AN INVESTIGATION INTO RIVER BANK IMPROVEMENT IN THE WINNIPEG AREA

A Thesis

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Edwin H. Klassen

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SUMMARY

In this study of "Investigation Into River Bank Improvement In Metropolitan Winnipeg" an effort has been made to gain insight into the problems, solutions and economics of providing complete overall protection for the river banks of Metropolitan Winnipeg. The basic principle has been that if protection is carried out in a "piecemeal" manner there may be a disturbance of the natural regimen of the rivers, resulting in movement of the destructive forces downstream from the isolated areas of protection. This action would render the original protection useless and further protection would be required downstream from the original location.

Investigations were made into three aspects of river bank stability, namely, geomorphology, hydraulics, and soils. Solutions were classified into the three categories of bank drainage, resedimentation, and structures. The economics of various combinations of these solutions were studied by a comparison of total benefits and total costs. Costs were computed in the usual manner. For a derivation of benefits a survey was made regarding the amount of bank recession occurring at various locations, together with the proximity of buildings to the tops of banks. These figures were then developed into a recession-damage curve. Of the seven projects studied, two were considered to be economical but were not further pursued because they lacked the adequacy of protection required. Two projects, according to the assumptions and calculations made, yielded substantial protection for the most adverse conditions. These projects, a retaining wall and a heavy toe berm, whose total costs were estimated at 147 million dollars and 20 million dollars respectively, were both in excess of the total estimated benefits of approximately 17.5 million dollars. It is suggested, however, that further study be made of the project of toe fill since the difference in cost and benefit is not unduly large for a preliminary study.

In the survey made, inquiries were also made regarding allocation of costs of bank improvement projects. These were analyzed and the figures arrived at allotted the Federal Government 48.8 per cent, the Provincial Government 35.2 per cent, Metropolitan Government 10.4 per cent and the owner 5.6 per cent. These figures were considered, by the author, to be unrealistic, and the division of costs was placed at 30 - 50 per cent for the owner and the remainder for Metropolitan Government, with substantial contributions by the municipalities concerned.

CHAPTER I

INTRODUCTION

Hargrave, in his study of the Red River states:

> "A study of the historic development of settlements along the Red and Assiniboine Rivers reveals the significance of the river as related to the sites of settlements. From earliest times the waterways were essential to transportation. It is, therefore, not surprising that practically every settlement was located along the two rivers or their smaller tributaries. The present site of Winnipeg, for example, was originally the chief point of departure and distribution during the period of early exploration and settlement, due to its strategic position at the confluence of the Red and Assiniboine Rivers". (1)

Further, Hargrave's account states that:

> "The houses in no place extend back from the rivers, proximity to which has hitherto formed the sole reliance of the inhabitants for their water supply".

Even today one of the most obvious proofs of this statement is the division of large portions of property into long narrow strips, extending perpendicularly away from the rivers.

After the Manitoba Act in 1870, government land surveys were begun, and once the potentialities of the land in the Red River Valley were realized, settlers began arriving in increasing numbers, initiating a period of intense agricultural development. Unlike agriculture, industry began at Winnipeg on a small scale some fifty to sixty years ago. Growth in this field has accelerated considerably since then and unfortunately, too much of the development has taken place near the river banks. Whereas in pioneer days stores and trading posts were relatively small and temporary and could be easily relocated because of river encroachment, today factories are of such a size that large amounts, indeed sometimes excessive amounts of money are required to remove an industry to safer ground. To this date, few if any moves have been necessary, but within the next few decades river encroachment will be a menace and a hazard to industry located near the banks of rivers. At present, 15,000 feet of bank are occupied by industry.

The problems and the accompanying hazards of erecting structures near river banks have been realized for many years. In the settlers' days, living near the river's edge during the flood meant, frequently, the sweeping away of all property, food and often domestic animals. In more recent years, while the largest floods have been of somewhat smaller magnitude than in settlers' years, structures have been built more soundly and lavishly, with the result that they have not been washed away but that the damage due to ice and water has been almost as critical, financially. Despite the fact that the river's potential danger has long been known, and is becoming more and more obvious through bank erosion, property adjacent to the river is becoming increasingly more desirable and more costly to purchase. Thirty to fifty years ago, it may have been possible to purchase all river bank property and convert it to stable banks, used as playgrounds and parks. This is now an almost insurmountable task.

The river banks throughout the Winnipeg area are generally fairly well defined and quite steep. Through many of the residential areas, particularly from the north end of the city to Middlechurch, the banks are well treed. The tree line in most cases extends but a short distance below the top of the bank. At many locations, where the banks have slipped extensively the trees have been carried down with the banks, but still flourish.

Erosion of the banks is now quite general and particularly prevalent on almost all concave or outside bends. Erosion is light on convex or inside bends where extensive willow growth exists, this growth, being prevalent in the northern and southern portions of the city.

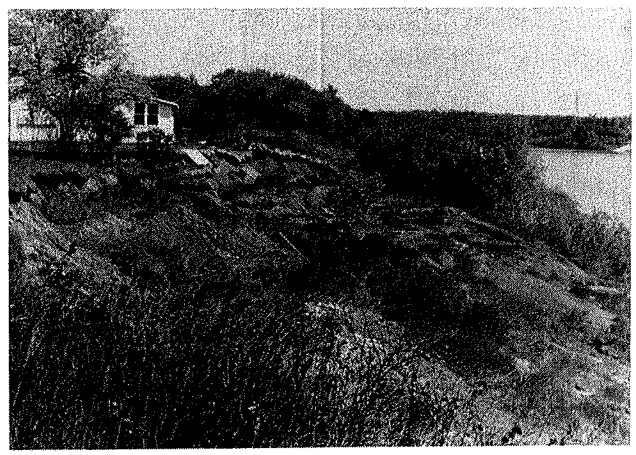
Throughout the built-up areas, both residential and commercial, many buildings, as mentioned above, are located close to the river banks. Subsequent erosion and slides, much of which appears to have occurred since 1950, have resulted in loss of property but only limited damage to buildings. In some instances buildings have had to be moved. Individual efforts have been made to prevent erosion and to rectify the condition of the banks resulting from the slides. These frequently consist of improperly located stone riprap, construction of cribs or driving of piles, and the placement of backfill, with the result that the problem is often only aggravated.

The main causes of bank failure by sliding are the overloading of banks by man, and secondly the reducing of the factor of safety against sliding by bank saturation and erosion. Indiscriminate dumping of waste or spoil material and the stockpiling of equipment and building supplies along the river banks, which has been common practice in certain areas, tend to overload the banks and may often contribute to subsequent slides. In April 1951 Bill 70, "An Act to amend the Rivers and Streams Act" was passed in the Manitoba Legislature. This act provides that no person or persons, without a permit, shall deposit any material or erect any structures within one hundred and fifty feet from the normal summer water level. Even this, later studies have indicated, may be inadequate. Dumping on the river banks has not, at the present time, been sufficiently widespread to make any appreciable change in the carrying capacity of the channel, although it does result in increased velocities where dumping has taken place, producing localized erosion. The driving of piles and placing of riprap has proven to be quite adequate for minor toe erosion but a survey of these installations has shown that when a slide occurs, these installations have moved along with the sliding banks. Photographs number 1 to number 6, page 5, show

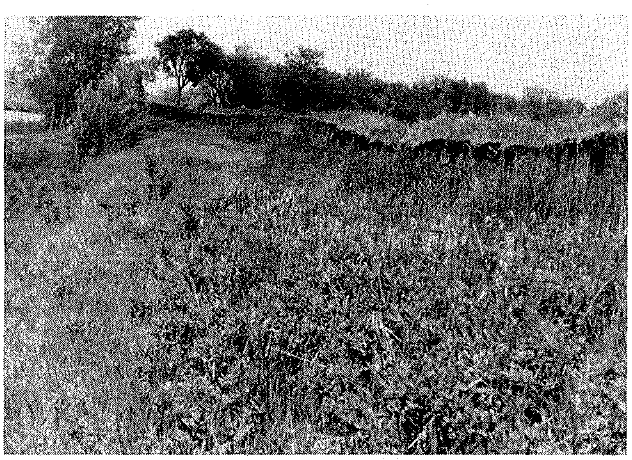
PHOTOGRAPH 1 - Recent Slide
Concave Edge, East Bank
Height 33 ft. Average Slope 6.8 : 1

PHOTOGRAPH 2 - Older Slide
Concave Edge, East Bank
Height 33 ft. Average Slope 6.7 : 1

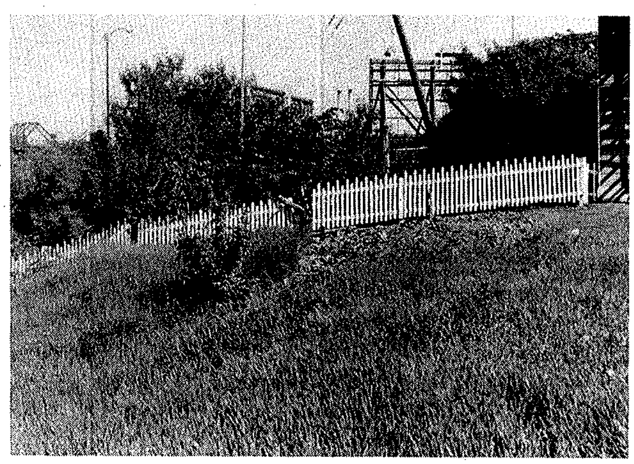
PHOTOGRAPH 3 - Recent Slide
Concave Edge, West Bank Looking South
Height 40 ft. Average Slope 5.1 : 1



PHOTOGRAPH 4
Same as Photograph 3
West Bank Looking North



PHOTOGRAPH 5 - Old Slide
Convex Edge, East Bank
Height 30 ft. Average Slope 5.4 :1



PHOTOGRAPH 6 - Old Slide with Fill
Concave Edge, East Bank
Height 34 ft. Average Slope 4.9 : 1
Note Fence Separation indicating continued movement

Courtesy Professor A. Baracos

some of the slides which have occurred at various locations in the city of Winnipeg. The first of two major problems, then, was created by man when he settled too near the river banks.

The second major problem arising, with regard to stable and unstable river banks, and one that may more feasibly be solved or alleviated, is the flood. In the usual concept, a flood exists where the water level of a stream exceeds a certain level and the flow threatens life or property. This definition is quite appropriate for the present discussion. In the problem of unstable river banks, it is the prevailing opinion of waterways authorities that most slides in the soil types that exist in Winnipeg, are instigated by a rapid variation in water level. The stipulation made in this study, therefore, is that when the water level reaches the 1950 flood stage and then recedes the river banks may become unstable and subject to failure. According to a well-known Soil Mechanics Engineer

> "All major bank failures in the Winnipeg area seem to have in common that they occurred during or after subsidence of the river after a severe flood condition." (2)

While this theory has not gained recognition by some authorities, it is felt that major floods can cause extensive overall instability in river banks and floods of lower magnitude than the defined flood stage and which occur after the mentioned major flood are known to have caused considerable damage to river banks, depending on the duration of high water and rapidity of drawdown.

Floods in the lower reaches of the Red and Assiniboine Rivers have always been associated with spring snow melt. Major summer rainstorms of sufficient intensity and extent to cause general flooding are extremely rare in this area. Although in an average year, snow makes up only about 17 per cent of the total yearly precipitation, heavy snowfalls, some totalling more than 100 inches have occurred. The accumulation of these heavy snowfalls in combination with other factors has been the major cause of general river overflows. Flood hydrographs associated with snowmelt run-off are characterized by slowly rising stages, prolonged high flows, and gradual recessions. Ice jams occur occasionally, and may cause local increased flood heights.

Records of floods were first taken in Winnipeg 1874 in the form of spring peak flood stages. Prior to this historical accounts indicate that floods of large proportions occurred in 1826, 1852, and 1861. The fourth largest flood on record was that of 1950. Floods of somewhat lesser proportions were recorded in the years 1882, 1893, 1897, 1904, 1916, and 1948. The following Table I shows the major floods on record:

TABLE I

FLOOD FLOWS

Date of Max. Discharge			Estimated Max. Discharge at Redwood Bridge	Max. Elevation at junction of Red and Assiniboine	Estimated Return Period
Year	Month	Day	Second-feet	G. S. of C. Datum	Years
1826	May	21	225,000	764.5	250
1852	May	21	165,000	62.5	84
1861	May	8	125,000	60.5	50.5
1950	May	19	103,600	58.5	36.0
1882	May	3	79,700	53.6	23.8
1916	April	22	71,200	51.6	17.3
1948	April	30	69,000	51.2	14.2
1904	April	24	66,000	52.2	11.8
1897	April	27	62,500	50.0	10.1
1893			63,300	49.7	8.8

From the table it is readily seen that the probability of having a flood equal to the 1950 flood stage is high.

Before urban development of any considerable scale had begun, floods were of small consequence. Recently, however, the direct flood damage and the potential damage due to bank instability has been increasing steadily due to the progressive development of lands along the river. In Winnipeg, the trend towards building on river banks in lower lying areas has been accelerated in recent years. Examples of this improvident settlement may be seen in the Riverview, Wildwood, and St. Vital areas of Metropolitan Winnipeg. Many slides in these areas occurred after the 1950 flood and much loss of property resulted and is continuing.

Although sliding has been a major cause of the degradation of the quality of the river bank, erosion, a factor often neglected, contributes much towards overall river bank instability. As can be seen on Plate No. I, page 141, the rivers follow a tortuous channel through Winnipeg. On all concave bends, unless protected, constant toe erosion takes place. Plates II and III, page 10 and 11, show some cross sections taken in the years 1886, 1912, and 1951. The accompanying description of these plates as given in an investigation of the Winnipeg flood hazards states:

> "The cross sections of the river channels have undergone little change in area." (3)

While this is true, it must also be observed that the cross section of the river has shifted somewhat, that is, the concave banks have eroded while the convex banks have been subject to sedimentation.

The shift as shown, appears insignificant for the period of record, which extends up to the year of a major flood, 1950.

The statement by Mr. A. Casagrande regarding the occurring of bank failures after a major flood may now bear significance. A

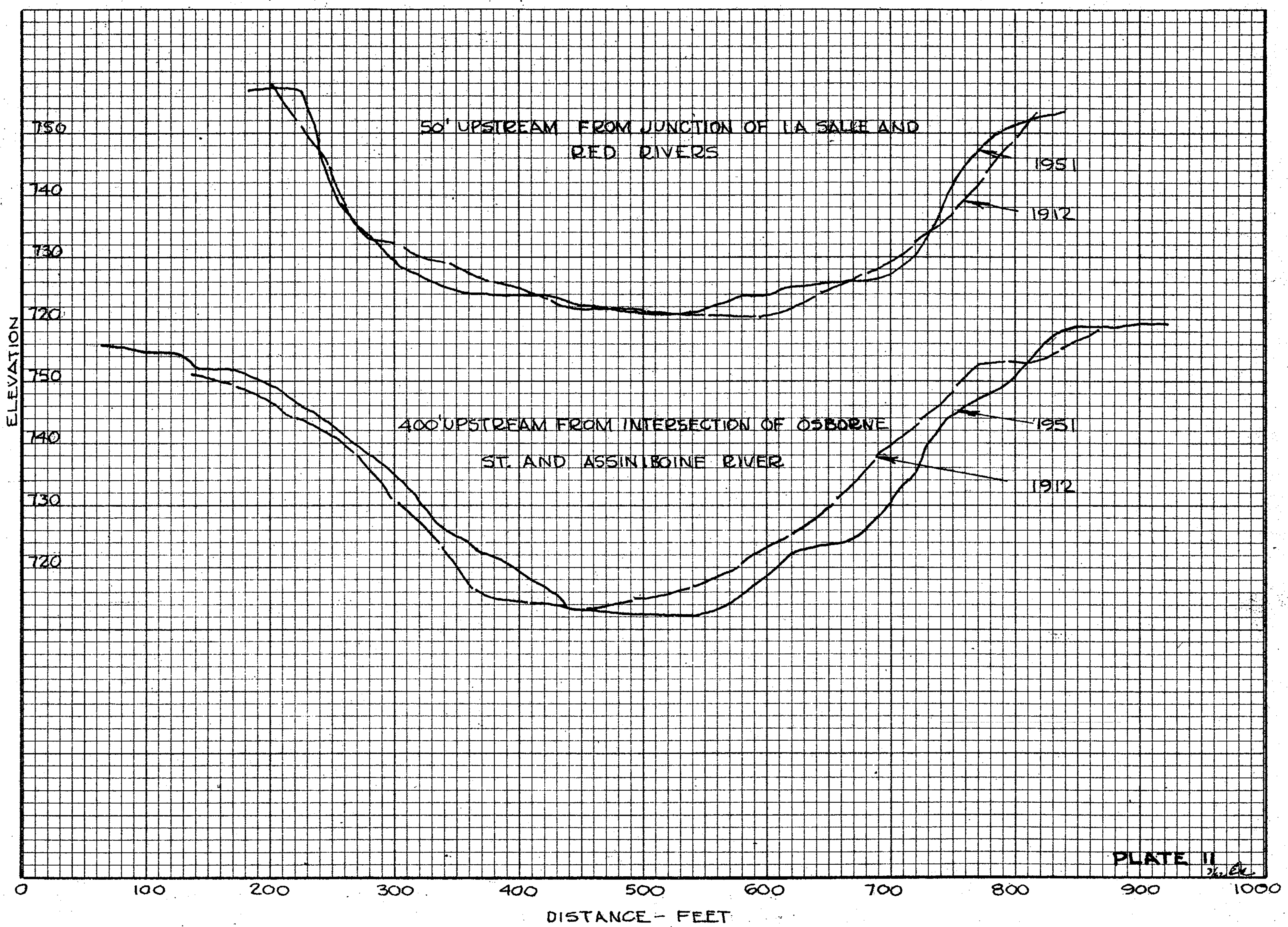
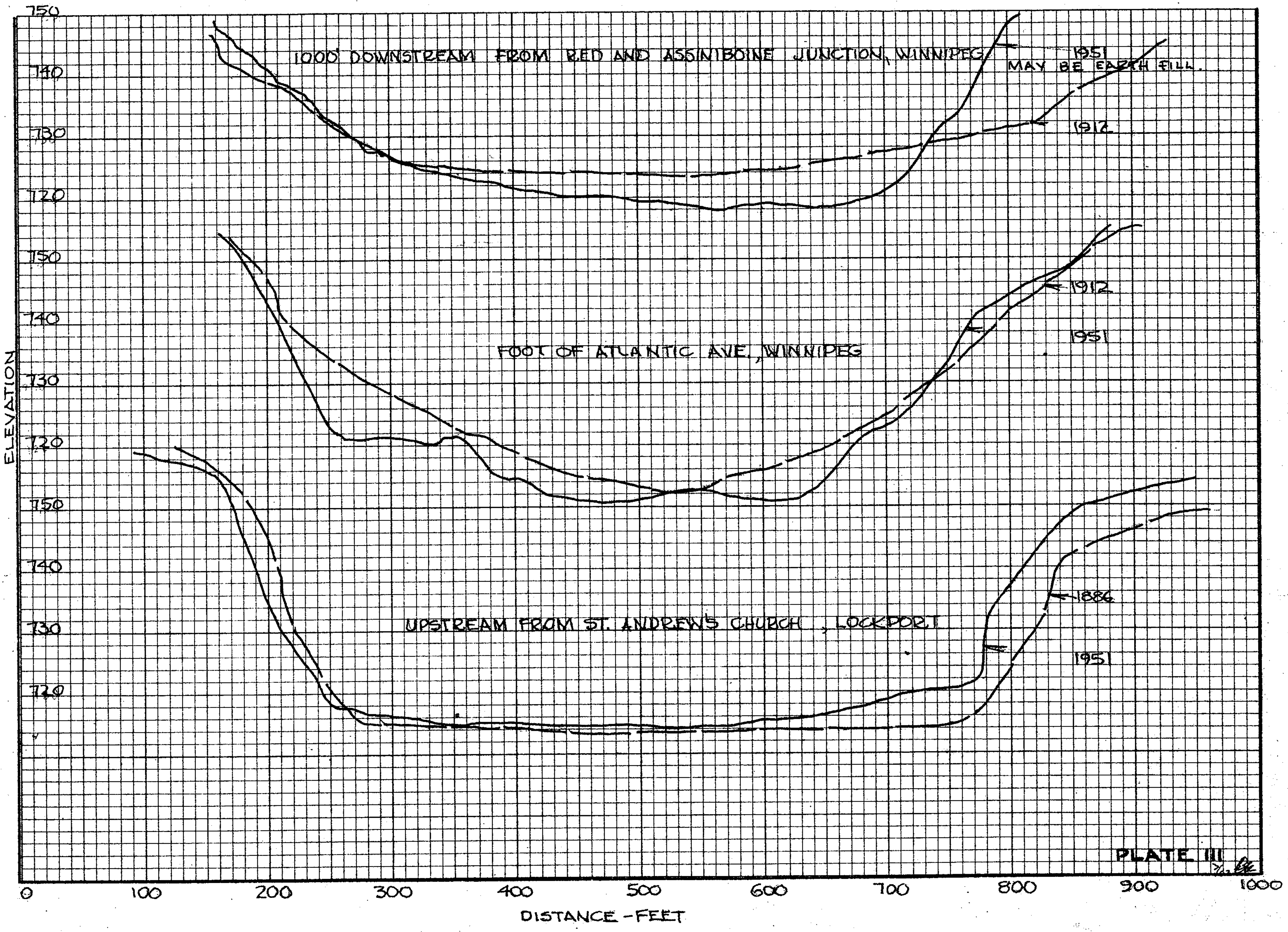

