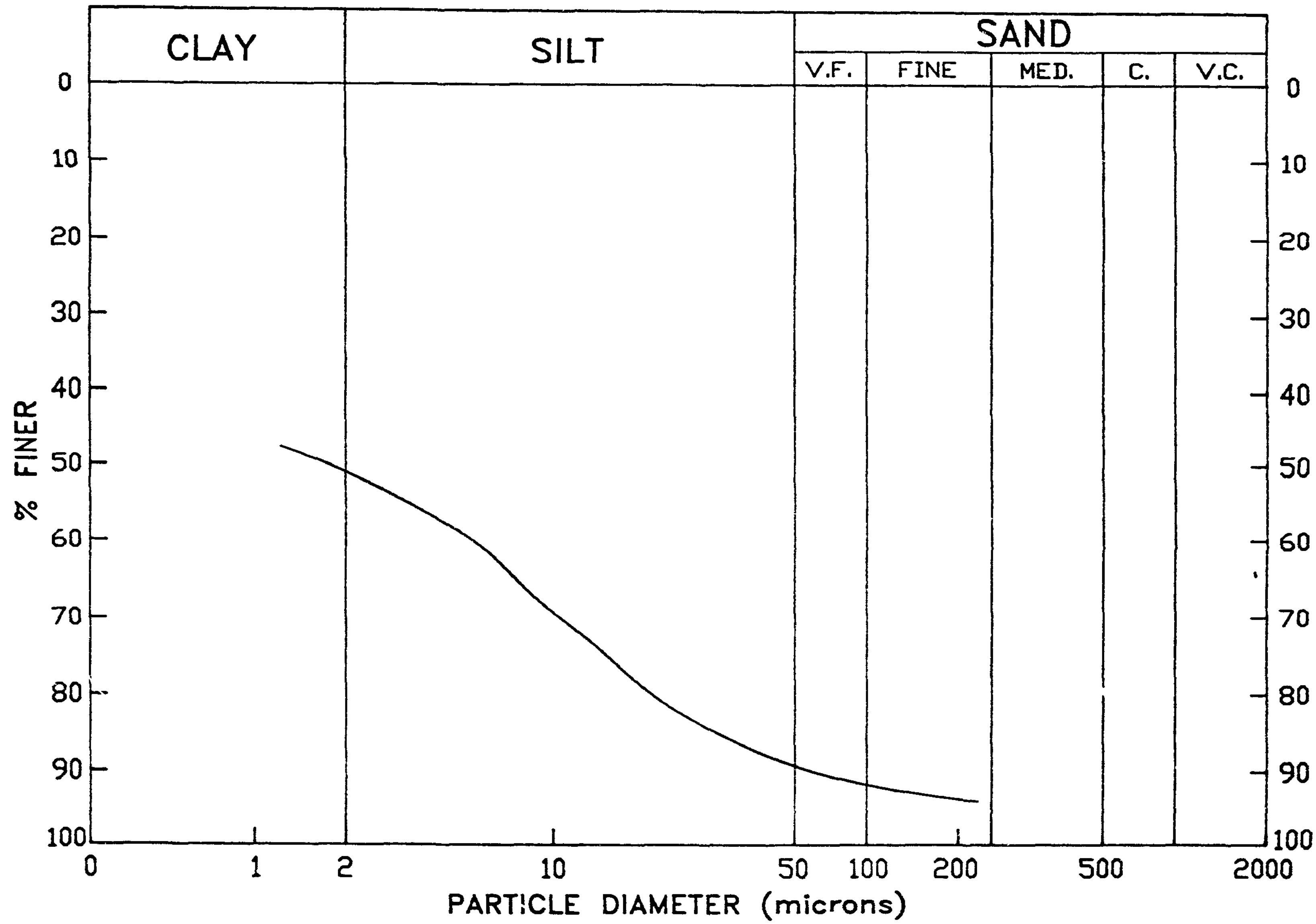


Figure C9: North Morrow – Sample Depth 1.8m

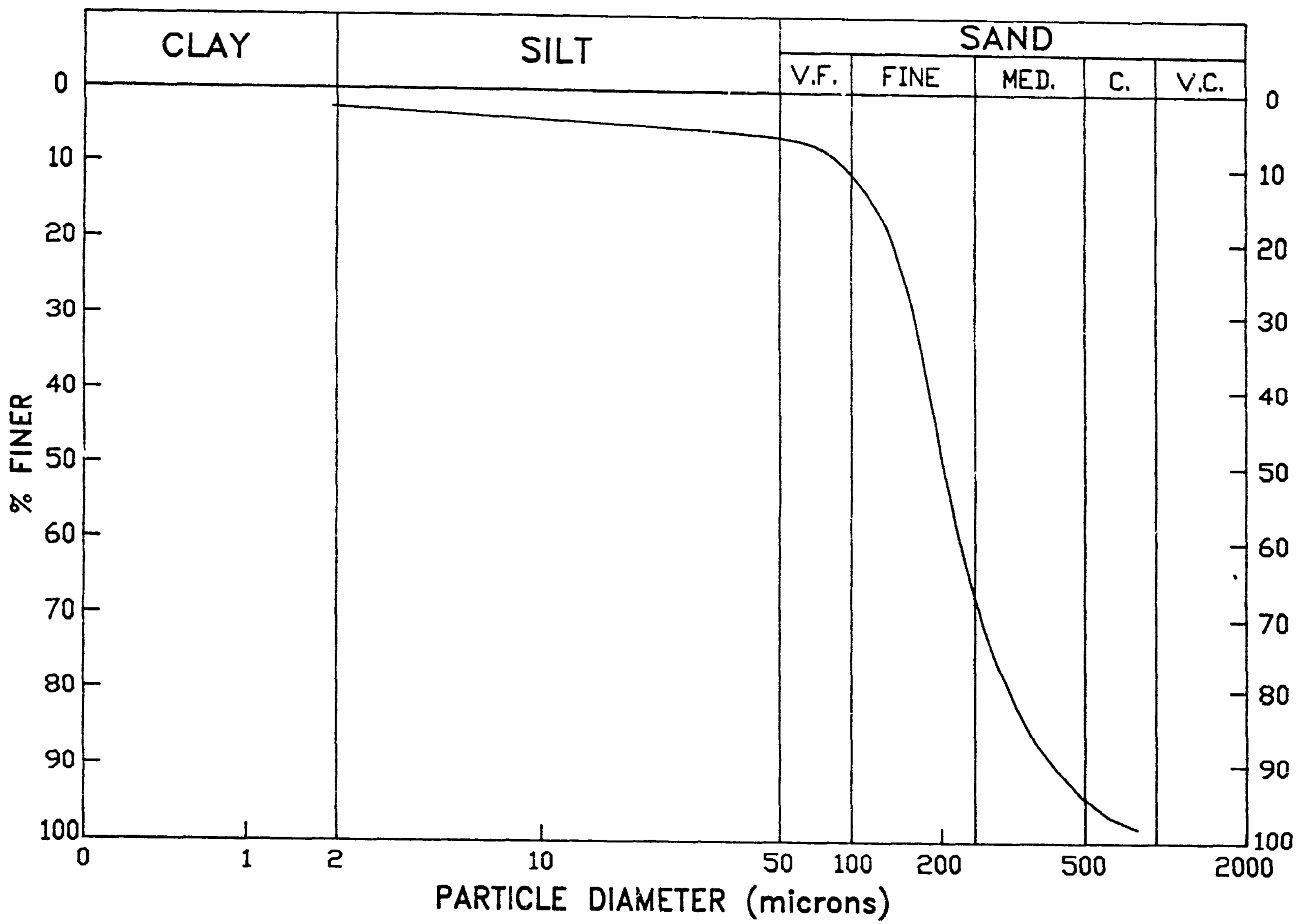


Figure C10: Hammond – Sample Depth 0.6m

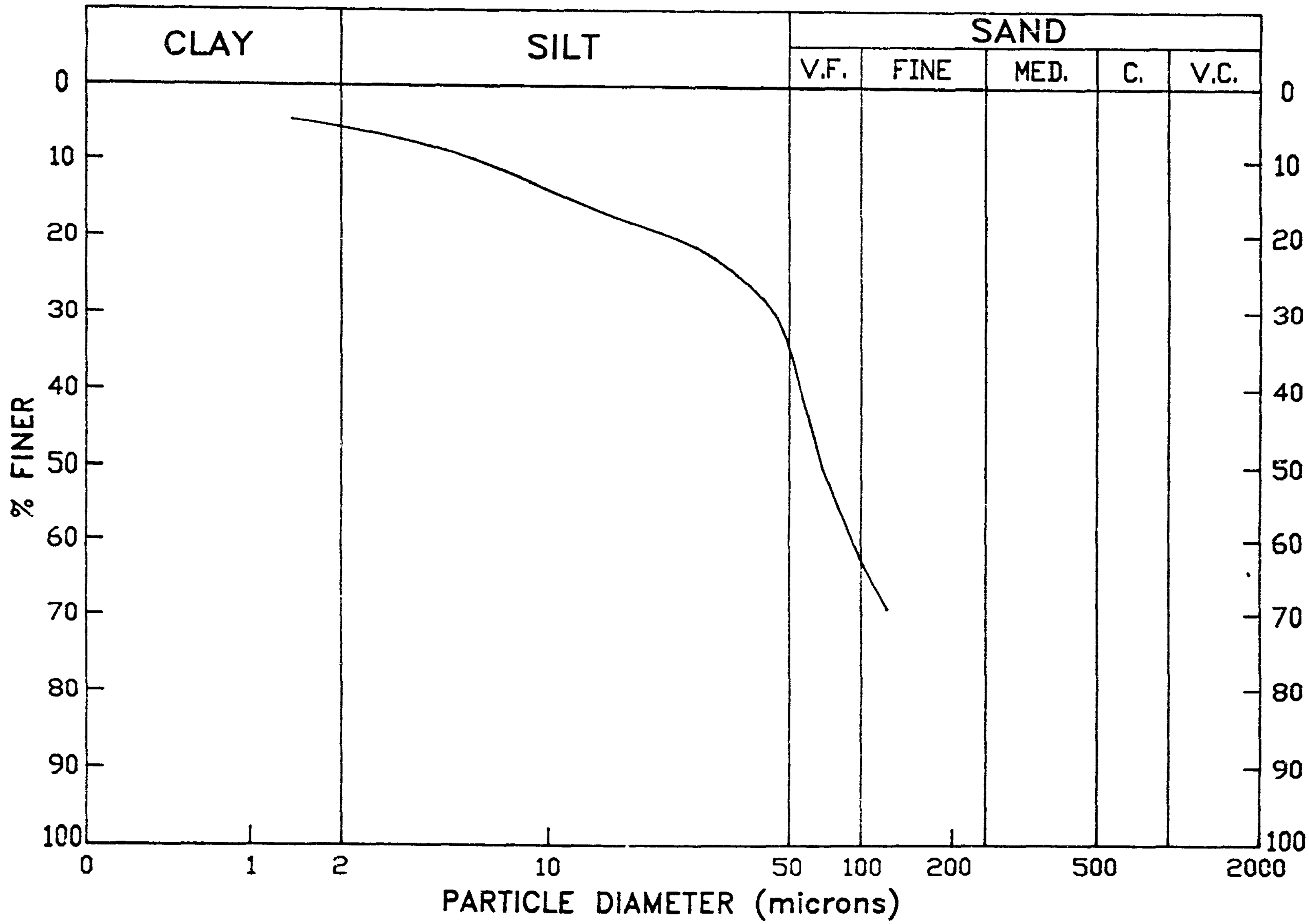


Figure C11: Hammond – Sample Depth 1.2m