50' UPSTREAM FROM JUNCTION OF LA SALLE AND RED RIVERS
400' UPSTREAM FROM INTERSECTION OF OSBORNE ST. AND ASSINIBOINE RIVER
1951
1912
1951
1912
750
740
730
720
750
740
730
720
ELEVATION
0
100
200
300
400
500
600
700
800
900
1000
DISTANCE - FEET

PLATE 11



1000' DOWNSTREAM FROM RED AND ASSINIBOINE JUNCTION, WINNIPEG
1951
MAY BE EARTH FILL.
1912
FOOT OF ATLANTIC AVE., WINNIPEG
1912
1951
UPSTREAM FROM ST. ANDREWS CHURCH, LOCKPORT
1886
1951
750
740
730
720
ELEVATION
0
100
200
300
400
500
600
700
800
900
1000
DISTANCE - FEET

PLATE III

survey was made by the author regarding extent of river bank losses. Plate IV page 13, shows the questions placed before many riparian inhabitants, whose periods of residence on the river banks varied from 1 to 30 years. On the concave bends of rivers property losses varied between 0 and 100 feet over the period of record. The graph on Plate V page 14, shows the results of the survey. While points on the graph are considerably scattered, the projection of a line as shown will give some representation of the rate of property lost over an extended period of time. It can be seen that substantial losses of river bank property have occurred. Since the above mentioned report states, and the accompanying cross sections show that up to 1950 only a small shift has occurred in the cross sections, the possibility exists that the large losses sustained have been experienced since the 1950 flood, a period of 11 years. This could indicate that the major flood created a weakness in the overall bank structure and when further aggravation occurred such as a high water period (spring runoff), toe erosion, or that caused by man, the banks yielded. Since aggravation by man has been partially stemmed and since toe erosion is a natural and perpetual occurrence, bank failures may be due largely to smaller floods which have been and which are being experienced subsequent to the major 1950 flood. For the sake of a specific definition, therefore, it is stipulated that when the rise of

The University of Manitoba

STUDY OF RIVER BANK IMPROVEMENT IN THE WINNIPEG AREA

Statistical Survey of Riparian Landowners

(1) Extent of property lost to erosion or slides ...... ft.

(2) Period of time during which this property was lost .. yrs.

(3) At what value would you place the lost property-..........

(4) Should you be reimbursed for the lost property? ... .

(5) Would you object to large scale bank improvement even if it meant altering up to 15 ft. of your property? ....

(6) Would you agree to increased taxation to cover bank improvement?....................................

(7) Who should pay for the improvement?

Federal Government .... %

Provincial Government .... %

Metro .... %

Owner .... %

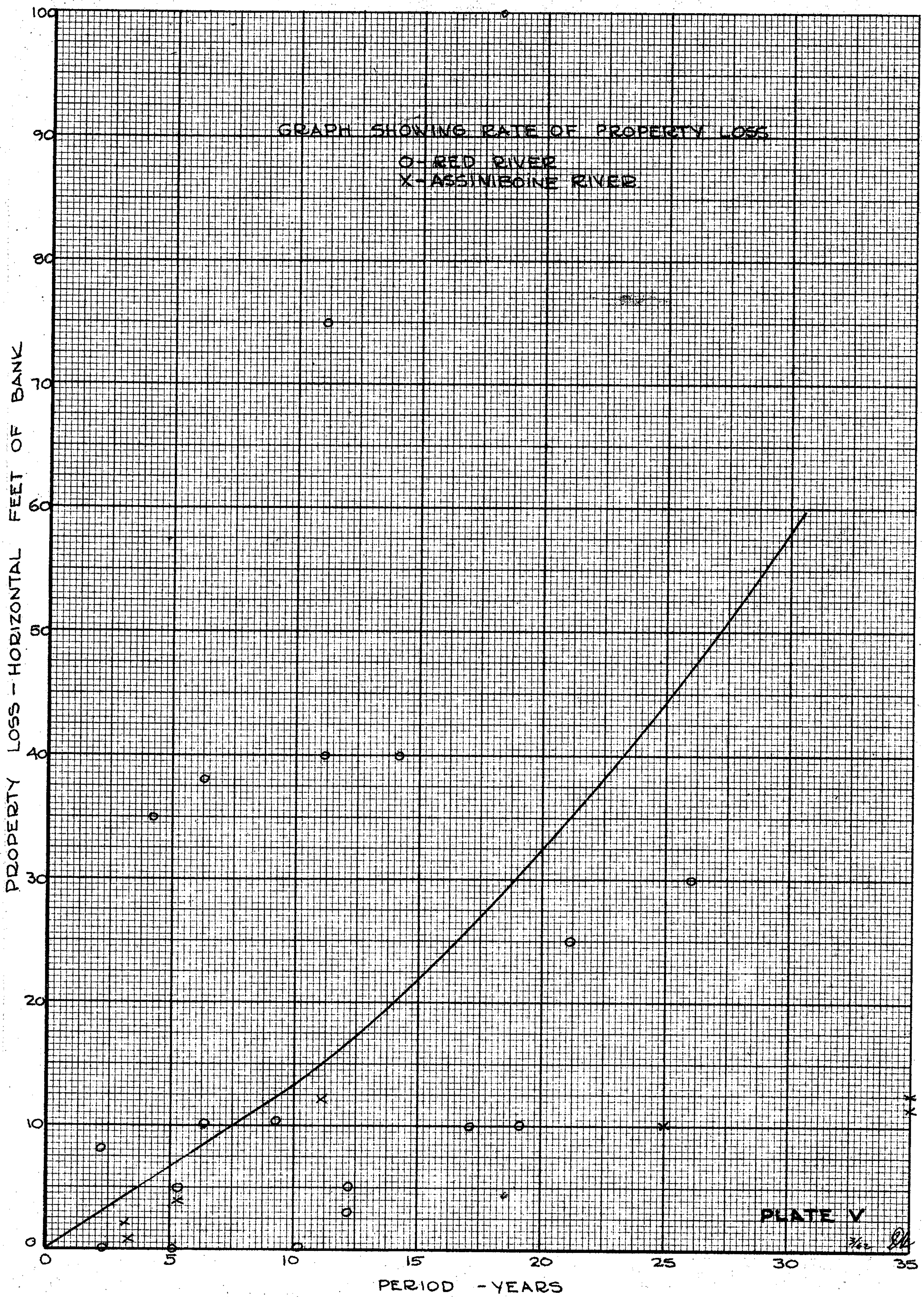
Assiniboine River
Red River

Address ....

Winnipeg, June 1961

E.H. Klassen, P. Eng.

PLATE IV


GRAPH SHOWING RATE OF PROPERTY LOSS
O - RED RIVER
X - ASSINIBOINE RIVER
PROPERTY LOSS - HORIZONTAL FEET OF BANK
PERIOD - YEARS
0
10
20
30
40
50
60
70
80
90
100
0
5
10
15
20
25
30
35
PLATE V

water in the spring reaches the level of the 1950 flood stage, extensive bank failures may result in the ensuing years.

CHAPTER II

PROBLEM OF EROSION AND INSTABILITY

A. Geography

"The land in the vicinity of Winnipeg, whose topography had experienced certain changes during the glacial era, was uncovered with the draining of Lake Agassiz. During the pre-glacial period the surface of the watershed was probably rough and the drainage well defined. However, when the ice moved south of the region, the glacial erosion levelled off divides, and filled up depressions and valleys, bringing the surface to a more uniform contour than before the Ice Age. The superficial material that had overlain the bedrock was plowed up and worked over by the slowly moving ice sheet. As the ice disappeared, this glacial drift was deposited over the basin as a heterogeneous mass of rock flour, clay, sand, gravel and boulders, termed "till". The thickness of this till on the bed of Lake Agassiz is 300 feet in some places and buries the pre-glacial, minor rough features of topography, not obliterated by glacial erosion.

Unmodified drift, also referred to as till or boulder clay, constitutes the greater part of the entire sheet of sub-glacial deposits. This unmodified drift usually lies on the striated bedrock and is covered by modified drift or the stratified gravel, sand, silt, and clay, deposited by streams which flowed down from the melting ice sheet or by lacustrene and fluvial deposits. The till or unmodified drift is a direct deposit of the ice sheet as shown by the indiscriminate mingling of rock flour, silt, sand, gravel and boulders. Its upper portion is commonly soft and easily worked, while below there is a sudden change to a hard and compact deposit. The probable cause of this difference in hardness was the pressure of the vast weight of the ice sheet (about one mile thick) on the sub-glacial till, while the upper part of the till was contained in the ice and dropped loosely at its melting.

In the vicinity of Winnipeg, the unmodified drift referred to locally as "hardpan" and consisting of silt, rock flour, sand, gravel and boulders, forms a very uneven surface and varies in thickness from a few feet to possibly 50 feet. It may be located at depths varying from 4 feet to 60 feet below the ground surface. This drift is generally overlain with lacustrine deposits of highly plastic clays, which have the capacity to hold large quantities of moisture, and are subject to considerable swelling and shrinking with change in moisture content. Limestone bedrock is close to the surface at several locations north of Winnipeg, that is at Lower Fort Garry and Lister's Rapids, 5 miles downstream from the north city limits.

Where areas have been affected by changes in the course of the river or the tributaries, the old course has been filled with fluvial deposits of silt and sand, to an elevation somewhat lower than the surrounding terrain. Thus low lying areas, susceptible to flooding were formed. Several such areas, which have been developed as residential areas, are Wildwood, Riverview, Norwood and a section of East Kildonan." (4)

B. Geomorphology

Geomorphology deals with that department of physical geography which comprises the form of the earth, the general configuration of its surface, the distribution of land and water, and the changes that take place in the evolution of land forms. In the following paragraphs the subject will be restricted to the rivers, and their adjacent areas, in the vicinity of Winnipeg.

C. Transport of Sediment

"Sediment is transported by a river as bed load and as suspended load. The bed load is comprised of relatively coarse material, and rolls, bounces and saltates near the bottom of the river. The suspended load is composed of relatively fine material and is kept in suspension by the turbulence of the river. Actually there is no sharp distinction between the two forms of sediment transport. The distribution of suspended sediment tends to vary with the velocity distribution and with the turbulence of flow. The finer particles may be fairly evenly distributed over a vertical plane, whereas the coarser particles usually have their largest concentration at the lower levels of flow. With an increase in the turbulence of flow, the distribution of all suspended sediment fractions tends to become more even. Also, transport of bed load is some function of the tractive force which the flowing water exerts on the periphery of the channel and the amount of sediment in suspension depends primarily upon the turbulence of the river. The tractive force is the component in the direction of flow of the weight of the water and can be represented by W.D.S., where W is the unit weight of the water, D is the depth of flow and S the slope of the river." (5)

From the stated relationships it can be concluded that the sediment transport will in general increase with an increase in depth and an increase in river slope. Some indications of increased sediment with increased depth were observed at locations where a deposit of up to 3 feet of sediment were deposited after the spring runoff.

The Red River at Winnipeg is a meandering river, and contains alluvial soils and wide flood plains. The following is a study by Professor Kuiper on the meandering of rivers. The study, despite its length, has been included here since it applies well to local conditions.

"Because of the meandering, the river periodically turns over the soil in the meander belt to a depth which is roughly equal to the depth of the river. The river cuts away the concave banks and builds up the convex banks. The latter deposits are composed of rather coarse material at the bottom and finer material at the top. Over an extended period of time, through the continual shifting of the river channel, these deposits are situated away from the active streams and are continuously being covered with deposits of silt and clay from occasional flood overflows. A boring taken at such a site may indicate 10, 20, or 30 feet of stratified alluvial deposits. An explanation of the reason for this is as follows:

It can be seen, with the use of a flow net, that the hypothetical flow of frictionless water around a bend will produce the highest velocities near the inside of the bend. In nature, there is no frictionless flow and it will be found that there is a certain velocity distribution over the cross section of a straight channel, with the highest velocities in the middle near the surface and the lowest velocities near the perimeter. As a result of this difference in velocities, the fast surface water, because of its inertia, will move towards the outside of the bend and consequently, the slow boundary water will have to move towards the inside of the bend. This second effect of the highest velocities accumulating near the concave bank offsets the first effect of the highest velocities being produced near the convex bank. Whether the one effect will dominate over the other will depend on the vertical velocity distribution. A river cross section may have a width of 10 to 20 times its depth. In the middle part of such a cross section a pronounced decrease in velocity from surface to bottom can be expected and therefore a pronounced movement of the high velocities towards the outside of the bends. It takes, of course, some distance for the water to reach the concave bank; hence it can be expected that at the beginning of the bend, the highest velocities are near the convex bank on account of the flow net principle, and at the end of the bend near the concave bank on account of the inertia principle. Due to inertia of the fast flowing water, the high velocities will remain for a while near the concave bank before returning to mid stream. In conjunction with the above described flow conditions, the movement of sediment, plays an important role in the formation of river bends. The "wash load" or finer particles in suspension, whose concentration depend on availability rather than flow conditions, has practically no significance in stream channel stability and formation of bends, while the bed-load or coarser sediment, which does depend on flow conditions, plays a leading role.

While the wash-load will move through a bend without getting a chance to settle at any place, the bed-material load which is in suspension will partly settle out near the lower part of the convex bank where the velocities tend to be consistently lower than average. The bed-material load that rolls and saltates over the river bottom will tend to move towards the convex bank for the same reason that causes the spiral motion. The spiral motion may be explained as follows:

In flowing around a bend, the river has a transverse gradient, sloping upwards towards the concave bank. This gradient is consistent with the mean velocity of the water. Water particles that have a higher velocity tend to move towards the concave bank. Water particles that have a lower velocity tend to move toward the convex bank. If a hollow tube were laid on the bottom of the river across the middle of the bend and held there, it would have a constant flow of water from the concave bank towards the convex bank. In the same way, the transverse gradient will act with a force towards the convex bank, upon any body of material that is lying still upon the bottom of the river. Hence a grain of bed-material that is not in motion is acted upon by the following three forces; one force, caused by the transverse gradient, acting toward the centre of the bend; another force, caused by the flowing water, acting in the direction of flow; a third force, caused by gravity, acting vertically downward. The last force has a component that must be taken into account when the surface of the bed is not horizontal.

After being deflected over a certain distance, the particles will come into the region of low velocities near and downstream of the convex bank and some of them will be deposited. This will continue until such a side slope is built up that the gravity component balances the transverse gradient and spiral flow.

In addition to some deposition of bed load, there will also be deposition of suspended bed material load near and downstream of the convex bank, due to the local decrease in velocities. Since the bed load is in many meandering rivers only a fraction of the suspended bed material load, the latter form of deposition may be the more important of the two. There is, of course, a limit to the deposition on the convex bank. If the concave bank would not erode, deposition would decrease the cross section of the river and therefore, increase the velocities and side slope until a state would be reached whereby no further deposition could take place. In a way, this is what happens in the formation of the proper channel shape in a straight stretch of river. The shape becomes such that no further erosion can take place with the existing velocity distribution. However, in a meandering river, the concave bank is usually composed of erodible material, hence deposition on the convex bank can continue to take place. It follows that the rate of erosion and the rate of deposition are interrelated. A river with cohesive banks or with little transportation of bed material will have a slower progress of its meander than a river with erodible banks or a large transportation of bed material. Since in an alluvial river both the composition of the banks and the transport of bed material are a function of the sediment characteristics of the river, it would seem logical to state that the rate of meander progress is also a function of the same. This would seem to eliminate the old controversy as to whether erosion of the concave bank is the initial cause which permits deposition on the convex bank or whether the growth of the convex bank causes the concave bank to erode.

From laboratory experiments and field observations, it is found that eroded material from a concave bank has little opportunity to be deposited on the opposite convex bank. Instead, it moves downstream and is deposited mostly on the first convex bank downstream, that is, at the same side of the river. This is probably because the most severe erosion takes place over the lower part and below the bend of the river, whereas the transverse gradient exists in the bend only. Moreover, it takes a certain length of travel before a particle is deflected appreciably. Here then lies the reason for the meandering of the river and the resulting conditions of its banks. Meandering results primarily from local bank erosion and consequent local overloading and deposition by the river of the heavier sediments which move along the bed. Meandering is essentially a natural trading process of sediments from banks to bars. The rate of trading depends on the rate of bank caving. In uniform materials and on a uniform slope, a series of uniform bends will develop. The radii of bends increase with increase in discharge or slope, the size and shape of bends depend upon the alignment of the flow into the bends. Cross sections of a meandering river are deeper along the concave banks of bends because of the impingement of the flow against these banks. The depth of the channel of a meandering river depends upon the resistance of the banks to erosion. Resistant banks result in deep cross sections and easily eroded banks result in shallow cross sections. Every phase of meandering represents a changing relationship between three closely related variables: the flow and the hydraulic properties of the channel, the amount of sediment moving along the bed, and the rate of bank erosion. These three variables constantly strive to reach a balance but never do, even with a constant rate of flow. Distorted bends and natural cutoffs are caused by local changes in the character of the bank material.

In general, there is a certain relationship between the meander pattern on the one hand and the discharge, sediment load, and slope of the river on the other hand. An extremely mild slope of the river indicates a sediment load that consists mostly of clays and fine silts. As a result, very little deposition on convex banks will take place, whereas the opposite concave banks are highly resistant to erosion due to their cohesive nature. Hence, the river will become practically a stable channel.

There are several factors which influence the rate at which meandering takes place. It is evident that the erodibility of the banks plays an important role, The looser the material, the faster the meandering will take place. Another factor of importance is the total amount of bed material that is transported by the river. A large load will result in active sedimentation of convex banks and therefore in active erosion of concave banks. In many textbooks on geology and geomorphology the activity of meandering rivers is presented as one of ever widening meander loops, culminating in cutoffs between adjacent meanders, once they become too large. That is, each cutoff initiates a new meander which culiminates in another cutoff. Following the analyzing of river behaviour in recent laboratory experiments and field studies, it is presently believed by most river engineers that the above described meander development is rather the spectacular exception than the common role. In the normal process of meander development, the meander banks migrate downstream at practically the same rate, more or less maintaining the same shape, without developing cutoffs." (6)

In metropolitan Winnipeg the Assiniboine River has at present one dangerous potential cutoff while the Red River has three locations which might be considered as potential cutoff bends. These form about 15 percent of the total number of bends in the Winnipeg area.

"In many cases, the progress of the upstream arm of a bend may overtake the slower progress of the downstream arm of the same bend because of nonuniformity of bank composition. One reason for this nonuniformity may be the fluctuation in flood flows which causes differences

> in gradation of sediment and deposition of this sediment at different places. Also, variations in vegetation in the meander belt cause variations in alluviation. The formation of an oxbow which is likely to be filled with coarse sediment near the place of cutoff and with clays and silts over the entire loop length, is bound to disrupt the orderly migration of subsequent meanders." (7)

D. Hydraulics

The Red River has cut a sinuous path within relatively straight so called meander belts. The meander belt of the Red River is about 1.5 miles wide in the southern portion of Winnipeg and narrows somewhat in the northerly portion of Winnipeg. The belt of the Assiniboine is approximately 2,000 feet wide in Winnipeg. The terrain of these belts is relatively flat except where the river banks drop about 30 to 50 feet to the river bed. The depth of the rivers has been somewhat limited by the underlying firm glacial till or bedrock which have not appreciably eroded. Between control points up to several feet of soft silt and other fluvial deposits may be found. Examples of control points occur, for example, on the Assiniboine River, near St. James Bridge, and at Lister's Rapids, downstream of Winnipeg. At St. James Bridge the river bed is composed of so called "hardpan" while at Lister's Rapids there exists an extrusion of bedrock. Professor A. Baracos states the following on the condition of the Red River in Winnipeg:

> "The rivers are subject to spring flooding with changes in level of over 30 feet occurring during major floods and frequently up to 18 feet. Drawdown to low winter levels

following spring high water is delayed until autumn by a control structure at Lockport, downstream from Winnipeg. River velocities are generally low except in times of flood where for example, peak velocities of the Red River range between 5 and 6 feet per second. Changes in the course of the river, are indicated by deposits of fluvial material on the convex banks and by instability of the concave banks. The extent of the unstable banks is evident when a field inspection is made. Practically every concave bank has either been affected by an old slide, is presently sliding or is showing signs of potential sliding, where stabilization has not been instituted." (8)

The problem of stabilization is therefore, a major undertaking. Raymond H. Haas states:

"Stabilization of a river is not wholly a matter of applying hydraulic formulae nor is it a matter of adapting design that has proved successful elsewhere. The purpose, basically, is to so control the flow that a favorable channel will be maintained where it is now adequate, from the view point of current impingement upon bank, and to reform the channel in places where it is unsatisfactory. The main difficulty of stabilization is the instability of the river itself. In its natural state we find a wide variation in depth, velocity, volume and direction of flow, as well as minor variations in width and slope. To this we may add that the local slopes and direction of flow continually change with each change in stage and each adjustment of the bank and crossing. Near the concave bends of the river, "pools" tend to form because of the increased scour caused by direct attack of currents. At low water the surface slope in the "pools" is less than across the shoals or "crossings", but at higher stages the slope in the "pools" becomes relatively greater and at flood levels the difference in surface slope between "pools" and shoals is largely obliterated. In straight reaches it is not unusual to find at low water that slopes directly across the reach are greater than for an equal distance along the thalweg. The gradual increase in surface slope of the pools, which occurs when the discharge is greater, produces a greater hydraulic radius and a corresponding increase in the mean velocity. Similarly, the influence of the reduced slope over the crossings is to decrease the average velocity there, with a resulting tendency to raise the crest elevation of the crossings. As the higher stages subside the opposite effects are produced and the river returns to approximately its original condition, although the lag sometimes leaves the crossings considerably higher than before the stage increase. The crossing may also scour at a different location due to local steepening of the transverse slopes during high stages, resulting in a change in the channel." (9)

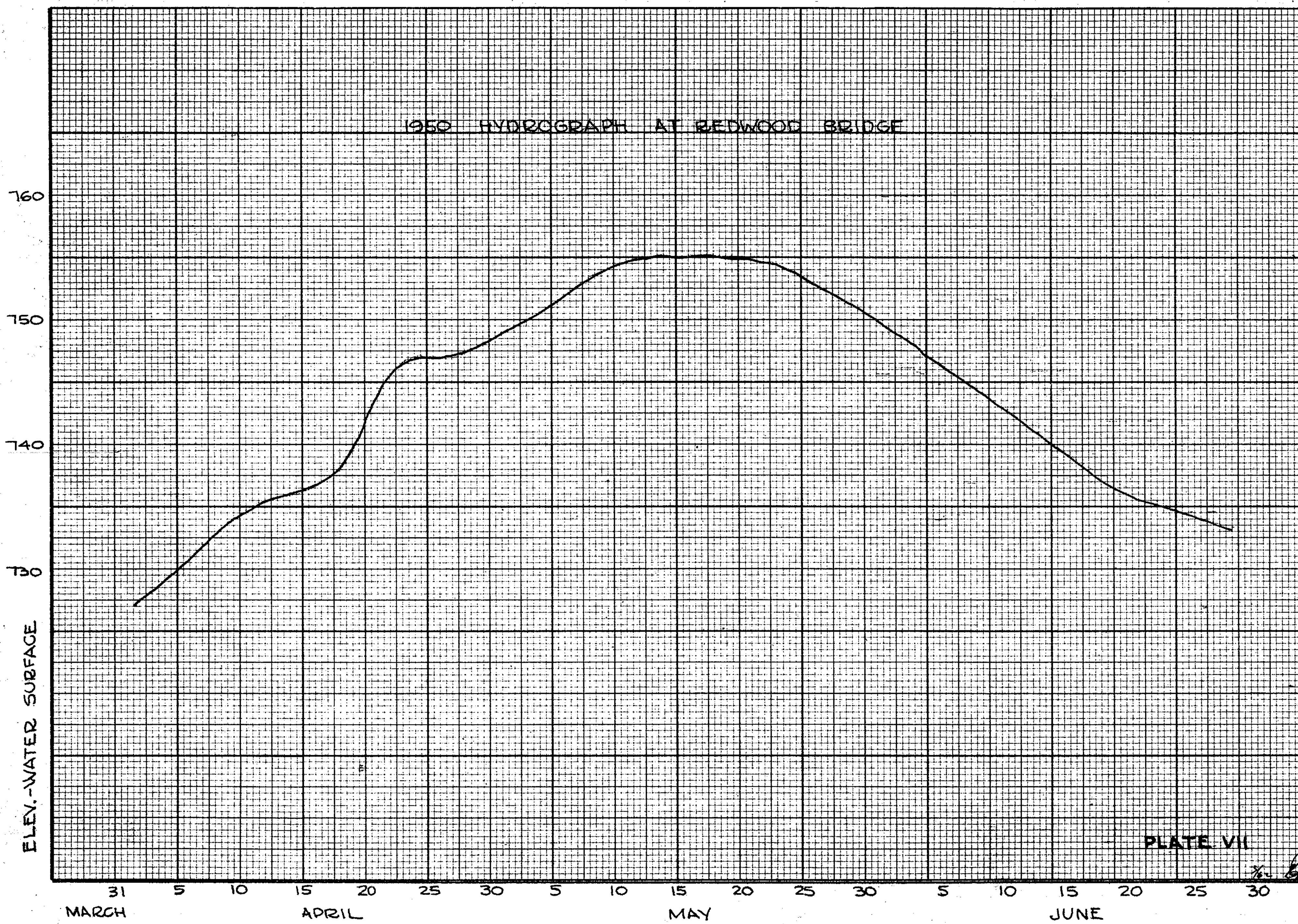
This may cause local toe erosion, which in turn may create in evolutionary fashion, more severe conditions. E. G. Stephens' experimental work has clarified much in connection with river bends. He states:

"Further, in connection with curves and crossings, it has been found in laboratory work that bends of shorter radii produce the greater attack on the banks and any stabilization works thereon, because bends of short radii tend to produce excessive depths within the bend as compared to bends of greater radii for the same location. Channel depth in the bend is also affected by the direction of flow into the bend. If the entrance of the flow into the bend produces a direct attack on the concave bank it has the same effect as a sharp bend. Accordingly, the channel depth in the bend is in some direct proportion to the entrance flow angle. The more direct the attack, the greater the depth. The entrance of the flow into the bend at about the same location and direction during both low and high water flows is also very important. The most favorable results towards stabilizing the location of the crossing from one bend to another have been accomplished by what is known as "hooking" the crossing. This is affected by making the bends an open spiral in lieu of true curves, with the shorter radii at the lower end of the bend. Over-hooking should, of course, be avoided and sometimes the amount of hook is so small that it cannot be observed in an actual field layout.

Bends of too great length also contribute to the development of excessive channel depths especially during periods of prolonged high water stages. The reason for this appears to be that during high flows, the current cuts across the bends, diverging widely from the course of the low water flow. The low water channel around the bend becomes silted up to some extent, thus when the river stage falls and the low water flow attempts to return to its old channel, its course is restricted and the flow is concentrated immediately against the concave bank. Unprotected banks in such cases are characterized by more rapid erosion. Protected banks receive severe attack against the revetment structure.

It has long been know that erosion of banks in concave bends becomes the most serious just following a high water period." (10)

While it is true that for the greater part of the year the stage of the river is held at a predetermined level, spring floods create conditions quite different from the normal conditions. Above are mentioned some of the problems of a varying stage. One other item of significance about a spring flood is the duration of the high flood waters. The hydrograph of the 1950 flood is shown on Plate VII page 28. From the hydrograph it can be seen that the water level remained at bankfull stage (753.1) for eighteen days. The length of time between normal summer water levels of 734.0 of the spring flood amounted to eighty days. During this whole period velocities were higher causing upper bank erosion to exposed banks. Also, an important factor in many instances is wave action caused by wind. Exposed banks, observation has shown, are very susceptible to erosion by wave action despite the fact that waves seldom reach extensive heights. Wave and erosive action over an extended


1950 HYDROGRAPH AT REDWOOD BRIDGE
760
750
740
730
ELEV.-WATER SURFACE
31
5
10
15
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25
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5
10
15
20
25
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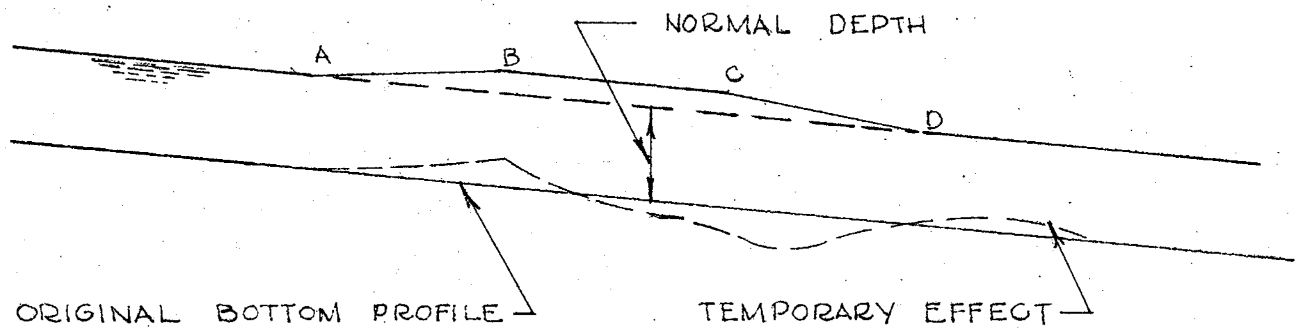
5
10
15
20
25
30
MARCH
APRIL
MAY
JUNE

PLATE VII

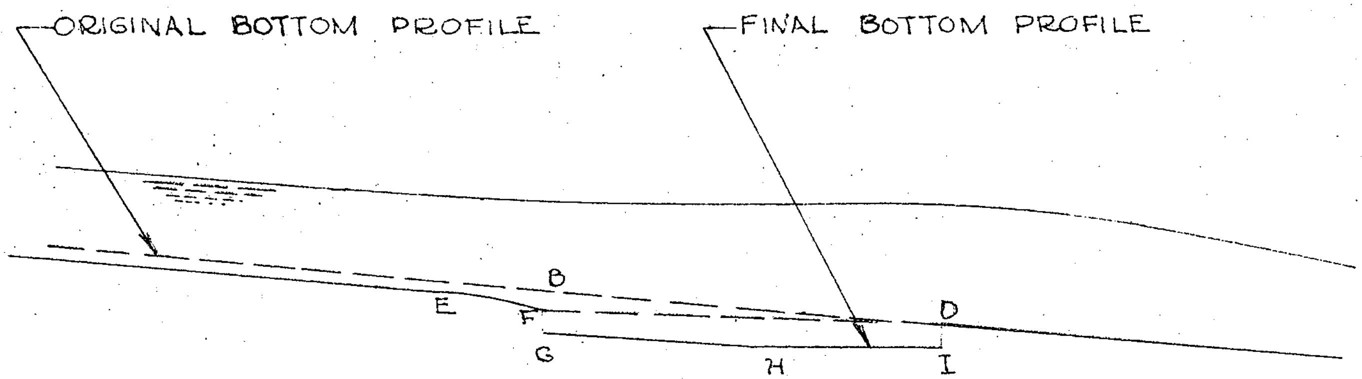
period of time can cause at least local damage to banks, if not general damage to the overall bank system. Local damage and alteration to banks almost inevitably affects downstream banks in some way and to a variable extent. All through the various flood stages, debris, to a small extent, and ice, in a more severe manner, aggravate the river bank conditions. Firstly, the constant grinding of ice and debris against the exposed portions of banks loosens and removes sediment extensively. Secondly, the action of ice, where banks are not already exposed, is to expose those stretches of banks, leaving them open to the action of wave and current. Very often, willows and shrubs, which got a strong foothold the previous summer, are gnawed off by the ice and swept downstream.