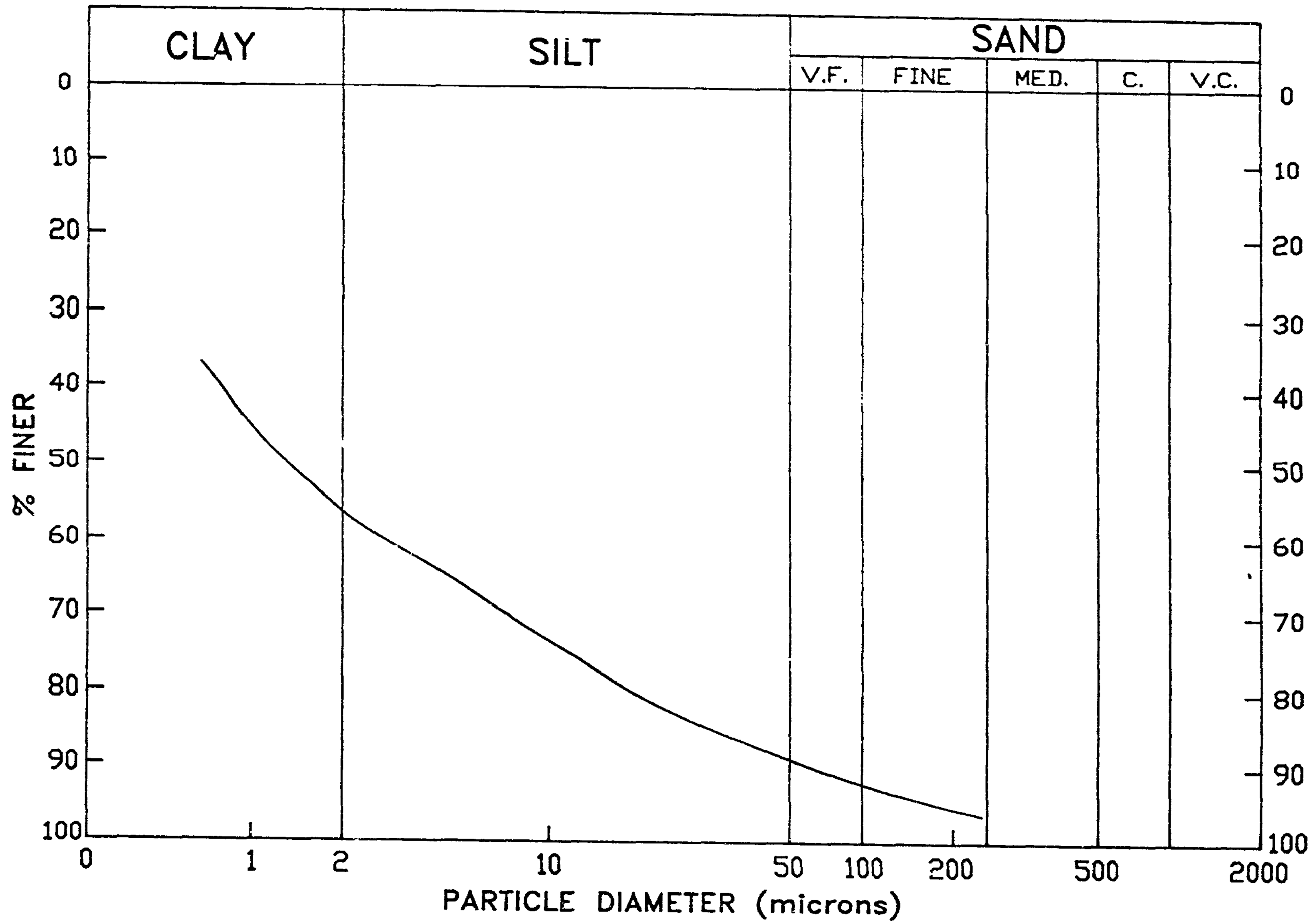


Figure C12: Hammond – Sample Depth 1.8m

Appendix "D"

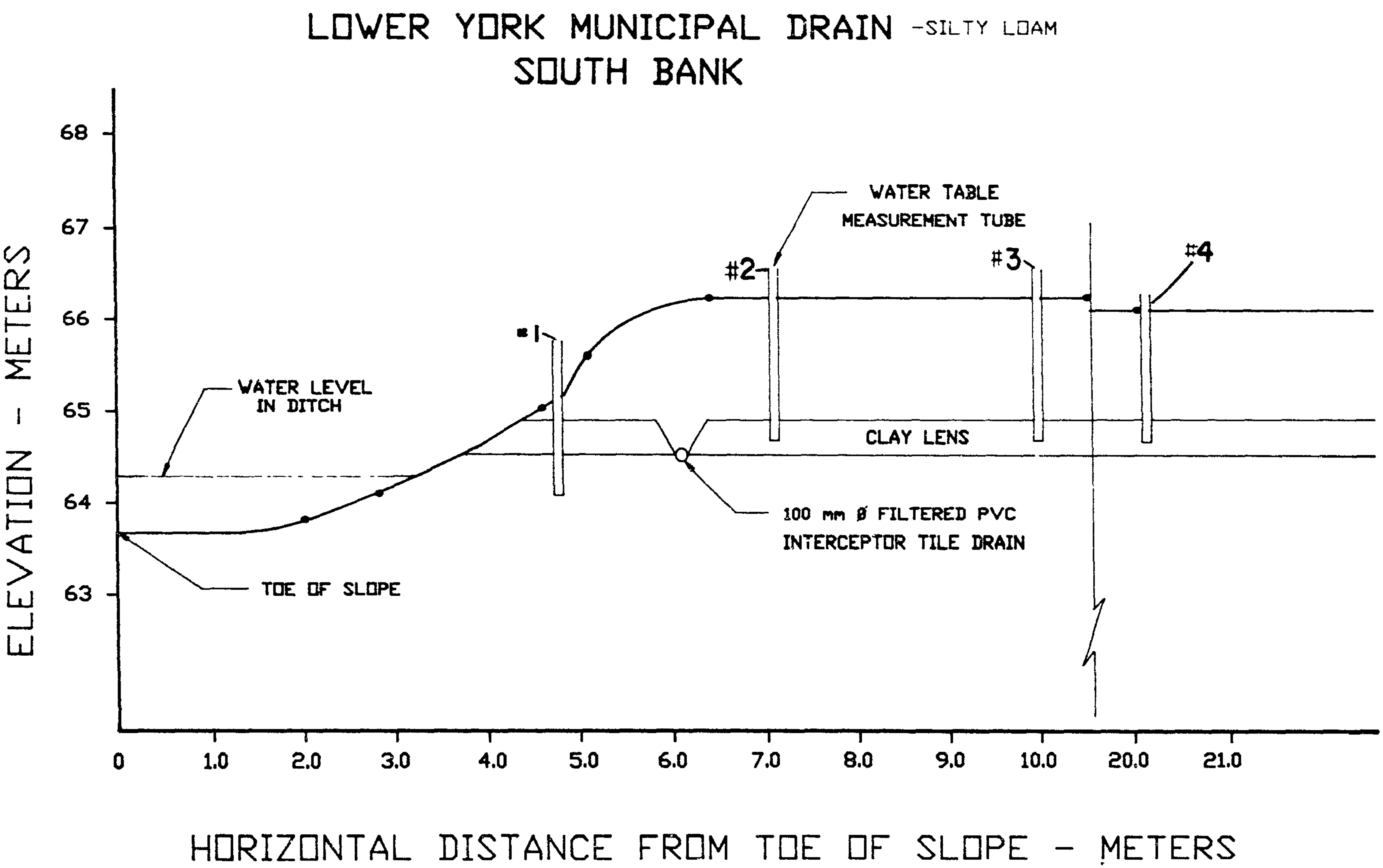

LOWER YORK MUNICIPAL DRAIN -SILTY LOAM
SOUTH BANK
ELEVATION - METERS
68
67
66
65
64
63
WATER TABLE
MEASUREMENT TUBE
#1
#2
#3
#4
WATER LEVEL
IN DITCH
CLAY LENS
100 mm Ø FILTERED PVC
INTERCEPTOR TILE DRAIN
TOE OF SLOPE
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
20.0
21.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

Figure D1

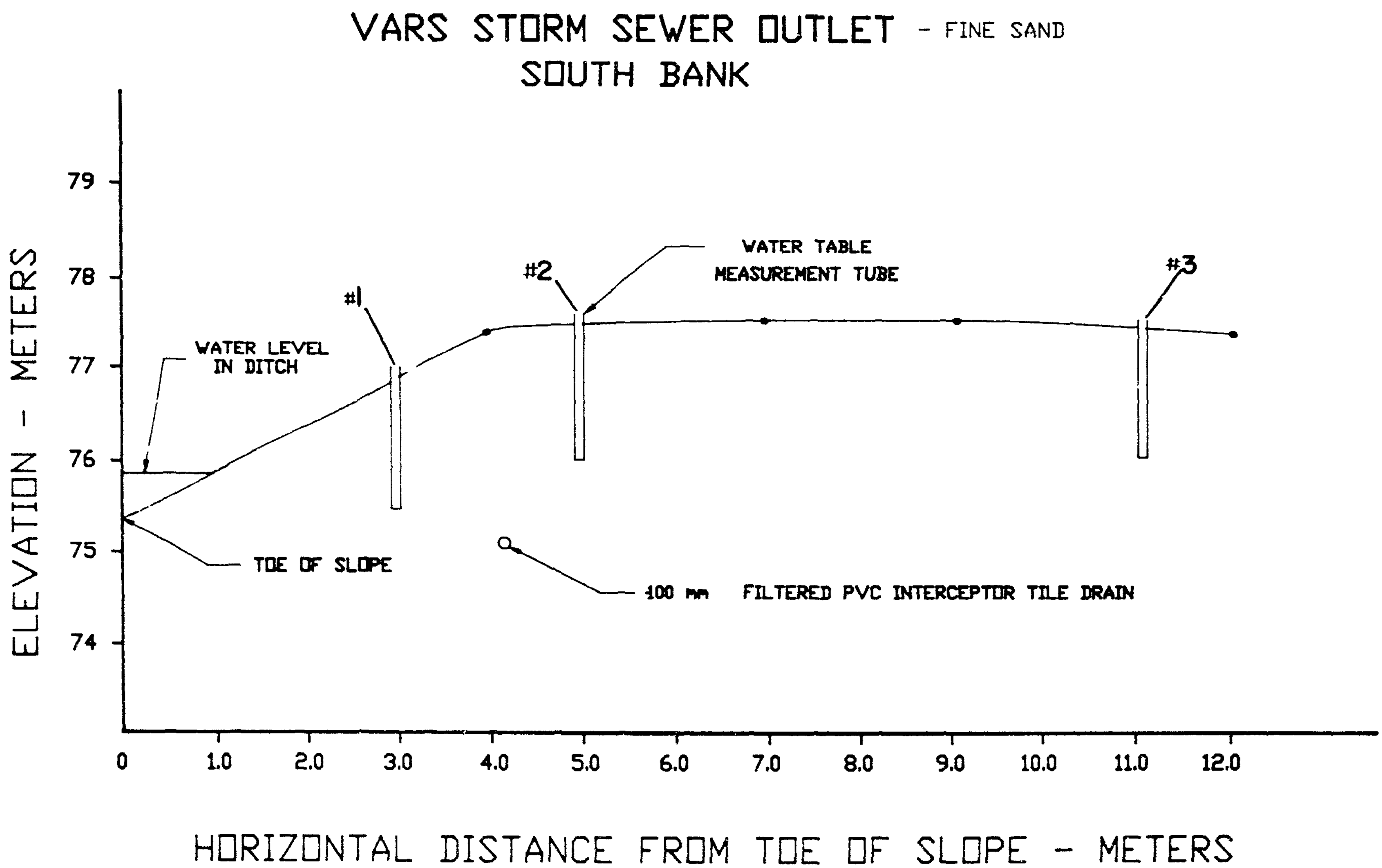

VARS STORM SEWER OUTLET - FINE SAND
SOUTH BANK
ELEVATION - METERS
79
78
77
76
75
74
WATER LEVEL
IN DITCH
TOE OF SLOPE
#1
#2
#3
WATER TABLE
MEASUREMENT TUBE
100 mm FILTERED PVC INTERCEPTOR TILE DRAIN
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