Stabilizing the banks in such a manner that the cross section is narrowed causes deepening of the channel. Professor Kuiper suggests the following explanation:

> "In Plate VI Figure A page 30, the initial result will be a rise in river levels between B and C, backwater between A and B, and drawdown between C and D. The temporary effect on the river bottom will be deposition of sediment between A and B, mild scour between B and C, severe scour between C and D and deposition downstream of D, due to the overloading of the river. The final effect is shown in figure B. The original bottom profile is represented by the line B - D, the slope has become milder due to the increased efficiency in sediment transportation. The change in slope is from B - D to F - D. The change in bottom profile is from B - D to G - H - I. This is because the narrowing of the river by



A



B

PLATE VI

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bank protection works was also compensated for by a deepening of the river. Between H and I, the bottom slope decreases due to the local drawdown effect. Upstream of the regulated stretch, the slope remains the same but the bottom profile is lower. Between E and F, there is a local increase in bottom slope, due to the backwater effect. The explanation has usually been that this was due to the saturation of the material in the bank by submergence of the bank during the high water period. (11)

E. G. Stephenson states:

"There is a substantial amount of proof, however, that the increased erosion is due to the wide divergence of the high and low water flow courses. It is a sound theory that the minimum amount of erosion or the minimum amount of attack against a stabilization structure is obtained when the low and high flows follow the same curve. Bend lengths that change the direction of the channel more than 120 degrees should receive special consideration to ensure that local ground conditions will not permit a major amount of the flow cutting across the bend at high stages." (12)

E. Soils

Soils in the Winnipeg area are described by Professor A. Baracos as follows:

"The soils in the Winnipeg area consist of highly plastic varved clays of glacial lake origin. In sequence of increasing depth, they occur as a brown layer locally known as "chocolate" clay, an intermediate layer not always found of brown and gray clay known locally as "mixed" clay, and an underlying generally somewhat siltier gray layer known as "blue" clay. It should be noted, where they occur in river banks, they generally show some or considerable disturbance and lower unconfined strengths. The more recent siltier and organic deposits of 2 and 16 feet thickness cover the glacial lake clays, which due to their shallow depth do not enter into stability calculations other than as an additional weight on the bank which must be considered.

Underlying the varved clays, a glacial till consisting of silt, rockflour, sand, gravel and boulders is usually encountered. The upper few feet of this material are often soft having unconfined compression strengths of under 1,000 pounds per square foot. This soft material in many areas is mixed with the overlying few feet of gray clay. Below the upper soft portion, the till is generally found in a very dense and cemented condition and is known as "hardpan". Occasionally no glacial till is encountered and the glacial clays are underlain by limestone bedrock. An exception to the general occurrence of the varved clays is in areas where the river has meandered and river deposits have formed. Such areas are, of course, found at convex bends of rivers. As may be expected these show considerable variation in soils which consist primarily of clayey or sandy silt with occasional sand or clay layers. The predominantly silty soils show little cohesion and a relatively small angle of internal friction in undrained triaxial tests in terms of total stress. They are often underlain by a few feet of the varved clays or in most cases by bedrock or "hardpan". Shells, wood fragments and other organic material are often found in the fluvial deposits. With regard to consolidation lacustrene clays show appreciable over-consolidation only in the shallower depths where desiccation has apparently occurred.

Groundwater conditions have been found most difficult to determine in the varved clay areas. Based on consolidation test data, these clays have permeabilities in the order of 10^{-9} to 10^{-11} centimetre per second. Free water is often not encountered until the glacial till or bedrock has been penetrated, at which time the water may rise 15 to 20 feet in a matter of minutes, otherwise the test holes may remain dry indefinitely. It may be noted that almost the entire depth of clay with the exception of the top 6 to 12 feet generally show complete saturation indicating a very substantial zone of capillary rise. Free water is also found at times in thin silt layers overlying the varved clays. Fissures in the upper layers of soil readily permit surface water to percolate into these silt layers. The amount of free water is generally small and prolonged dry weather results in its disappearance." (13)

From a soils mechanics aspect, the most significant factors in river bank instability are "sliding" and creeping". Dr. Terzaghi defines a slide as:

> "A rapid displacement of a mass of rock, residual soil or sediments adjoining a slope, in which the centre of gravity of the moving mass advances in a downward and outward direction.
>
> Regardless of the type of slide, the slide movement occurs where the driving force exceeds the resisting force. The driving force consists of gravitational and seepage forces and hydrostatic pressures; and the resistance to sliding is the shearing resistance of the soil mass. In a river, the water would tend to resist sliding by creating a hydrostatic head against the adjacent bank. Preventation and correction of slides involves either reduction of the driving force or increasing the resisting force or both." (14)

There are two major external causes which produce slides. The first is the steepening or heightening of slopes by river erosion or man-made excavation. Also, the deposition of material along the upper edge of slopes often may cause slides. Such external causes produce an increase of the shearing stresses at unaltered shearing resistance of the material underlying the slope. These causes have been discussed sufficiently in above sections.

The effects of ground water are described by Dr. Karl Terzaghi:

> "If a slope fails in spite of the absence of an external cause, it must be assumed that the shearing resistance of the material has decreased. The most common causes of such a decrease are an increase of the pore water pressure and progressive decrease of the cohesion of the material

adjoining the slope. The popular concept of moisture causing slides by "lubrications" of soils is considered somewhat fallacious. Groundwater acts in two ways to induce slide movement; first the weight of a soil mass increases somewhat as the soil becomes wet, thus adding to the driving force tending to cause movement. Second, and of greater importance, the presence of water results in a reduction in shear strength of the soil. In the case of cohesive soils the cohesion decreases with increase in moisture content. In addition to these factors groundwater movement may result in seepage forces inducing sliding movements. Water in undoubtedly the most important single factor contributing to sliding movement.

Seepage forces affect the stability of existing slopes in four ways:

They reduce the shearing resistance of the ground by raising the pore water pressure.

They eliminate apparent cohesion produced by the surface tension in drained soil.

They eliminate real cohesion by removing cementing materials in solution, where such materials exist.

They may cause slope failure by retrogressive underground erosion by water veins emerging at the foot of the slope." (15)

Seepage water, originating either from a natural source (rain or flood) or from man-made source (lawn sprinkling, etc.) enters the ground through dry weather or frost cracks or through scattered permeable layers of soil. Some failures take place during periods of heavy rainfall or in spring when the snow melts. However, rain or melting snow belong to the normal existence of a slope. Hence if a slope is very old, heavy rainstorms or rapidly melting snow can hardly be the sole cause of a slope failure. It seems unlikely that they are without any precedent in the history of the slope. Seepage, however, can be considered an important contributing factor. These then are some of the internal causes of slides.

Intermediate between slides due to external and internal causes are those due to rapid drawdown, subsurface erosion and spontaneous liquifaction. Rapid drawdown refers to the lowering of the water level in a storage reservoir or to the descent of the water level in a river after a flood, (or locally due to opening of the locks) at a rate of several feet per day or even less. The effect of this process on the stability of slopes forming the sides of the river is illustrated on Plate VIII page 36. The following analysis is based on the assumption that the voids of the soil are completely filled with water both below and above the piezometric surface. The effect of capillary forces on the stability of the slope is here disregarded.

When a high water condition has existed for some time, the river water level and the piezometric surface in the river bank are at approximately the same elevation. A slow descent of the water surface will allow drainage of the soil at the same or at a somewhat slower rate than the drop of river water level. The more impermeable the soil and the more rapid the drawdown the farther the lowering of

EFFECT OF RAPID DRAWDOWN

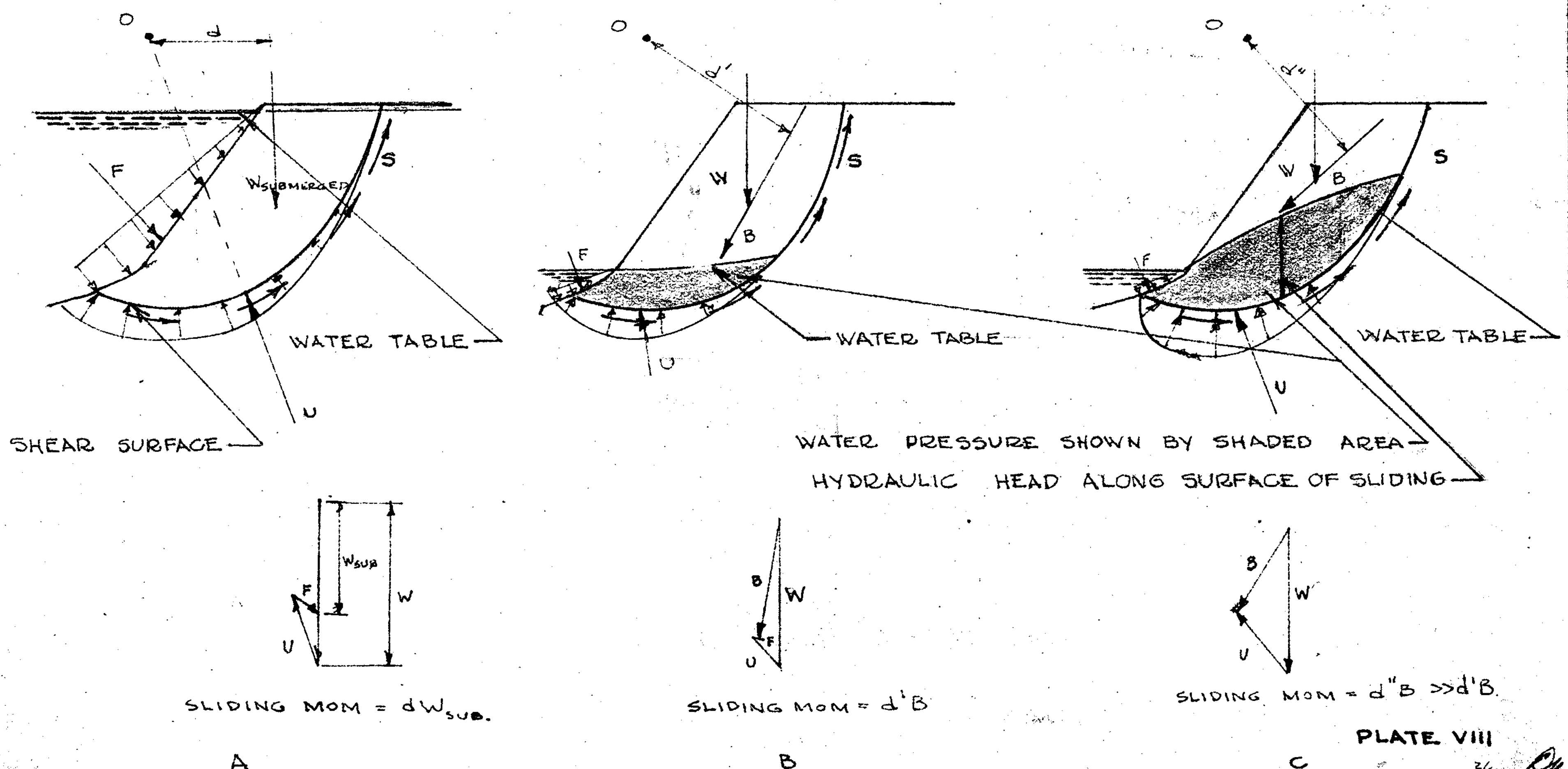

PLATE VIII

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the piezometric surface will lag behind the river drawdown. (See diagrams 1 and 2 Plate VIII). When drawdown takes place very rapidly the descent of the piezometric surface lags far behind the descent of the free water level and at the end of the drawdown the piezometric surface rises from the foot of the slope as shown in diagram 3 Plate VIII. At this point the bank is under the force of a hydrostatic head and, if a certain number of the other, above mentioned conditions exist, a bank failure is imminent.

Subsurface erosion and spontaneous liquefaction, while being causes of slides, are not generally applicable in the Winnipeg area. They occur, normally in areas where coarser grained soils, such as sand and silt occur. The main concern here is the fine grained clay.

Aside from slides, there exists another type of bank failure, which, while not occurring as rapidly as bank slides, will eventually result in the same type of damage. This type is known as soil creep and occurs on steep clay banks. It may often be the result of internal forces. As a rule the movement is slow and is destructive only where the soil involved in movement supports a fill, roadway or structure. Retaining walls alone may serve no useful purpose because the soil can creep to, and move with the wall. Creep is also a postslide occurrence. If a slide causes disintegration and breakdown of non thixotropic sediments such as glacial tills during a period of heavy rainfalls, the slide material is likely to creep very actively for many years and every new rainstorm accelerates the creep. With lacustrine clays this may not be the case. When a slide comes to rest its factor of safety against sliding or creeping may again be unity or greater and the slide may appear to be stable. Most slides do not occur instantaneously as Dr. Karl Terzaghi states:

> "It is important to realize that slides do not occur without warning. No slide can take place unless the ratio between the average shearing resistance of the ground and the average shearing stresses on the potential surface of sliding has previously decreased from an initial value greater than one, to unity at the instant of the slide. The only slides which occur almost instantaneously are those due to earthquake and instantaneous liquefaction. All others are preceded by a gradual decrease in the ratio which, in turn, involves a progressive deformation of a slice of material located above the potential surface of sliding and a downward movement of all points located on the surface of the slide." (16).

A short report has been written by Dr. A. Casagrande, Professor of Soil Mechanics, Harvard, stating his opinions of the river banks in the Winnipeg area. (This report was preliminary and expressed first impressions only.) Rather than summarizing the report, a copy has been included in Appendix D at the back of this thesis.

CHAPTER III

POSSIBLE ENGINEERING SOLUTIONS

A. Outline of Basic Principle

B. Drainage of Banks

C. Re-sedimentation

D. Structures

  1. Retaining Wall

  2. Revetment

A. Outline of Basic Principle

In this study two stipulations will be followed. First, an attempt will be made to evaluate the possibilities of providing complete bank protection in the entire Metropolitan Winnipeg area. Secondly, only such protection will be considered suitable, that will protect the banks against the worst possible conditions.

In studying various projects of even greater magnitudes at other locations it seems to be the generally accepted fact that the only practical approach is that of overall protection. E. G. Stephenson, Corps of Engineers, Omaha, states:

> "The success of the stabilization of banks of an alluvial stream is dependent upon starting the work at some stabilized point and progressively working downstream thereform. Stabilization of banks at isolated locations in a piecemeal manner has not met with other than temporary success. This

theory has been demonstrated by laboratory study and 50 years experience on the Missouri River Commission." (17)

Samuel A. Story of La Gloria Corporation states, after observation of the effects of stabilizing one bend in a river:

"The installation has been very effective. However, slight restriction created in the channel is causing changes in the river current, which are resulting in bank erosion just above and slightly below the installed works. This new erosion required the placing of additional revetment, from 2,000 feet to 2,700 feet, an increase of 35 per cent over the original cost." (18)

Charles Senour, a U. S. consulting engineer states the following:

"Where a large levee, highway or other expensive improvement has been endangered by bank caving, riparian interests frequently have revetted the bank or have built groins or training walls, and thereby have protected the areas immediately threatened. This often has contributed little toward the permanent stabilization of the river's banks because the installation of isolated and unrelated stabilization works ignores an important aspect of regimen previously noted herein, namely, the tendency of river bends to migrate downstream. As the result of this tendency, a caving bank in the course of time is blanketed by a sand bar, and the point of current attack moves downriver. If the erstwhile caving bank was revetted, the installation is no longer useful because it is no longer under attack, and the total mileage of the caving bank is the same as it was before the revetment was placed.

This has been a hard lesson to learn. There are miles of expensive bank revetment behind wide sand bars along the major alluvial rivers. It requires firm resolve to expend large sums to protect a river bank where there is no imminent threat to an expensive improvement. But the testimony of mile on mile of inactive revetment irrefutably points to the necessity of a comprehensive plan of control if the problem is ever to be finally solved." (19)

There are two possible reasons for believing that the above cases may not be equally applicable to the situation at Winnipeg. First, there is evidence that the amount of sediment in some of the southern rivers may be considerably greater than in the Red and Assiniboine Rivers. Secondly, the soil is generally somewhat coarser in the southern rivers, making the banks more easily eroded than local banks.

The southern rivers, and in particular the Missouri River, and the Red River have several important similarities, however. Firstly, they have approximately similar gradients. Secondly, they are both subject to major high water periods during spring runoff and consequently are both subject to rapid drawdown conditions following the spring floods. (Missouri flows are, of course, proportionately greater than the Red River flows due to a larger watershed). The profile of the water table adjacent to the two rivers is very similar, that is, beginning approximately at the water's edge and then rising as it proceeds away from the river's edge. Thus, the Red River and the Missouri River have similar flow fluctuations, similar gradients and, to some degree, similar subsurface conditions. In addition, they follow a similarly meandering channel through cultivated and, in many cases, eroding farmland, giving rise to their acquiring of sediment.

It will be considered, therefore, that the general principle evolved in the above references, that is, the improvement of the entire river bank system, in Metropolitan Winnipeg, is justifiable here. The process of erosion is definitely a major contributor to bank instability and it is the author's opinion that the conditions of the Red River are similar to those of the Missouri River, with the exception that the whole process of erosion and downstream migration of bends, on the Red River occurs at a slower rate of time than that of the Missouri River.

The second stipulation of this thesis is the following:

Only such protection that will protect the banks against the most severe condition possible will be considered advisable. Partial protection is considered to be uneconomical and impractical because of the frequency of occurrence of severe conditions in the Winnipeg area.

Solutions that cope with the problems of river bank stabilization must, of necessity, vary from location to location because of the differences in conditions that exist at the various locations. Certain types of problems have been alleviated at certain locations, for example, the Arkansas River, by adopting methods tried elsewhere, as on the Missouri River, but these methods have usually been altered to suit local conditions.

Basically, three methods of improving the river bank conditions in Winnipeg can be considered. The first method consists of the draining of the water from the river banks. Another method is that of creating extensive re-sedimentation conditions. largely at concave banks, so that there will be a build up of sediment here. This will have a tendency to increase the safety factor of the bank by adding weight to the toe, while at the same time it will decrease the slope of the bank. This type of improvement has two stages, namely, that of re-sedimentation and secondly, that of protecting the newly acquired sediment with some relatively light cover of asphalt, concrete or other material. Without the second state the first would in most cases be of no value.

The third method of improving the conditions is by erecting structures which will prevent a bank slide or which will prevent toe erosion or, as may be the more logical case, which will perform both tasks.

These three methods, in the following pages will be discussed briefly as to past experience in other areas and applicability in this particular area. The order of discussion will be; bank drainage, re-sedimentation and structures.