Figure D2

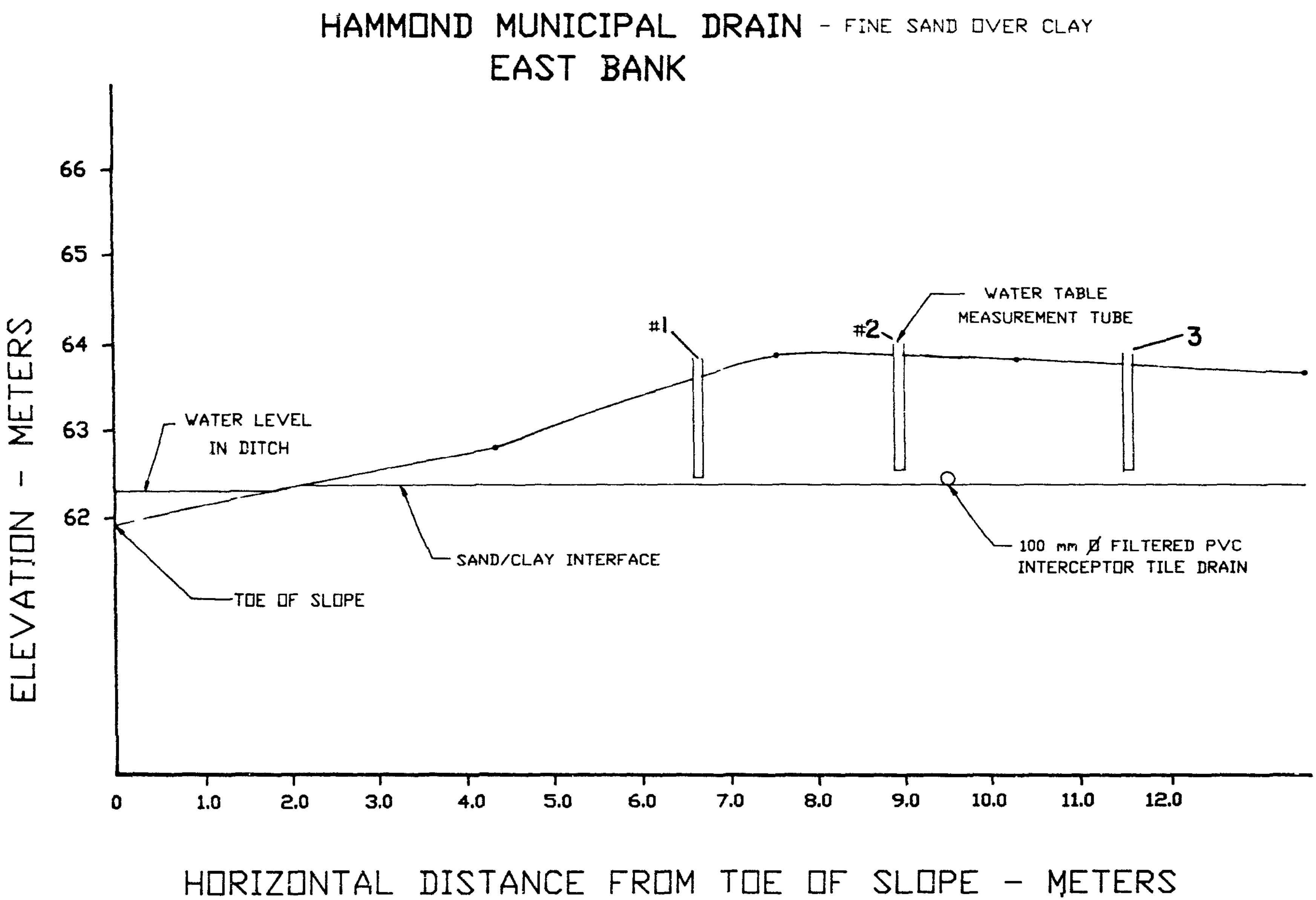

HAMMOND MUNICIPAL DRAIN - FINE SAND OVER CLAY
EAST BANK
ELEVATION - METERS
66
65
64
63
62
#1
#2
3
WATER TABLE
MEASUREMENT TUBE
WATER LEVEL
IN DITCH
SAND/CLAY INTERFACE
TOE OF SLOPE
100 mm Ø FILTERED PVC
INTERCEPTOR TILE DRAIN
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
HORIZONTAL DISTANCE FROM TOE OF SLOPE - METERS

Figure D3

X-SECTION OF NORTH MORROW MUNICIPAL DRAIN -VARVED CLAY
WEST BANK

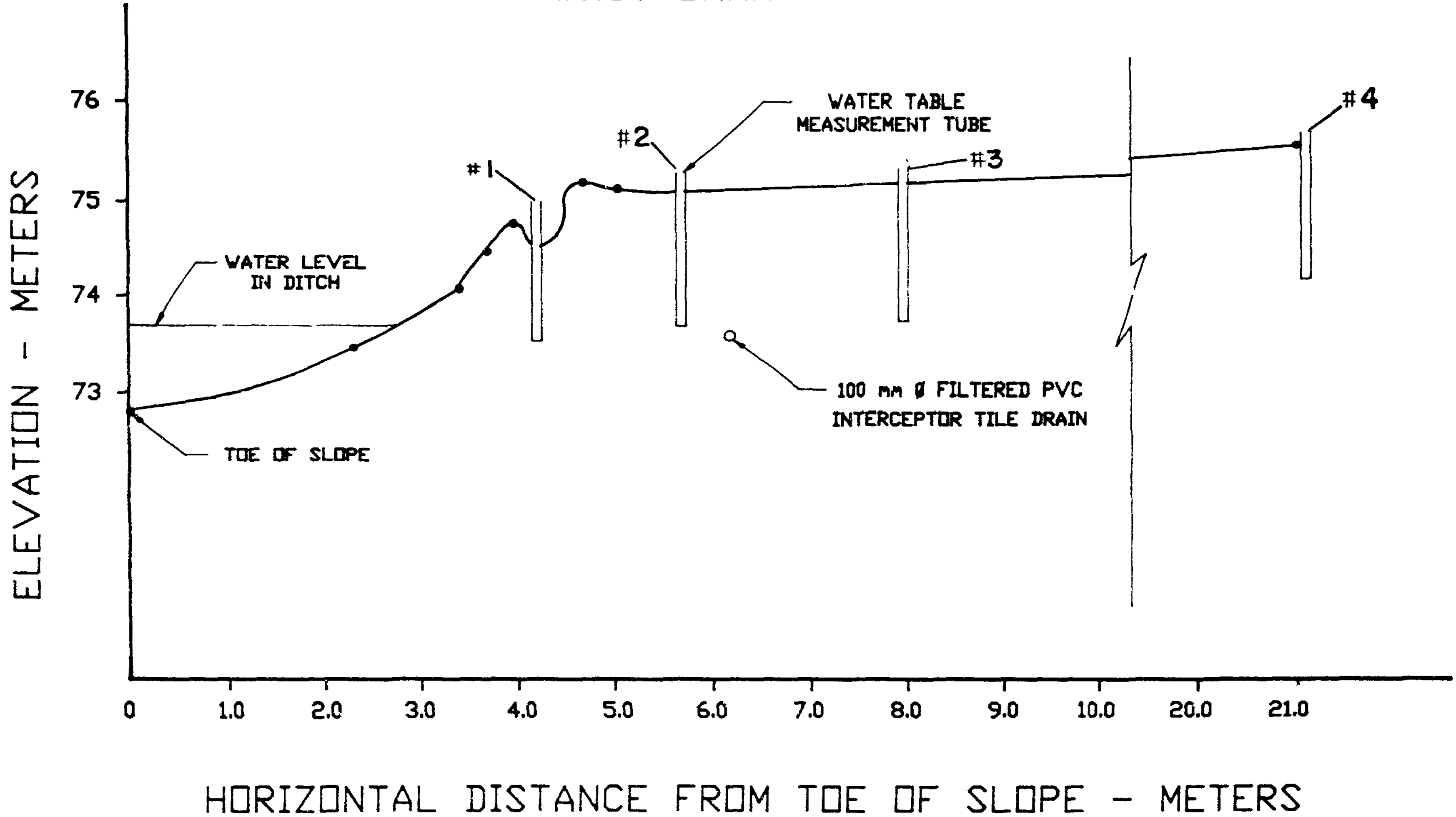


Figure D4

Appendix "E"

Vegetal Concentration by Degree of Cover on Streambank Face

Showing average percentage cover in each degree on a scale of abundance combined.
(Adapted from Trepp, 1950)

Scale of combined estimate of cover and abundance	Range of cover (in percentage)	Average percentage cover
+	up to 1.0	0.1
1	1 to 9.9	5.0
2	10 to 24.9	17.5
3	25 to 49.9	37.5
4	50 to 74.9	62.5
5	75 to 100.0	87.5

Appendix "F"

Nomenclature (with reference to section 2.5.2)

h = depth of watertable to slip line (m)
l = arc length of slip line (m)
ϕ'= effective angle of internal friction (degrees)
C'= effective cohesion (kPa)
u = water pressure (kPa)
A_1= area above watertable (m)
A_2= area below watertable (m)
Y_1= specific gravity of soil
above water table (unsaturated) ($1.6 T/m^3$)
Y_2= specific gravity of soil
below water table (saturated) ($2.0 T/m^3$)
a = angle subtended by trial radius from the vertical
to the arc center (degrees)

$$S.F. = \text{Safety Factor} = \frac{\sum M_r}{\sum M_d} = \frac{\sum C'L + (W \cos a - uL) \tan\phi'}{\sum (W \sin a)}$$

TABLE F-1

NORTH MORROW
WITH TILE

STRIP No.	C'	L	C'L	h	u	uL	A_1 A_2	W_1 W_2
1	10	.235	2.35	.06	.588	.138	.0125	.245
2	10	.26	2.60	.23	2.254	.518	.0056 .0563	.088 1.104
3	10	.275	2.75	.24	2.352	.647	.058 .058	.909 1.137
4	10	.35	3.50	.13	1.274	.446	.1025 .0325	1.6 .637
5	18	.5	9.0	0.0	0.0	-.-	.0788	1.236

WITHOUT

STRIP No.	C'	L	C'L	h	u	uL	A_1 A_2	W_1 W_2
1	10	.235	2.35	.07	.686	.161	-.- .0125	-.- .245
2	10	.26	2.6	.24	2.352	.612	-.- .0619	-.- 1.213
3	10	.275	2.75	.38	3.724	1.024	.0218 .0938	.342 1.838
4	10	.35	3.5	.3	2.94	1.029	.075 .0594	1.176 1.164
5	10	.5	5.0	.115	1.127	.058	.0519 .0269	.814 .527

MUNICIPAL DRAIN
DRAIN

W	a	ø'	Wcosa	tanø'	C'L+(Wcosa-uL)tanø'	Wsina
.245	7	8	.243	.141	2.365	.03
1.192	16	8	1.15	.141	2.69	.329
2.05	27	8	1.83	.141	2.91	.931
2.24	40	8	1.72	.141	3.68	1.44
1.236	56	13	.69	.23	9.16	1.02

$\sum W = 6.963$

$$\boxed{S.F. = \frac{\sum M_r}{\sum M_d} = \frac{20.805}{3.75} = 5.54}$$

TILE DRAIN

W	a	ø'	Wcosa	tanø'	C'L+(Wcosa-uL)tanø'	Wsina
.245	7	8	.243	.141	2.362	.034
1.213	16	8	1.166	.141	2.678	.339
2.18	27	8	1.942	.141	2.879	.990
2.34	40	8	1.793	.141	3.608	1.504
1.34	56	8	.749	.141	5.097	1.111

$\sum W = 7.318$

$$\boxed{S.F. = \frac{\sum M_r}{\sum M_d} = \frac{16.62}{3.389} = 4.17}$$

TABLE F-2

HAMMONI
WITH TILE

STRIP No.	C'	L	C'L	h	u	uL	A_1 A_2	W_1 W_2
1	3	0.38	1.14	0.15	1.47	0.56	.0525	1.03
2	3	0.26	0.78	0.25	2.45	0.64	.0625	1.225
3	3	0.28	0.84	0.22	2.16	0.61	.0075 .0575	0.1176 0.735
4	3	0.33	0.99	0.15	1.47	0.48	.0225 .0375	0.353 0.735
5	6.0	0.48	1.44	-.-	-.-	-.-	.0350 .0025	0.549 0.049

WITHOUT

STRIP No.	C'	L	C'L	h	u	uL	A_1 A_2	W_1 W_2
1	3	.38	1.14	.15	.47	.56	.0525	1.03
2	3	.26	.78	.25	2.45	.64	.0625	1.225
3	3	.28	.84	.30	2.94	.82	.0650	1.228
4	3	.33	.99	.30	2.94	.97	.0600	1.176
5	3	.48	1.44	.17	1.67	.80	.0375	.864

ND MUNICIPAL DRAIN
LE DRAIN

W	a	ø'	Wcosa	tanø'	C'L+(Wcosa-uL)tanø'	Wsina
1.03	7	30	1.022	.577	1.406	.126
1.225	8	30	1.213	.577	1.045	.171
1.245	20	30	1.17	.577	1.16	.426
1.088	34	30	.90	.577	1.23	.61
.598	49	32	.39	.63	1.686	.45

$\sum W = 5.186$

$$\boxed{S.F. = \frac{\sum M_r}{\sum M_d} = \frac{6.60}{1.78} = 3.7}$$

IT TILE DRAIN

W	a	ø'	Wcosa	tanø'	C'L+(Wcosa-uL)tanø'	Wsina
1.03	7	30	1.022	.577	1.406	.126
1.225	8	30	1.213	.577	1.045	.171
1.274	20	30	1.197	.577	1.058	.436
1.176	34	30	.97	.577	1.109	.658
.864	49	30	.57	.577	1.31	.652

$\sum W = 5.569$

$$\boxed{S.F. = \frac{\sum M_r}{\sum M_d} = \frac{5.93}{2.043} = 2.90}$$

Appendix "G"

EXHIBIT #1 VARS DRAINAGE OUTLET

View of open channel in Spring, 1984. Sideslopes were constructed 2h:1v and hydroseeded. Lateral interceptor tiles were installed prior to excavation to stabilize ditchbanks.

EXHIBIT #2 VARS DRAINAGE OUTLET

Same channel during Summer of 1984 showing dense vegetal cover which has further increased bank stability.

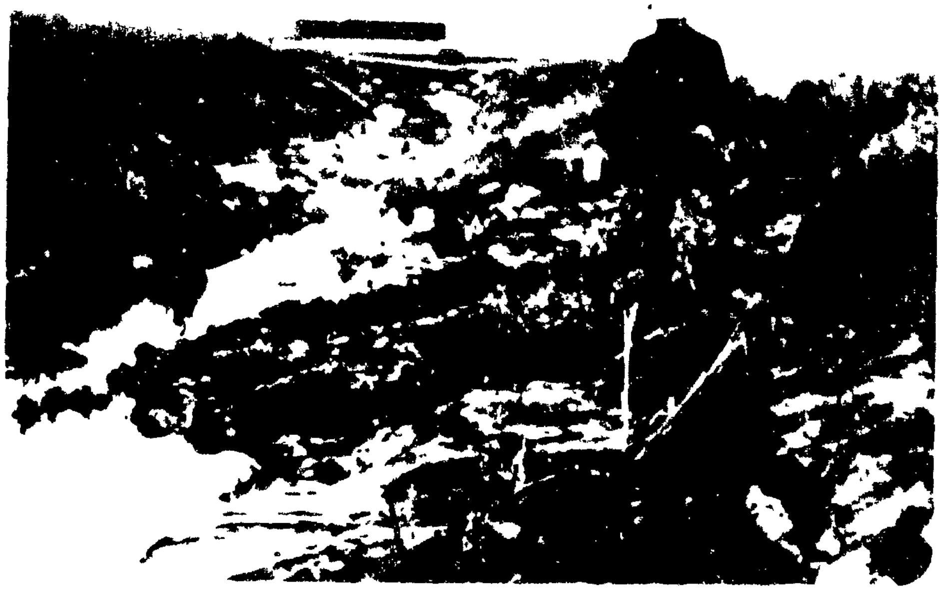


EXHIBIT #3 NORTH MORROW MUNICIPAL DRAIN

Pictured in the Spring of 1980, 2 years after construction. Drain is located in a varved clay. Both sideslopes exhibit extensive slumping.