B. Drainage of Banks

Since most slides are partially caused by an abnormal increase of pore water pressure in the slope forming material or in a part of its base, radical drainage is indicated. The first requisite in a drainage study is the determination of the subsurface conditions and physical characteristics, by geologic survey and exploratory boring. The saving in ultimate cost of the corrective work will repay such preliminary work many times. For determination of moisture content and drainage characteristics under some conditions of sliding ground, soil samples should be taken of the full column over each entire area considered. In a concave bend, it is thought for example, soil samples should be taken at least at the toe of the slope and at the top of the bank and spaced at a maximum of 100 feet laterally. In addition to these locations, where serious bank conditions exist, samples should also be taken 50 feet back from the top of bank and 50 feet into the river from the toe. Ground water sources and the seasonal water levels and pressures should also be determined.

It is logical to assume that perforated drains, drilled and placed, will not serve the purpose well unless extensive use is made of lateral and "criss-cross" drains also. The soil is too impermeable for this process. The spacing required would be such as to make the project impractical and uneconomical. These drains function very well in coarser soil such as sand, and fairly well in silts, and have been widely used by the California Department of Public Works. They can not be considered in clays. Also excluded because of excessive expense are the electro-osmosis and vacuum methods of reducing pore water pressure. These methods require extensive study to determine their actual applicability in the Winnipeg area.

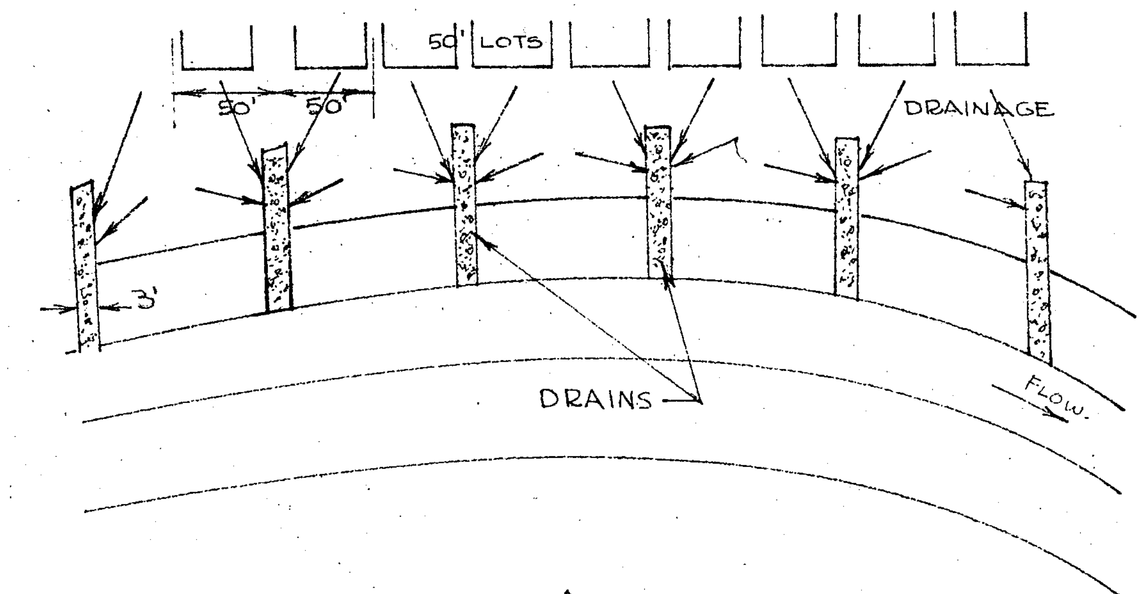
Two methods of bank drainage which yield good results in certain types of soils are toe trenches and inverted filters. These two systems are particularly useful in halting the effects of seepage from subsurface water veins. Insufficient knowledge as to the existence, location, and frequency of occurrence of water veins in the Winnipeg area eliminates the present study of these systems. It is assumed that they would be only a small portion of a comprehensive drainage system, should a practical system be evolved. From a study of the uses of above mentioned schemes in other areas, and the existing conditions in question, it is thought that any effort to drain the banks proper would not be a plausible solution to the problem for the above mentioned reasons. A well-known authority has stated in a recent report that:

> "Seepage control has been mainly limited to providing unimpeded surface drainage and that the clays generally cannot be drained." (20)

Surface drainage is an absolute necessity and would be a certain benefit to the overall problem. In the study of river bank conditions on the Red River made by the Province of Manitoba recently, one of the items investigated was surface drainage.(21) Of the 136 locations studied in Metropolitan Winnipeg, the majority on concave banks, only 17 per cent of the locations had "good" drainage, whereas 53 per cent had "fair" drainage and 30 per cent were considered to have "poor" surface drainage. It appears therefore that the providing of good surface drainage is essential, and any location on the local rivers which have been classified as having "fair" or "poor" drainage should be provided with the required improvements. Every precaution must be taken to prevent saturation of a river bank by surface runoff. Surface ditches should be constructed above the potential slides when necessary to intercept storm runoff; slopes should be reshaped, and benches or ditches constructed to carry off surface water as rapidly as possible. Paving of surface ditches may be required to prevent percolation of surface water into the slide material and into frost or shrinkage cracks. Surface water should be diverted from slide areas in a separate drainage system, rather than be allowed to drain at random.

To carry the collected surface water safely to the river below requires a system of large drains. These could possibly be tile or metal drains or even open-cut, lined channels. It is proposed, however, that a trench system be set up whereby the surface water and such amounts of subsurface water as seep out of the bank proper into the drains may percolate safely to the river. This proposal is shown in Plate IX, A page 48. At all concave bends and wherever potential slides exist vertical trenches are excavated and backfilled with suitable material. These trenches could serve to drain either one or two building lots from either side. Where industry exists along the river banks, trenches spaced at 100 feet could be installed. The widths could be the minimum of a mechanical backhow, which is approximately 3 feet. The depths perpendicular to the surface of the bank, would be the maximum as provided by the mechanical backhow, or approximately 20 feet. In cross section the trench could be as shown in Plate IX (B) page 48.

The benefits from a scheme such as this are, of course, not entirely known. It is definite that all water from leaking sewers, watered lawns, rains, and other sources which ordinarily seeps into the bank, will be caught by trenches such as these. Frost and shrinkage cracks, which are potential reservoirs will also be drained since their depth rarely exceeds 20 feet. The benefits accrued from subsurface drainage, that is from the clay itself are difficult to calculate and a quantitative analysis has not been attempted. This could be done if test trenches were made and observations were carried out. It is assumed, however, that some benefit will be derived by at least partial drainage, over a period of years. In Plate IX (c) page 48 is



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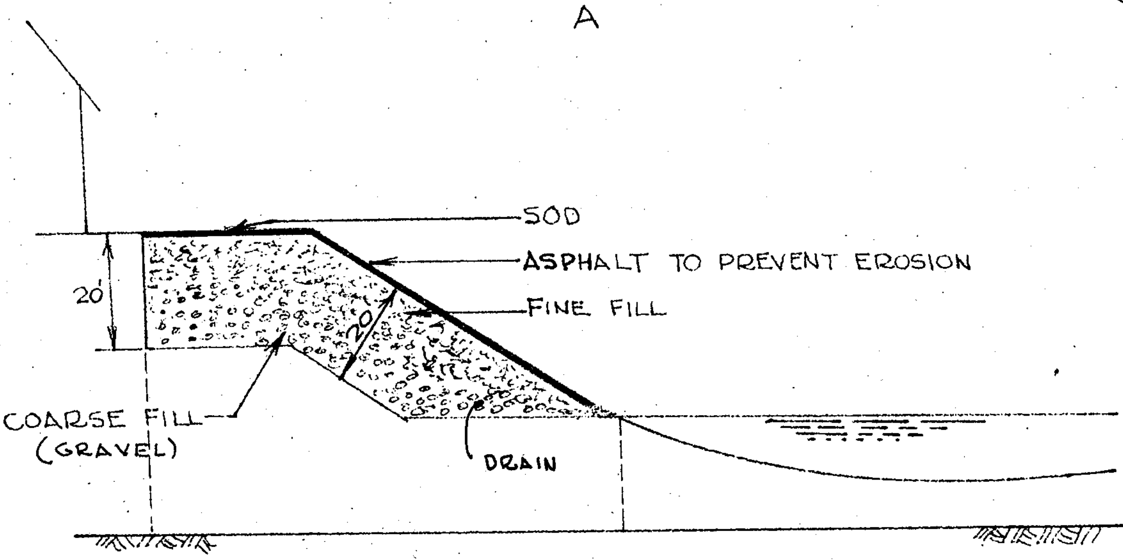
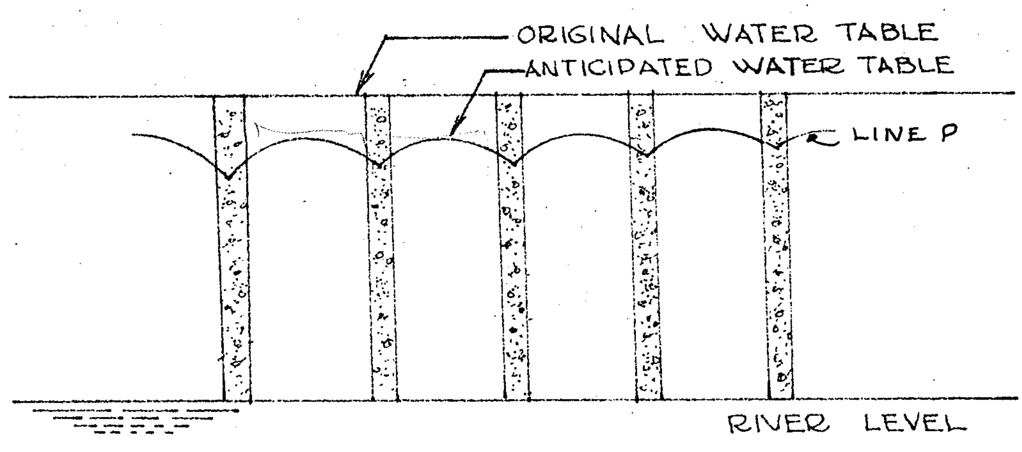


B



C

PLATE IX

3/62

shown a longitudinal view of the river bank. If the bank is completely saturated before construction of trenches, it is safe to assume that the trenches will tend to affect after a period of 2 - 5 years, the water profile as shown by line P. Actual investigations, would show just how the water table would be affected. While it would appear that floods may clog these drains it is important to note that the bulk of the sediment carried by the river is the bed load which is normally at or close to the river bottom.

It is thought that the most suitable machine, as mentioned, to do this work would be the backhoe. The operation in some areas, where the bank is accessible from the street side, could be carried out during the summer season. Where this is not the case, work would be done during the winter, despite the fact that frost would be a hindrance. The river ice is in most locations suitable to carry a medium weight machine. During both summer and winter, "deadmen" or anchors would have to be placed at the top of bank or natural ones such as trees used, to enable the machine to operate freely up and down the bank by cable attachment. During the summer, the digging of a trench should be a relatively rapid operation, and in the winter the frost would assist in maintaining an open trench. Some shoring would be necessary. The backfill of the trench would consist of material as shown on Plate IX (B) page 48.

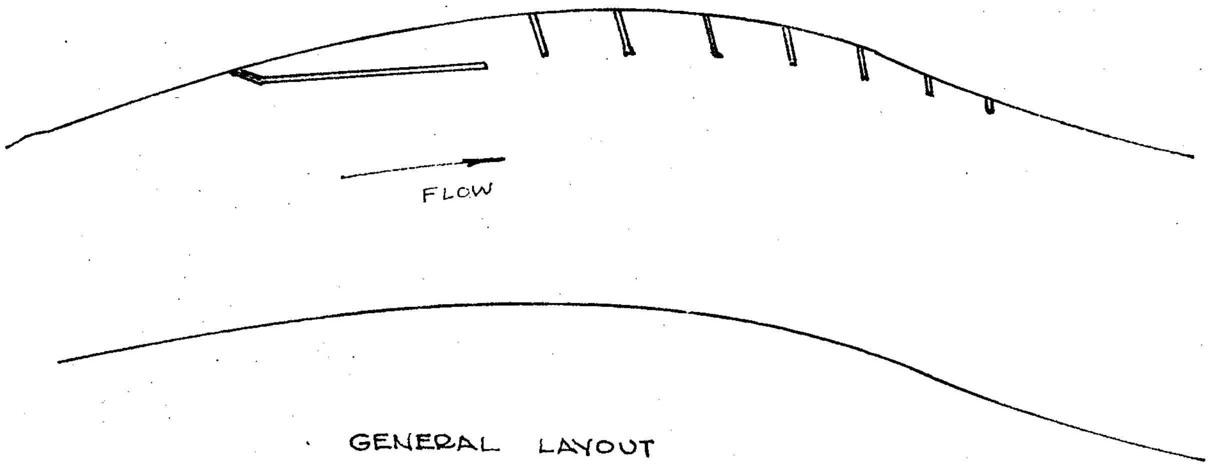
It is thought that this type of drainage should be initiated, in addition to, and regardless of any other type of improvement being made. It should be incorporated into whatever scheme is used.

A much more radical scheme and one which, of course, would be highly uneconomical because of extensive shoring, and settling of all adjacent buildings, would be the actual replacing of the unstable river bank, by suitable fill, such that a reasonable slope be obtained.

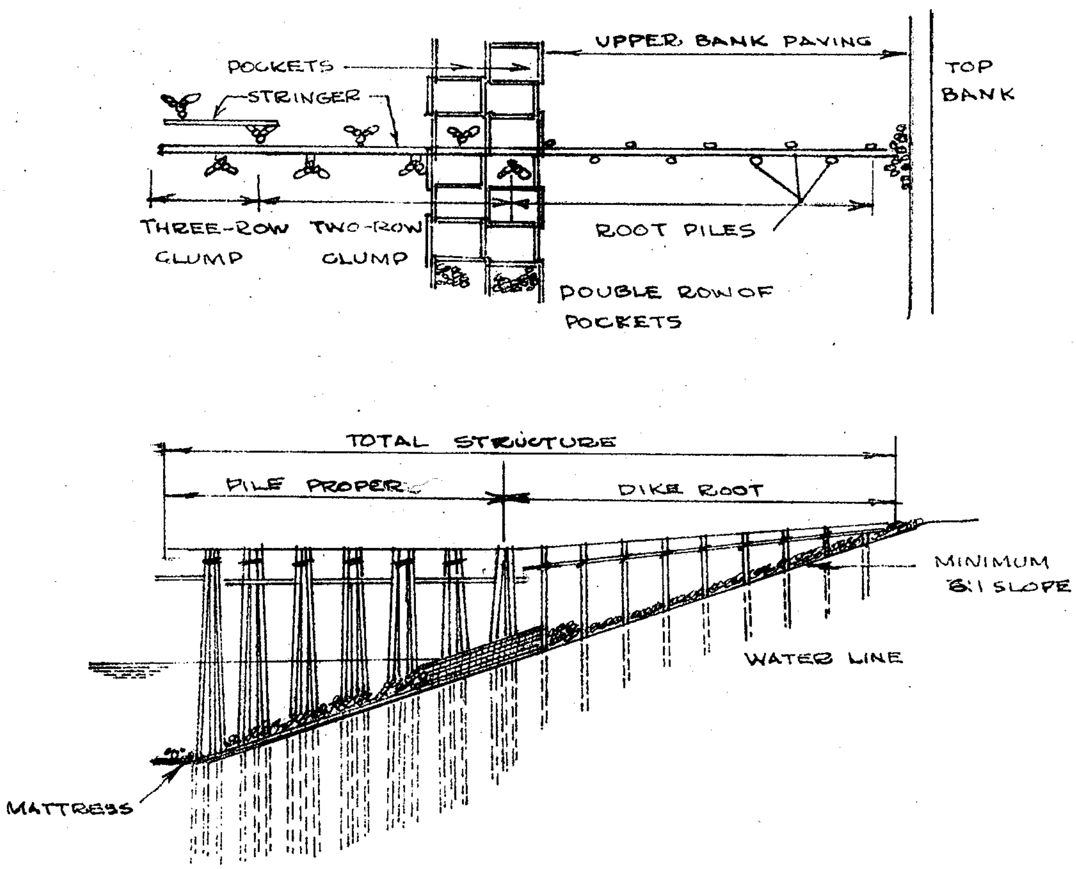
C. Re-sedimentation

Channel stabilization works utilizing the method of re-sedimentation is accomplished by the use of intermittent protection such as dikes or groins. While a revetment is designed to give direct protection to a bank, the intermittent structure protects the bank indirectly by inducing accretion in the dike or groin field or by deflecting the currents away from the area to be protected. The following are some of the possible methods of stabilizing a river, and hence its banks; methods which may cope with the situation at large.

Pile dikes, which have had considerable use on the Mississippi, consist of two or more, up to a maximum of seven, rows of pile clumps, three piles constituting a clump (See Plate X, page 51.) The rows are spaced approximately 5 feet apart, with pile stringers placed



GENERAL LAYOUT



DETAILS OF PILE DIKE SYSTEM

PLATE X

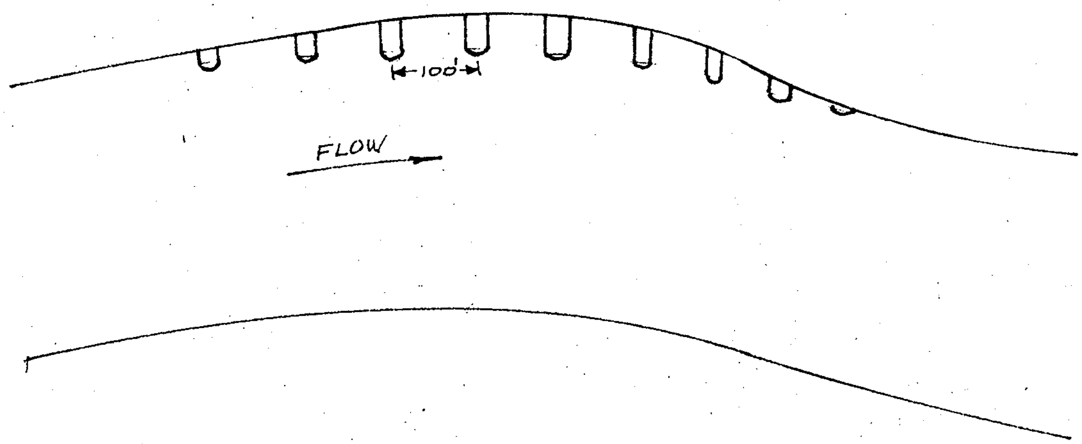
3/62 Ch

between each row. Clumps are spaced from 15 feet to 20 feet apart depending on the number of rows in a dike. Piles and stringers are secured with several turns of 3/8 inch galvanized wire strand fastened with boat spikes. The pile penetration varies from 20 feet to 30 feet below the river bed. Each dike is constructed on a woven willow or lumber mattress (or protected by rock), extending from the water's edge from 45 feet to 75 feet beyond the channelward end of the dike proper. Mattress widths vary from 77 feet to 100 feet. The mattress is ballasted with 15 pounds of stone per square foot and an additional 500 pounds of stone per row, per lineal foot, are placed in the dike line to fill holes torn in the mattress by pile driving. River bottom protection may also consist of a rock and rubble floor extending 25 feet to either side of the dike to protect against erosion around the piles.