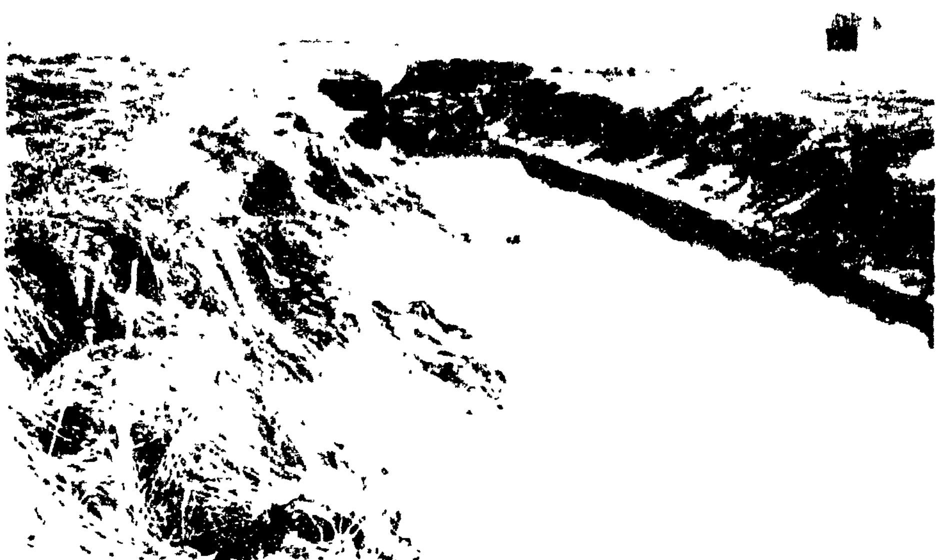


EXHIBIT #4 NORTH MORROW MUNICIPAL DRAIN

Same drain in the Spring of 1984. Interceptor tiles and parabolic template were constructed in Fall, 1983. Sideslopes appear to be stable.

EXHIBIT #5 LOWER YORK MUNICIPAL DRAIN

High seepage and undercutting at the toe of the ditchbank. Note the saturated loose sediment at the bottom of drain which will eventually be washed away.

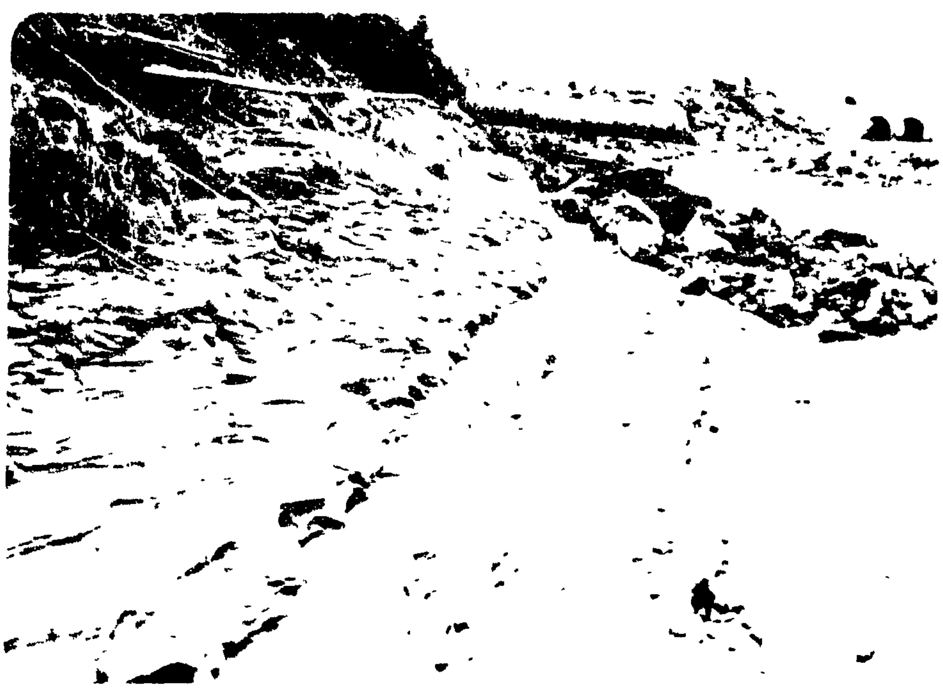


EXHIBIT #6 LOWER YORK MUNICIPAL DRAIN

Bottom of this watercourse reveals a varved clay. Lateral seepage in thin layers of silt and sand washing out between the clay layers.

EXHIBIT #7 LOWER YORK MUNICIPAL DRAIN

View of open channel during Spring runoff of 1983. Three stages of slumping are illustrated here. The middle section demonstrates a slide of ditchbank material. Immediately above, a similar slide which has progressed further and slipped into the drain. In the lower part of the picture, the final stage of failure, shows where ditch bank has been washed away and now forms a parabolic shaped section.

EXHIBIT #8 NORTH MORROW MUNICIPAL DRAIN

Advanced tension crack along ditchbank in a varved clay. Opening is 500 mm deep and 150 mm wide with a saturated silty sand at the bottom of the crack.