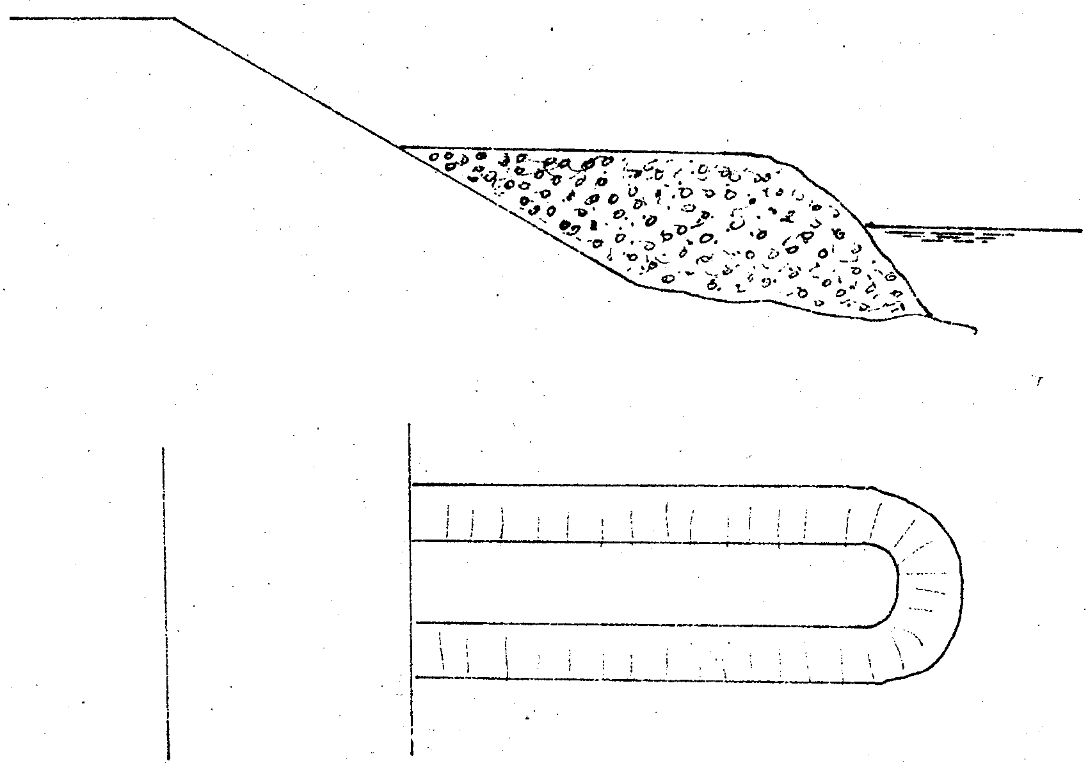
The crest elevation of the pile dike is usually set at mid-bank stage, which is usually from 15 feet to 17 feet above the low water plane. There are two reasons for choosing this elevation. First, pile lengths would become excessive if the top of the dike were raised substantially above this elevation, and, second, the most severe current impingement on the bank in a bend often occurs between low and mid-bank stages. After the latter stage is exceeded, the river has inundated the bar opposite the bend and the flow may become more or less axial.

Bank protection dikes are spaced from 1 1/2 times to 2 1/2 times the length of the upstream dike, depending on the radius of curvature of the bend and the angle and intensity of the current attack. The upstream dike of a system is inclined downstream at a small angle with the bank and extends to the rectified channel line. This upstream structure is designed to turn the current slightly off shore. The remaining dikes in a system are placed normal to the bank line or angled approximately 15 degrees downstream from normal.

Stone, because of added permanence, is coming to more general use for dikes, especially where a stone supply is readily available. (See Plate XI, page 54). The specifications for this stone limit the dirt and fines to five percent and the maximum size of a single stone to 750 pounds. The method of construction dictates the cross section of the dike. If the dike is constructed by river equipment, it is usual to provide no crown for dikes. The side slopes are approximately 1 to 1.25, the angle of repose of the stone. If land equipment is used, however, a 10 foot crown is provided to allow passage of the stone trucks. The crest elevation is usually lower than for pile dikes and varies from 5 to 10 feet above the low water plane.



GENERAL LAYOUT



DETAILS OF STONE DIKES
(SIMILAR TO GROINS)

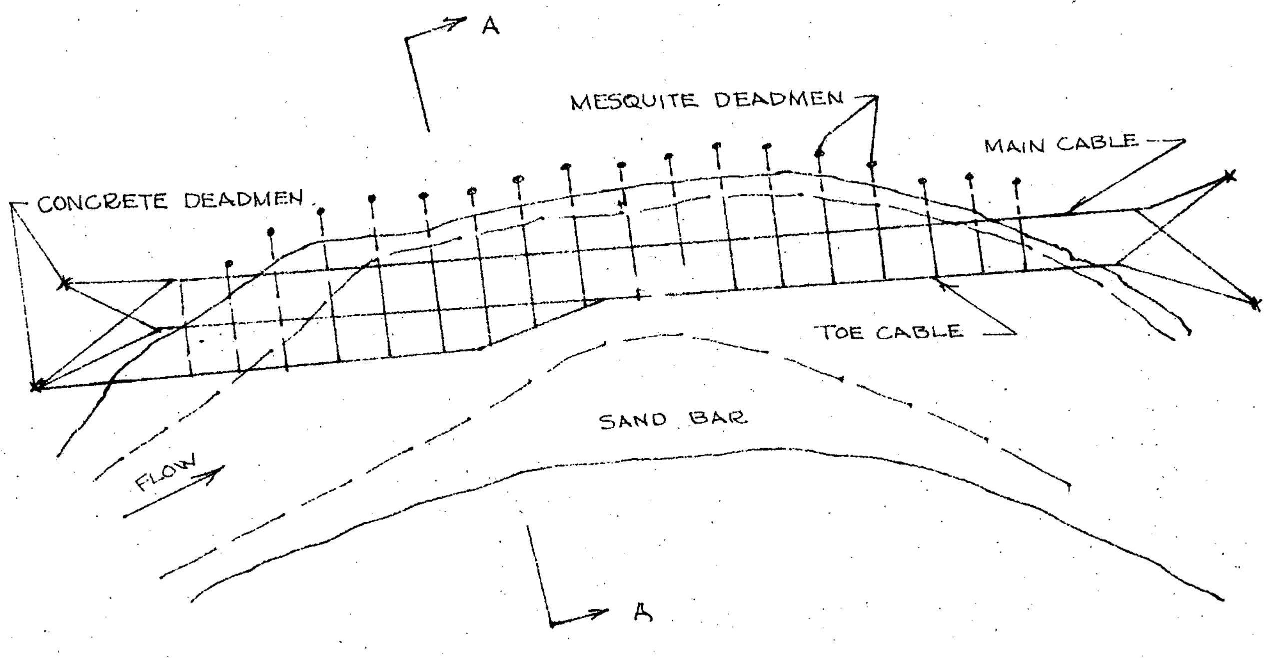
PLATE XI

3/62

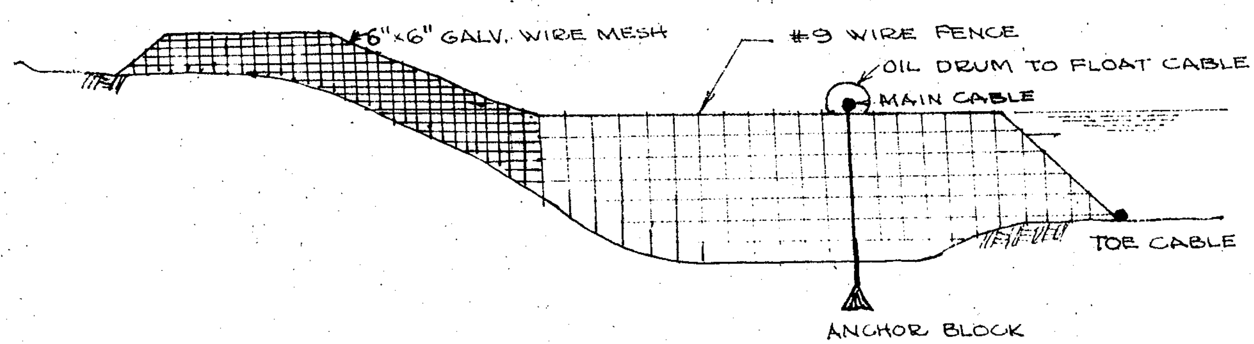
Groins, usually constructed of stone, are similar to the stone dikes except a constant height of groin above the river bed is maintained instead of a fixed elevation. The height will vary from 5 to 10 feet, usually no crown width exists, and the side slopes are the angle of repose of the stone.

An intermittent structure, which has seen some use is the wire fabric fence. (See Plate XII, page 56). A series of wire fabric fences is built transverse to the sloping bank of the river and wire fabric nets extend from the fences out into the river. The wire fabric fencing is supported by second hand boiler tube posts. Each fence is guyed upstream and downstream to adjacent fences. The wire fabric nets are supported by messenger cables which are strung transverse to the water's edge. The messenger cables are supported by a toe cable which runs parallel to the bank, and supported by a main cable, also parallel to the bank, between the toe cable and the water's edge. Concrete weights placed at regular intervals anchor the nets to the sloping bed of the river. The main cable is supported by empty oil drums sufficient in number to support the combined weight of nets and cables.

This type of revetment is not intended to cut off or entirely divert the current but rather to decelerate its velocity so that the material carried by the river is deposited between the fences and



GENERAL LAYOUT OF WIRE FABRIC FENCE



SECTION A-A

PLATE XII

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nets. The revetment is eventually buried in this hydraulic fill. The installation has, in many cases, been very effective.

To keep such a structure from becoming unsightly and to keep the fence to such a height that the forces produced upon it by the current are minimized, the posts which are driven into the river bed could be made more sturdy. These sturdy posts could then be so constructed as to allow extensions at the top. Thus, if a three foot high fence were constructed, and if after several years these fences had been buried by accretion, a three foot extension could be raised where desired to accumulate such additional sediment as may be required. In this manner, the amount of sediment deposit may more easily be controlled.

A somewhat similar method of sedimentation is achieved by the method of fascine boxes. These consist of a framework of angle iron, with strap iron used as braces held together with stove bolts. The method used for forming the retards is to use 2 inch mesh chicken wire or hog wire fencing tied to the framework with baling or number 8 wire. The relatively short life of the chicken wire has led to the almost exclusive use of welded hog wire having a 4 inch by 4 inch or 6 inch by 6 inch mesh. After construction and placement the boxes are anchored to "deadmen" or trees situated well back from the river bank by 3/4 inch wire rope.

As a general rule the boxes are placed on the downstream end of a bend in the river. The permeable retard design of the boxes serves to slow the velocity of the current and also catch drift in the netting. By thus retarding the river flow a deposition of the sediment carried in suspension is brought about. In time this process builds up a bar along the river bank and causes the channel to be pushed back from its previous location. Metal is used in the fabrication of the boxes to secure durability so that the boxes will be effective in keeping the river from encroaching again. The durability of the boxes also keeps the bar stable long enough for vegetation to grow so that eventually the bar becomes a permanent bank.

The boxes are assembled on the river bank and are moved into place by barges, sometimes in sections as long as 400 feet. Field boring of bolt holes is kept to a minimum by factory boring most parts. The boxes are placed as far as is practical at right angles to the river current and are anchored at the shore end. After being placed in the river, the boxes frequently are rolled by the current and at times may move to a position as much as 45 degrees from their original placement. After they have come to rest, the silt settling and drift catching process ensues, and in time the boxes either have become buried in the bar or the drift catching in the boxes raises them up until the bar has built itself above the river, in which event the boxes are on dry land.

In the case of caving banks, longitudinal boxes are constructed along with and interlocked to the lateral boxes extending out into the river channel. These longitudinal boxes serve to catch silt and drift in time of high water and sometimes also keep portions of the bank from sliding into the river. It is estimated that 10 men can make up to 400 lineal feet per day, including placement.

Many other types of intermittent structures exist, some of which are variations of the above mentioned and some of which are not applicable to the conditions concerned. The above are, therefore, a selection of the most suitable types available, either in original form, or altered by the author to make them adaptable.

D. Structures

Structures, which alleviate the problems of unstable banks, consist of two types; namely the retaining wall type of structure which contains the bank in the existing position and the revetment type of structure which offers protection from scour. While it is feasible for the wall type structure to protect a bank individually, this is not so for the revetment type. This structure can be applied only after the bank has been stabilized and is used largely to prevent undercutting of the bank to avoid further bank failures.

1. Retaining Wall Type Structures

A retaining wall-type structure will be defined here as one which prevents movement of the bank due to its base friction. For simplicity of presentation toe berms will be included under "structures". Toe berms may consist of rock, earth, or concrete placed at the toe of the bank while retaining walls consist of the actual retaining wall structure.

To properly analyze a river bank condition it is required to obtain some knowledge of the forces acting on the bank. The most serious slides have been found

> "to involve movement along an approximately circular arc tangent at its base to the firm glacial till, or bedrock where till is not encountered." (22)

Analysis of the mechanics of slide will be made by the use of the Swedish Circle Method of Analysis which is described in most Soil Mechanics text books and which is commonly used. It must be noted that while the following analysis may give an indication of the forces existing in a bank, the method of analysis, as carried out here, and the resulting data, are for the very simplest case only.

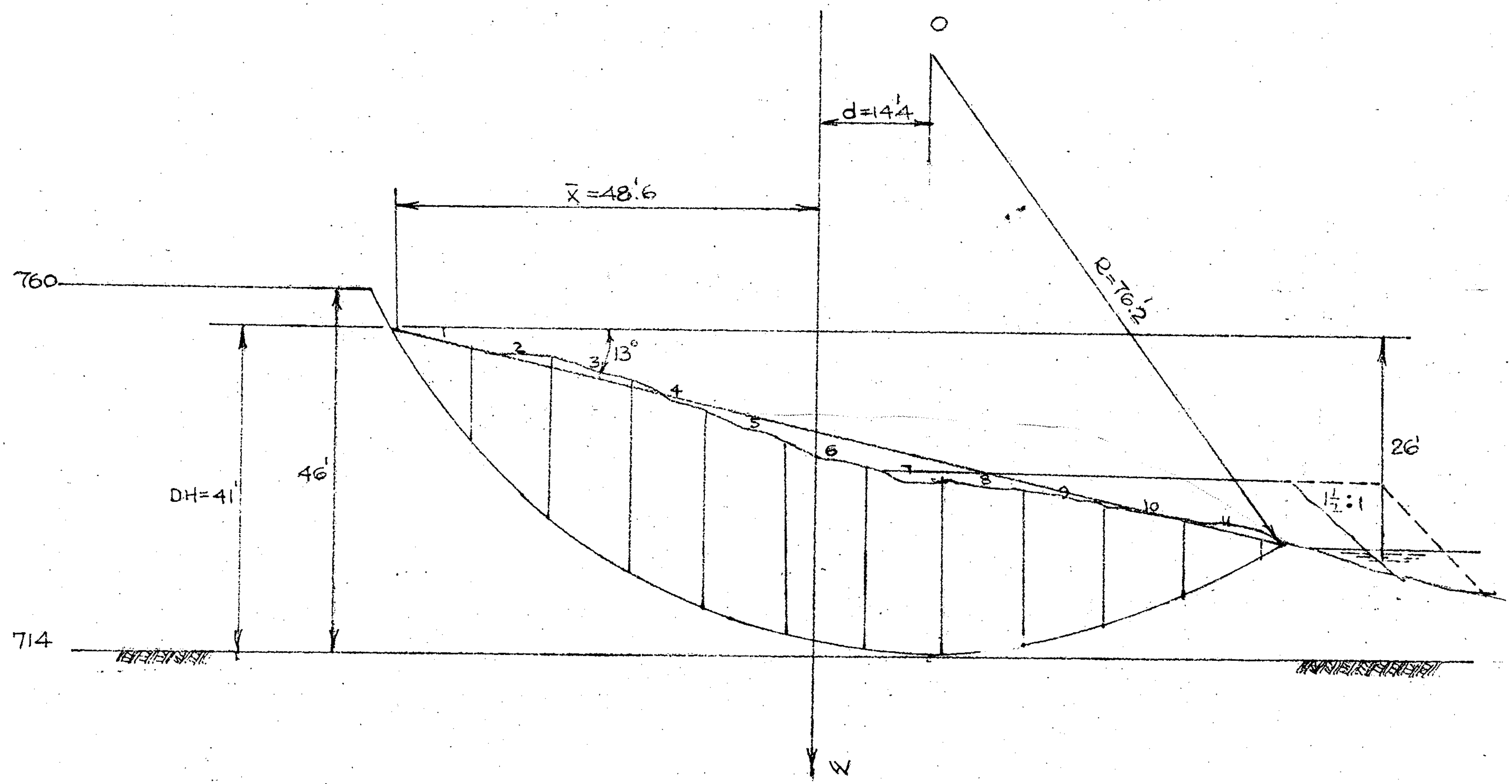
For an actual case, of course, a much more detailed study is required. The case here considered will be a bank at impending failure due to rapid drawdown from flood level to summer water level. The water table is assumed to be at ground surface at all locations. Seepage forces were considered using the approximate o = o case with a reduced cohesion value. However, according to Taylor,

> "it may be shown that the case of steady seepage is in general slightly more stable than the sudden drawdown case, and thus the stability number for the sudden drawdown case may often be used as a conservative approximation of the stability number under steady seepage". (23)

The bank, which may be considered "typical" for this study, is shown on Plate XIII, page 62. The analysis of the bank is given in Appendix "C". In the analysis of the bank it was shown that the safety factor of the bank at impending failure was close to one. Two different quantities of toe loading were applied to the bank and the safety factors obtained were found to be 1.20 and 1.26. It is seen from the calculations that a massive toe loading is required to appreciably increase the safety factor. In the final results it is a matter of economics. The toe fill will necessarily cause a constriction in the river; the extent of this has been reviewed in Part II.