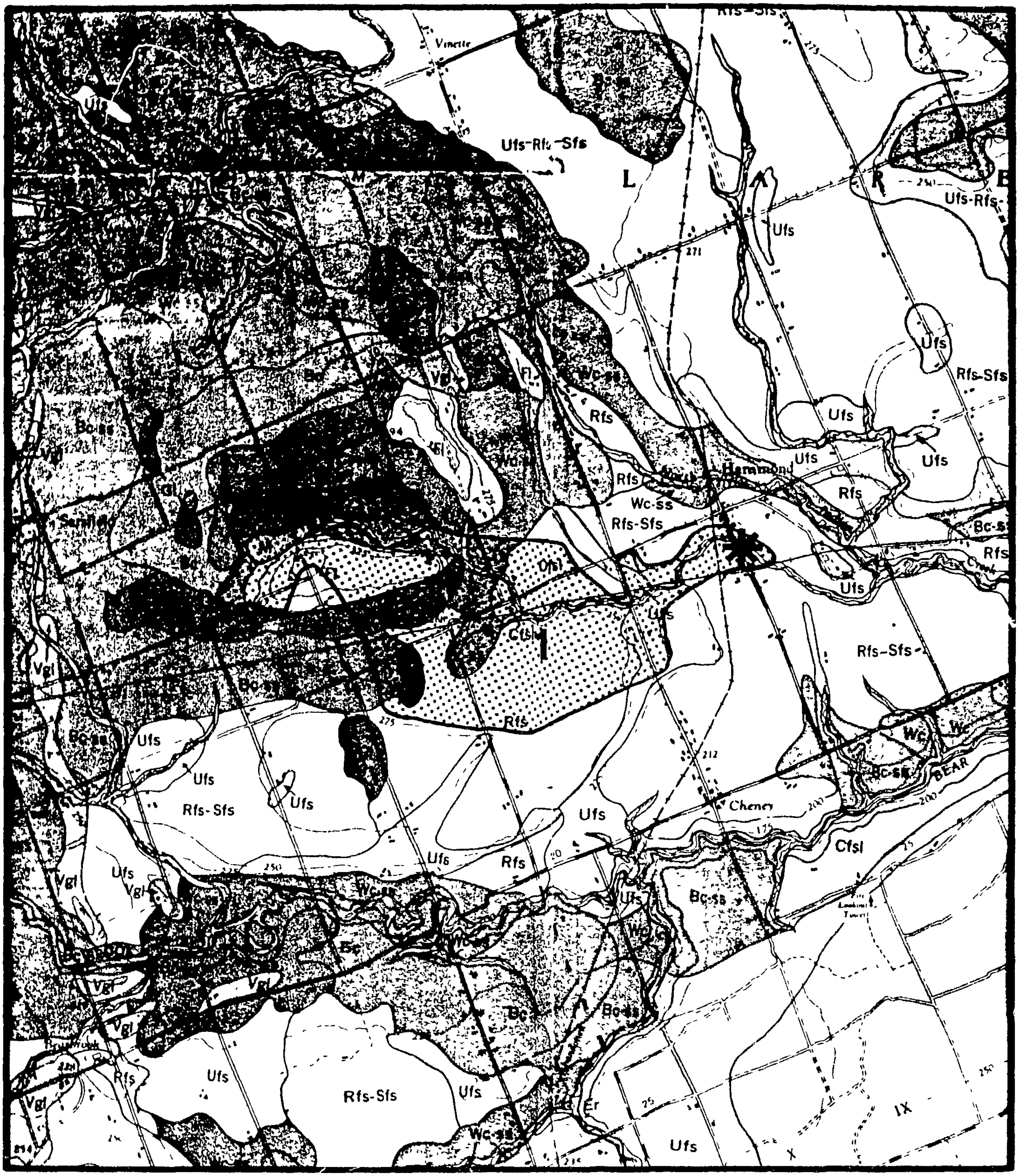
EXHIBIT #9   HAMMOND MUNICIPAL DRAIN

Test pit dug adjacent to drain shows loamy sand over clay. Seepage at the sand clay interface results in flow of loamy sand.

EXHIBIT #10   HAMMOND MUNICIPAL DRAIN

Extreme left of picture illustrates results of seepage as discussed in Exhibit #7. Remedial measures included interceptor tile drain and reseeding.

Appendix "H"



DRAINAGE AREA AND SOIL DESCRIPTION FOR EXPERIMENTAL SITE

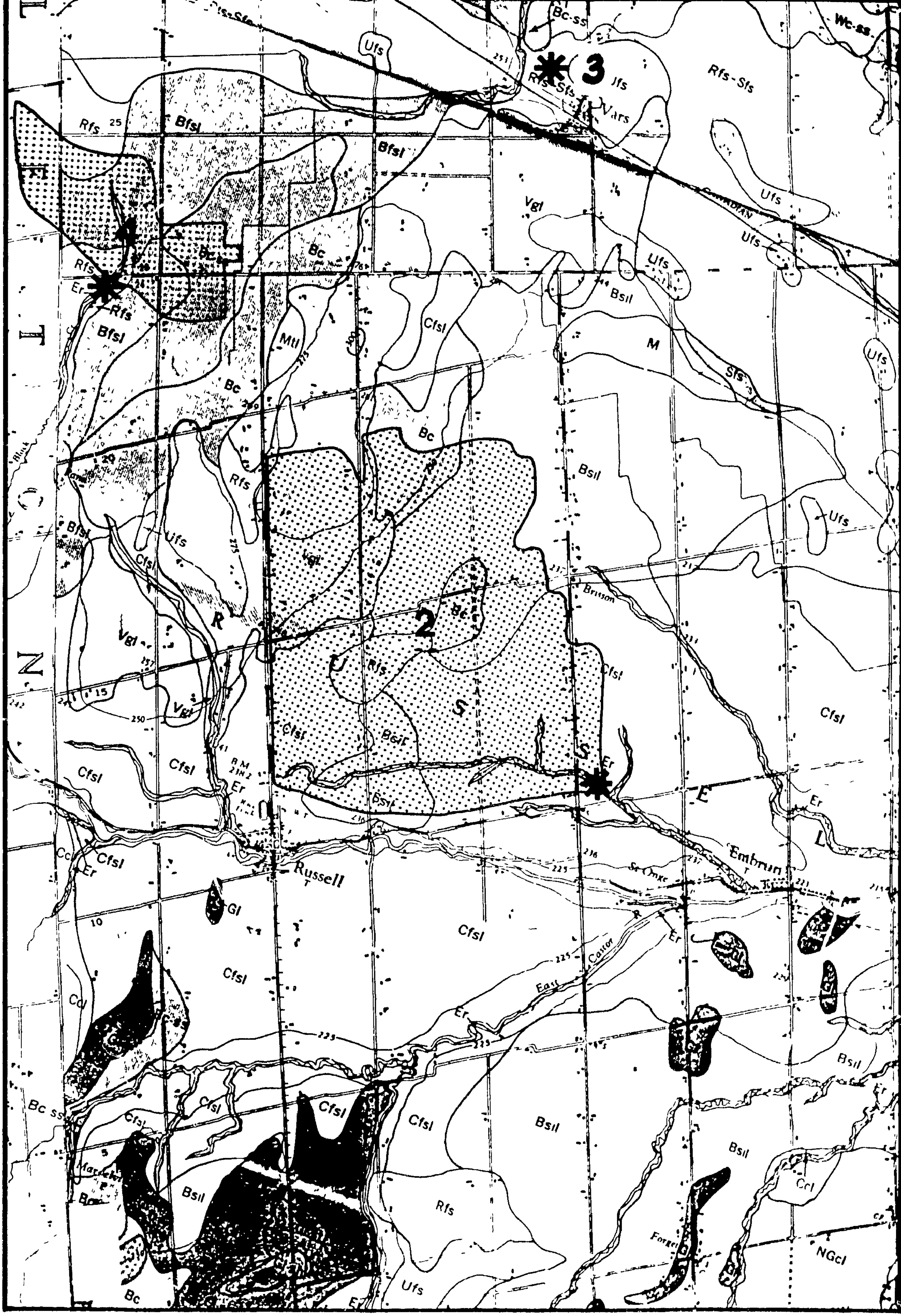
1 - HAMMOND MUNICIPAL DRAIN

2 - LOWER YORK MUNICIPAL DRAIN

3 - VARS STORM SEWER OUTLET

4 - NORTH MORROW MUNICIPAL DRAIN

EXPERIMENTAL SECTION



3
Vars
Rfs-Sfs
Bc-ss
Wc-ss
Ufs
Rfs
Bfsl
Bc
Vgl
Bsil
Cfsl
Mtl
M
Sfs
1
2
R
U
S
S
E
L
Brisson
Embrun
Russell
Er
Gl
East Castor
Cl
Ccl
NGcl
Bc ss
Forget
T
N