In the design of a retaining wall conditions similar to the above were considered, that is, rapid drawdown. Plate XIV, page 63 shows the relative location and forces acting on a retaining wall. Line ABC is the approximate assumed failure zone with the apex B in contact with hardpan. The resulting assumed lateral pressure distribution is shown. Here again, as in the above case it must be remembered that the method used is approximate and the design conservative.

For major protection works, therefore, two methods are suggested. Where the bank slope is relatively mild and where the



SWEDISH CIRCLE METHOD ANALYSIS

PLATE XIII

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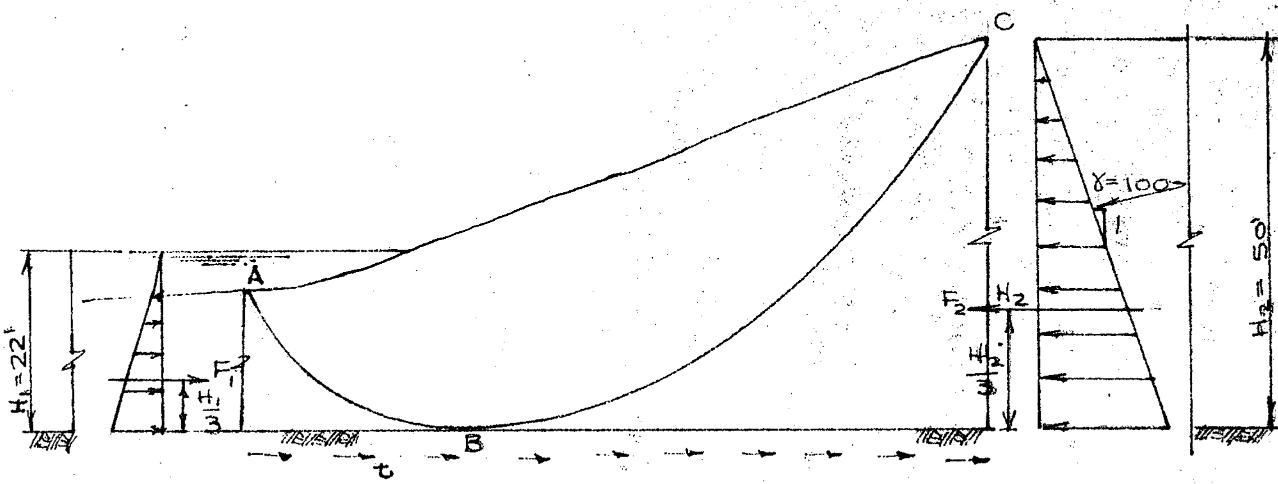
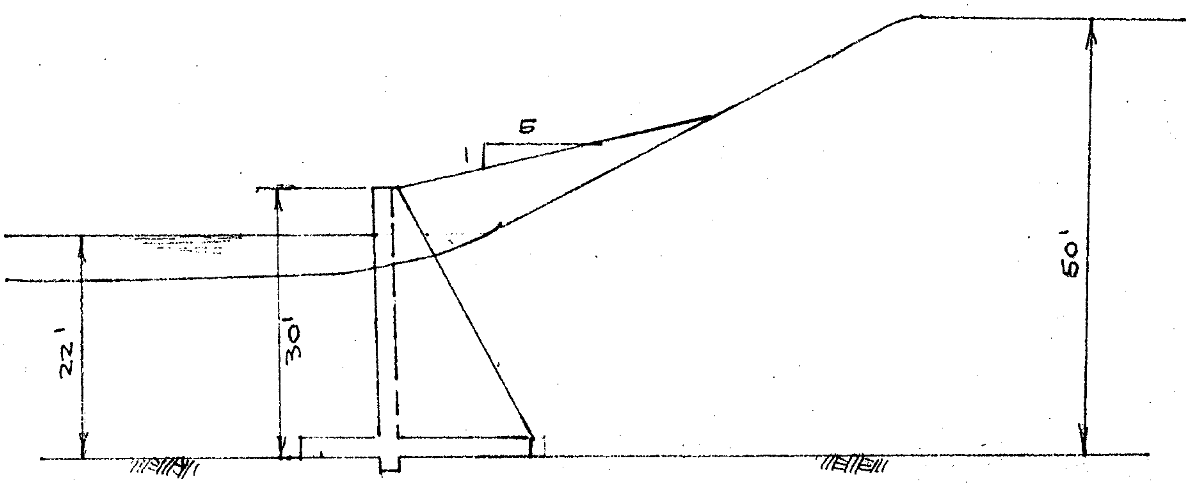


$F = \frac{1}{2} \times \gamma \times H^2$

SIMPLIFIED FORCE REPRESENTATION
FOR BANK ANALYSIS



RETAINING WALL

PLATE XIV

3/62

river has scoured deeply adjacent to the bank, one solution consists of heavy toe fill as shown on Plate XV - D, page 65. The conditions existing on this bank are considered to represent a typical cross section.

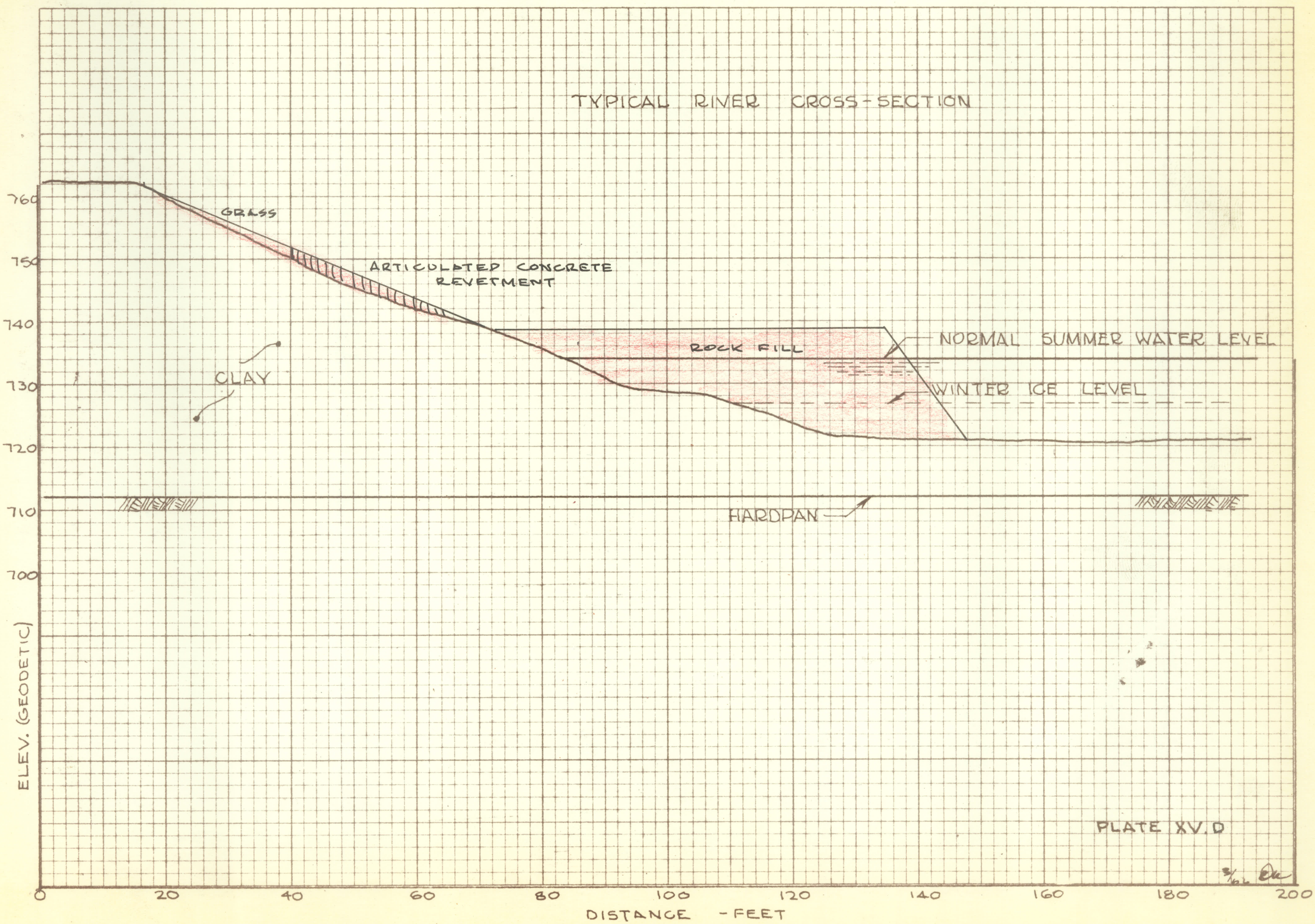
Where the bank slope is relatively steep, one solution will consist of a retaining wall keyed into the hardpan below as shown on Plate XIV, page 63.

Three types of retaining walls are commonly used, namely the gravity wall for heights up to 15 feet, the cantilever walls which are economical for heights of 10 to 20 feet and counterfort walls for greater heights. Since the river bottom here consists of clay, it is somewhat unsuitable as a base for a retaining wall. The wall will be taken down to and keyed into the hardpan below the clay. A study was made to determine the mean depth of hardpan below normal summer water level. This is shown in Appendix "A". The figure arrived at was 22.0 feet.

To create a slope of 6 to 1 as suggested by local authorities the top of the retaining wall is required to be 8 feet above summer water level. The height of retaining wall, therefore, is 30 feet. The design of the retaining wall is given in Appendix "B".

The overall design is shown on Plate XVII, ,

page85A. The design is admittedly of a preliminary nature. A



TYPICAL RIVER CROSS-SECTION
GRASS
ARTICULATED CONCRETE
REVETMENT
ROCK FILL
NORMAL SUMMER WATER LEVEL
WINTER ICE LEVEL
CLAY
HARDPAN
760
750
740
730
720
710
700
ELEV. (GEODETIC)
0
20
40
60
80
100
120
140
160
180
200
DISTANCE - FEET
PLATE XV.D

check should be made on the external stability due to changes in wall and slab thicknesses, Balanced steel ratios have been used and should be checked due to alterations in wall and slab design.

However, it is thought that the overall design gives the approximate proportions of a wall that is required to withstand the maximum earth pressures. Perhaps further investigations would alter the ratio of height to width resulting in a more economical cross section. It must be realized that a wall resting on hardpan is on a very rigid support, as compared to a wall resting on clay, for example. Hardpan, when cleared of overlying loose rubble and gravel, can resist a bearing pressure of up to 15 kips per square foot. More accurate design would, needless to say, reduce the retaining wall and base slab thicknesses somewhat.

Actual construction of the retaining wall would entail the driving of two rows of sheet piling; one row to contain the bank proper temporarily, and a second to keep the river out of the excavation area. This temporary piling which constitutes the major portion of such a construction project, would create many difficulties, particularly in the withstanding of earth pressures from the bank. Conditions, such as low water table, and minimum bank saturation and overload, would have to be optimum. The overall operation would be one of constant vigilance regarding earth movement and constant readiness to reinforce the temporary structures by means of anchors and "deadmen".

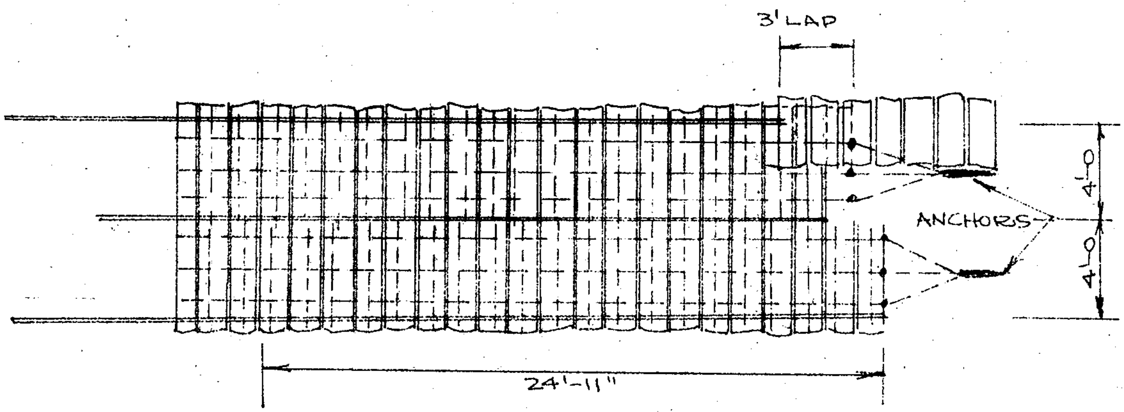
Another form of retaining wall which may be successful in some cases is the pile or sheet retaining wall which is anchored to "deadmen" on the bank beyond the sliding arc. In the existing case, the forces were found to be so excessive that this type of wall was not considered. It would be possible to place anchors at approximately every lot line which may be 50 feet. Even though the extremely large forces in the tension members connecting the wall and the "deadmen" could be contended with, the 50 foot spacing would create such large moments in the wall between the connections as to make the design impractical. The use of this method would find application at isolated locations where banks are relatively low.

2. Revetment Type Structures

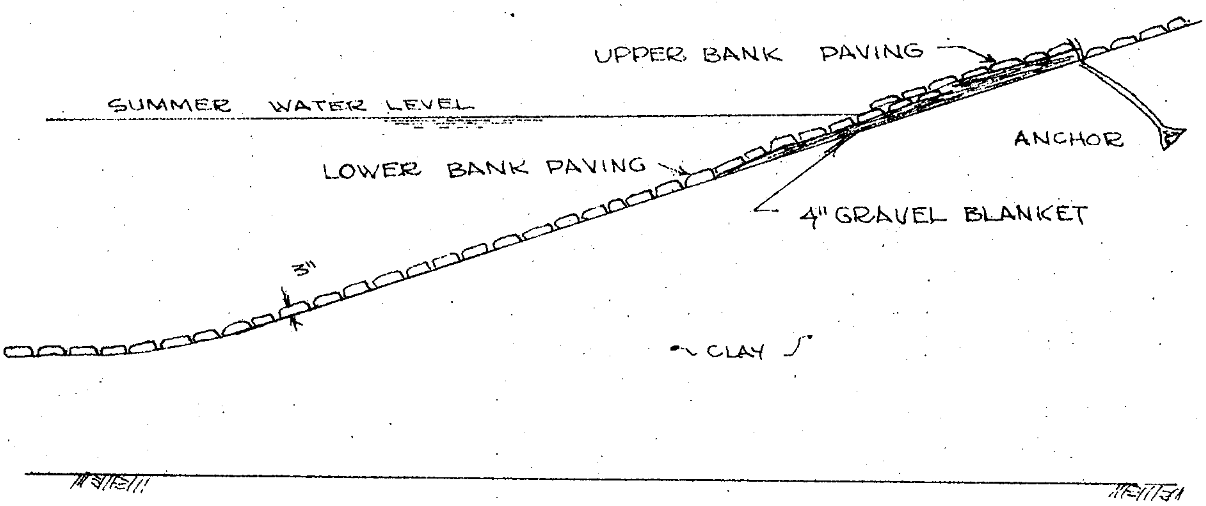
A revetment is defined as a structure designed to protect the river bank from erosion and consists of protection for below normal water level and bank paving from this water level to some location on the upper bank. Revetments, to be effective, must be long enough to protect the entire concave bend. They must be flexible, and must have sufficient strength. They should be relatively impermeable and continuous to prevent soil particles from leaching out through the revetments (due to currents). Revetment material should be relatively indestructible in air and water. As banks, particularly in the underwater position, may be of irregular contour, it is desirable, or even necessary, that some grading be done.

The history of revetments in America, and largely in the United States, dates back to the 19th century. For many years woven willow mattresses were the predominent type of revetment. It was realized in about 1890 that concrete might be adopted for river bank revetment but it was assumed suitable only for the upper portions of bank. However, in about 1915 when willows became extremely scarce in the south central States, experiments were begun with underwater concrete mattresses. The first underwater concrete mattresses were monolithic reinforced mattresses and were constructed on barges provided with sloping ways from which mattresses were launched by cantilever cranes. Before this type of mattress was abandoned the slab size had increased to 240 feet by 12 feet by 3 3/4 inch. It was flexible enough to take the shape of the bottom but the main reasons for abandonment of the monolithic mattress were the difficulties of launching and the fact that it was found impossible to control the final position of the mattress. The procedure was also very expensive.

As long ago as 1915, the development of the articulated concrete mattress (see Plate XVI page 69) was put in hand and the pro-



PLAN



SECTION OF ARTICULATED CONCRETE MATTRESS

PLATE XVI

3/62

cedure adopted, with modifications suggested by experience, is still followed but a further advance was made in 1943 when the so called flexible concrete mattress was devised. This latter type, which may be rolled up for transportation has advantages from the point of view of handling and laying. An articulated mattress which is able to adapt itself to the contour of the bank on which it is laid has obvious advantages over a monolithic mattress which beds itself only by fracture of its structure.

The present standard type of articulated mattress used in U. S. Army Corps of Engineers projects consists of 20 blocks, each 3 feet by 10 1/2 inches long, 14 inches wide and 3 inches thick. The blocks are spaced 1 inch apart on a non-corroding reinforcement. The entire mattress is 25 feet by 4 feet in area. A number of these mattresses are assembled side by side to make up bank paving. They are unrolled in the direction perpendicular to the river bank. The reinforcement on which the blocks are carried, consist of three longitudinal wires passing through the full length of the mattresses and provided with loops at the ends so that connection may be made to other mattresses. The longitudinal wires are connected by transverse reinforcement consisting of rectangles made of 0.18 inches diameter wire. The rectangular frames project from each end of the blocks, forming brackets by means of which mattresses may be connected side by side. Launching cables are also connected to these brackets when a mattress is being laid, so that the launching stresses do not come on the longitudinal reinforcing wires. Reinforcement is made of copper-coated steel.

Sulfate resisting cement is used and the concrete requires a compressive strength of 2,000 pounds per square inch, in some cases air entrainment is used. Casting takes place in steel forms, slotted to hold the reinforcing wires. The concrete is run into the steel forms, vibrated, and then finished. When the concrete has taken the initial set, usually in about 4 hours, the steel forms, which had previously been coated with a mineral oil to allow easy separation, are removed. The first mattress of a batch is cast with the forms lying at ground level; when it is finished it is covered with two layers of heavy paper, over which the forms are reassembled and a second mattress is run. This is done until approximately 12 have been built up. The heap is left to cure for 10 days.

For laying, the 12 mattresses, resting in a built up frame, are transferred to sinking barges. Most barges lay mattresses built up of 25 feet by 4 feet units of total width of 100 feet. The sinking barge is equipped with two cranes which are used to transfer the mattress from the mattress barge to the sinking barge. Barges can be loaded from shore by means of piles and rails or by available dock facilities.

As these articulated mattresses do not form a continuous slab there is a certain amount of leaching through the interstices between the blocks but this is generally not serious. Improvement in this particular aspect has been attained by development of the roll-type flexible mattress, which is less permeable than the articulated type. The large advantage of the roll-type is in the simplicity of the launching arrangement and the fact that since it can be unrolled in-direct contact with the bottom of the river, laying is less liable to be interfered with by the action of the current, where current is a hazard.

The roll-type mattresses are made up in 60 feet by 24 feet sections and these consist of slabs 4 inches wide by 1 1/8 inches thick by 24 feet long. The slabs are carried by a 2 inch by 4 inch non-corrosive wire mesh reinforcement and the spaces between them are 1/2 inch on top and 1/8 inch on bottom. The mattress is then rolled on a wooden core 17 inches in diameter, secured with steel tape, and is ready for application.

The above types of revetments are, in many localities, standard revetments. However, there are many other types of revetments, which for some particular problem, serve the purpose well. Some other methods which have been used elsewhere are mentioned here.

"Another method of revetment for the upper bank is to use 55 per cent washed concrete gravel, 38 per cent washed sand and 7 per cent of 40/50 asphalt cement. The bank is graded (1:3) but it is not necessary to prepare the surface except for the removal of drift and the smoothing out of irregularities. The mixture is applied at a temperature of 300 to 350 degrees Fahrenheit and raked to an average thickness of 6 inches. Little raking is required since the mixture is almost self-flattening. For lower bank (under water) paving, the process of mass asphalt has been used. A sand asphalt mix containing about 12 per cent asphalt cement is released from bottom dump barges at 350 degrees Fahrenheit in masses of 2 to 300 tons. Upon reaching the river bed it spreads and congeals to form an impervious coating 8 to 15 inches thick.

A pavement of concrete blocks 2 feet square, 4 inches thick and reinforced with galvanized wire mesh has seen limited use on the Missouri River. The mesh has loop strands for fastening the blocks together. This pavement has been discontinued in favor of cast stone paving. The cast stone paving is composed of stone varying in size from 6 to 75 pounds, placed on a graded bank to a thickness of 10 inches. The stone is dumped on the bank by truck or dragline and rearranged as necessary to attain the specified thickness.

Frequently used in river banks on the Mississippi River today is the lumber mattress. In lieu of brush, 1 by 4 inches boards of various lengths are used. The weavers are spaced 4 feet on centres and the mattress boards are placed 1 to 4 inches apart. Top boards are placed on top of the weaving boards directly over the weaves, and cross binders are placed across the top boards at intervals of 20 to 25 feet. Nails and wire lashings are used to fasten the mattress together and the mattress is reinforced both longitudinally and laterally with wire strand. The mattress is begun and ended with a header, consisting of 6 boards lapped to extend the entire width of the mattress. To overcome buoyancy, 1 to 2 tons of stone per square are used to sink the mattress and hold it securely on the river bottom. Cribs consisting of boards laid alternately along and across the mattress are placed on the edge to hold the stone in place.

The mattress is anchored at 50 foot intervals by means of wire strand secured to "deadmen". (24)

The most recently developed revetment is the toe trench revetment. It has found extensive use on the Missouri and Arkansas Rivers. It consists of a fill of not less than 5 nor more than 9 tons of quarry stone per linear foot placed to a depth of 7 feet be= low the low-water plane in a trench excavated prior to erosion by the river or partly in a trench and partly in the area already eroded. Where this type of subaqueous protection is used, paving for the upper bank is placed 15 inches in thickness abutting the rock fill and decreasing to 10 inches at the top of paving. Quarry run stone is also used for the paving. For this protection, quarry run is similar to the stone used for dikes and dams except the maximum size of stone is limited to 250 pounds. Due to the fines present in this stone, no crushed stone or gravel blanket is specified.

Where the bank has receded landward of the desired limit line, pile revetment is sometimes used to restore the bank line. This type of protection is a two or three row pile dike built along the desired line. A lumber mattress is often placed at the bottom of the river along the axis of the pile revetment. These pile dikes are impermeable and have been successful in preventing bank recession. The dike is begun at the upper end of bank caving, running at a flat angle with the bank for a distance of 500 to 1,500 feet, supplemented by a series of 2 to 5 spur dikes running normal to the shore line out to a line drawn roughly parallel to the bank line from the outer end of the trailing dike. The tops of the piles are at about mid bank stage and the dikes are continued up the top of the bank by single piles with stringers protected against flanking by riprap extending 50 feet up and downstream. (See Plate X, page 51)

CHAPTER IV

SELECTION OF APPLICABLE PROJECTS

In this study, the limits for improvements were set on the Red River from St. Norbert in the south to the St. Paul Kildonan boundary in the north. The Assiniboine River was studied from its mouth at the Red River westward to a point two miles east of the Perimeter Highway. While it is true that the consequences to the two rivers due to bank improvements may vary, the improvements will, for lack of time, be considered applicable to both. For the rivers combined, there are approximately 64 miles of river bank within the above stated limits. These have been classified into requiring major improvements consisting of say retaining walls or extremely heavy toe fill, and light improvements requiring only minor toe protection to prevent erosion. As was stated above, the two stipulations made in this study are; firstly, that complete protection is required, and secondly, that protection will be adequate to accommodate the worst possible condition. Thus, where old, existing, or potential slides are found, major improvement is considered to be necessary. Where banks are low and stable, largely on convex banks, and in reaches, minor improvements will be necessary. Of the 64 miles, it is estimated that 28 miles of major improvement and 36 miles of minor improvement are required. It must be noted that the major costs will be created by the major improvements, which will also yield major benefits. Minor projects are located largely at convex bends or at the straight reaches where damage is slight.

A. Drainage of Banks

Drainage of the clay, which composes the bank, is, as mentioned above, not a satisfactory method of stabilizing the bank to the degree required. It is thought, though, that the type of surface drainage as proposed above will certainly assist in the overall development of stabilization projects. This type of drainage will be included in some, if not all, of the other projects developed. Costs will also be estimated for carrying this type of drainage, that is, for constructing the gravel filter not only 20 feet down but directly down to hardpan level. This may assist in eliminating the subterranean water veins by intercepting them before they reach the toe of the bank. This method is shown with broken lines on Plate IX, page 48.

B Re-sedimentation

Considerable work has been done by Thomas R. Camp on sedimentation velocities and the results forthcoming have been considered acceptable. The velocity required to bring into suspension particles of a specific size and density have been found by Camp with the use of the following equation:

$$V_c = \sqrt{\frac{8\beta}{f} g(s-1) D} \quad (\quad)$$

This equation will be used to find, not the velocity of induced suspension but the velocity required to allow particles of a certain size and density to settle to the river bottom. That is, with a certain velocity, particles of a certain size or greater will begin settling. The terms of the equation are as follows:

V_c - velocity - feet per second (average)

f - friction factor

g - force due to gravity , feet per second squared

s - specific gravity

D - diameter of the particles, mm

β - a constant and a function of

$$\frac{T_c}{(\gamma - \gamma_s) D}$$

where T_c is the critical tractive force, γ_s the specific gravity of the particle, γ the specific gravity of water and D the diameter of the particle. For very fine particles β has been found by Camp to be 0.04.

The friction factor suggested by Camp is 0.03.

The specific gravity of the suspended sediment is taken as 2.65.

The diameter of the particles sampled by the Government of

Canada were .002, .004, .008, .016, .031, .062, .125, .250, .500 and 1.000. (Samples taken in spring, 1957 - 1960)

Using the above equation, the following table was drawn up:

TABLE II

SEDIMENTATION VELOCITIES

Particle Size	Per Cent Finer Than Indicated Size		Maximum Allowable Velocity for Settlement
m.m.	(1)*	(2)**	Feet per Second
.002	44	27	.061
.004	51	37	.086
.008	78	57	.122
.016	71	77	.173
.031	95	89	.241
.062	97	95	.340
.125	98	97	.483
.250	99	98	.684
.500	100	98	.967
1.000	100	100	1.370

*For sediment samples taken on the Red River at Elm Park Bridge.

**For samples taken on the Red River at the Redwood Bridge, below the confluence of the Red and Assiniboine Rivers.

Sediment samples were taken by the P.F.R.A. Branch of the Government of Canada, Department of Agriculture, and the data were analyzed by the Water Resources Branch, Department of Northern Affairs and Natural Resources, Ottawa. Locations of sampling stations within Metropolitan Winnipeg were at Elm Park Bridge on the Red River, and at the Redwood Bridge on the Red River below the confluence of the Red and Assiniboine Rivers.

Table II, page 79, shows the different concentrations of particle sizes at the two stations, showing that the Assiniboine River as can be expected, contributes somewhat coarser sediment than the Red River.

Table III, page 82, shows the result of the sampling in tons of sediment carried by the rivers.

The above data, therefore, give a resumé of the suspended sediment available in Metropolitan Winnipeg. The velocities required to begin accretion are relatively small, and a relatively impermeable structure would therefore be required. No significant information is available regarding velocity reductions afforded by re-sedimentation structures. It is assumed that the selection of the correct structure has been one of trial and error. Experimentation, in this case also, would determine the usefulness of each particular structure. Model studies of pile dikes, wire mesh fences and rock groins would be particularly beneficial in determining their worth.

More significant, however, would be the experimenting with prototypes, not only for obtaining more accurate results as to the reliability of the structure as an accretionary asset, but also for gaining more knowledge about actual quantities of bed-load being carried by the rivers. At present, no practical method exists for measuring accurately the quantity of bed-load.

For this reason and because it is doubtful whether the existing suspended sediment can be satisfactorily utilized, there exists the necessity for experimentation.

If it is assumed that the velocity of the Red River, which during spring flows varies between 3 and 6 feet per second, can be reduced to one third foot per second, then 5 per cent of the sediment passing Redwood Bridge could be utilized providing approximately 1,000 tons per day. The area of the structures would normally be approximately 5 per cent of the river cross section during spring flows, when a great portion of the re-sedimentation would occur. A structure 10 feet high, such as a rock groin would extend 0.3 depth above the river bottom when the depth of flow is 30 feet. The velocity distribution is such that it increases from 0 at the river bed to somewhat below the mean at about 0.2 of the depth above the river bed. The coarser sediment is carried very near the 0.2 depth above the bed and it has been conservatively estimated therefore, that approximately 10 per cent of the settleable sedi-

TABLE III

SUSPENDED SEDIMENT LOADS OF THE RED RIVER

AT REDWOOD BRIDGE

Year	Dated Started	Dated Ended	Period Days	Discharge Acre-Ft Total	Discharge Acre-Ft Mean Daily	Sediment for Period - Tons Total	Sediment for Period - Tons Average Daily
1956	Apr 18	June 7	51	4,187,000	82,100	2,188,700	42,900
1957	Apr 4	June 20	78	2,000,000	25,600	938,000	12,000
1958	Apr 10	May 2	23	308,000	13,400	159,800	6,950
1959	Apr 15	Apr 30	16	374,000	23,400	187,600	11,700
1961	Apr 19	Apr 28	10	107,000	10,700	28,100	2,800

AT ELM PARK BRIDGE

Year	Dated Started	Dated Ended	Period Days	Discharge Acre-Ft Total	Discharge Acre-Ft Mean Daily	Sediment for Period - Tons Total	Sediment for Period - Tons Average Daily
1956	Apr 21	June 5	46	2,498,000	54,300	1,443,000	31,400
1957	Apr 16	June 20	66	882,000	13,400	302,500	4,580
1958	Apr 8	Apr 20	13	125,000	9,600	24,200	1,860
1961	Apr 20	Apr 28	9	69,900	7,800	19,200	2,140

ment would be trapped by the structures. During a forty day period when the average daily sediment has been measured at 20,000 tons, 100 tons of accretion would be obtained, or some 2,000 cubic feet per day. It appears, therefore, that this estimated quantity of suspended sediment, plus an unknown quantity of bed load, plus additional sediment in the remaining 325 days each year, could amount to a considerable quantity of re-sedimentation.

No definite proposal will here be put forth but the limited information available warrants further study.

C. Structures

Most of the solutions that cope with the major problems involved are structures of different types. One solution then is the use of rock fill as shown on Plates XV-A, page 84, and Plate XV-B page 85. This will produce about 550 cubic feet of fill per lineal foot of bank. In conjunction with this backfill, the bank, where at all possible, will be sloped to a maximum extent, and revetted with one of the forms of revetment mentioned above. The revetment must extend 25 feet up the bank to protect against high water, at which point a hardy grass which is capable of strong rooting, must be seeded.

Another solution is the construction of a retaining wall. A retaining wall copes with the most dangerous situation. A cross

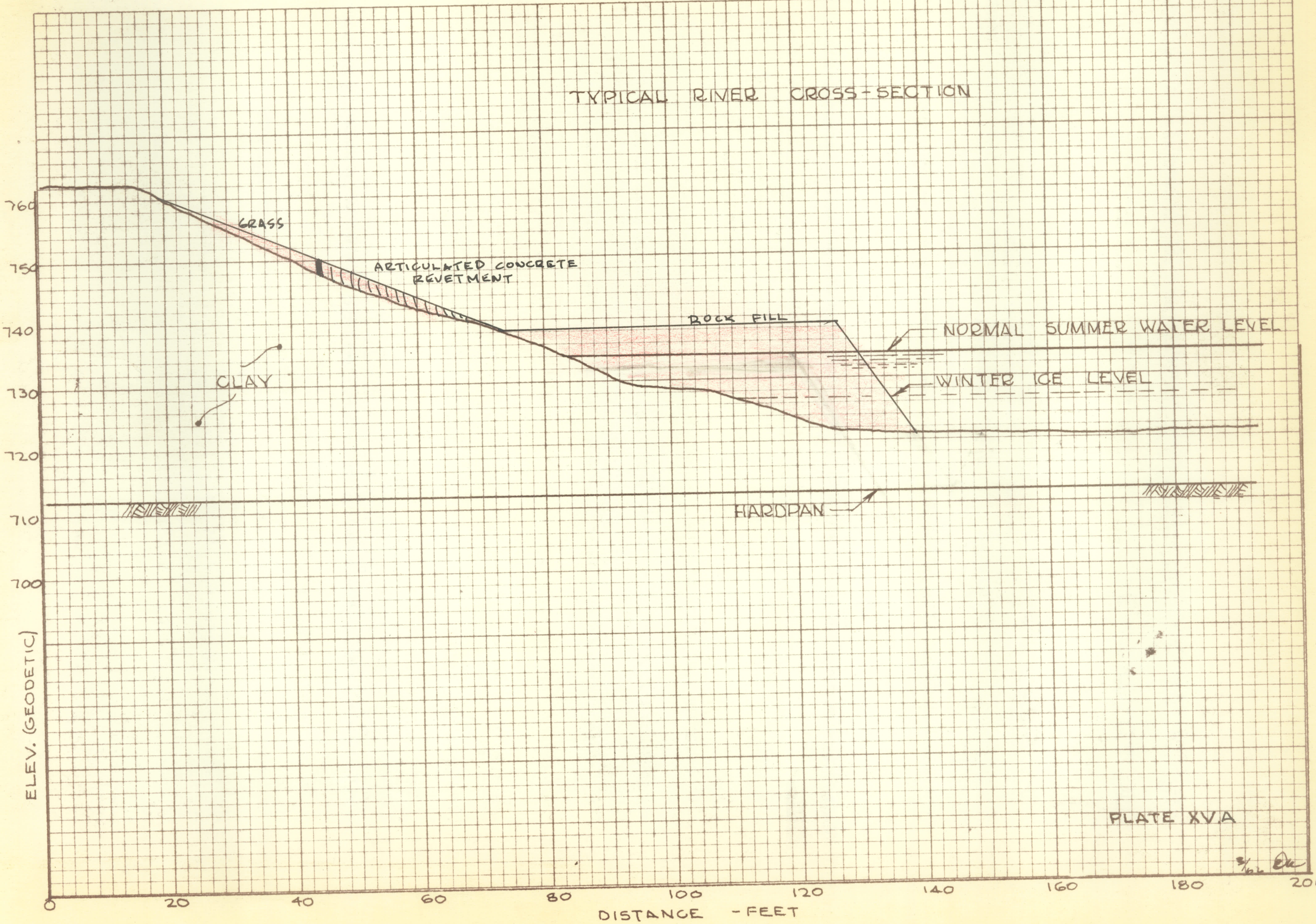
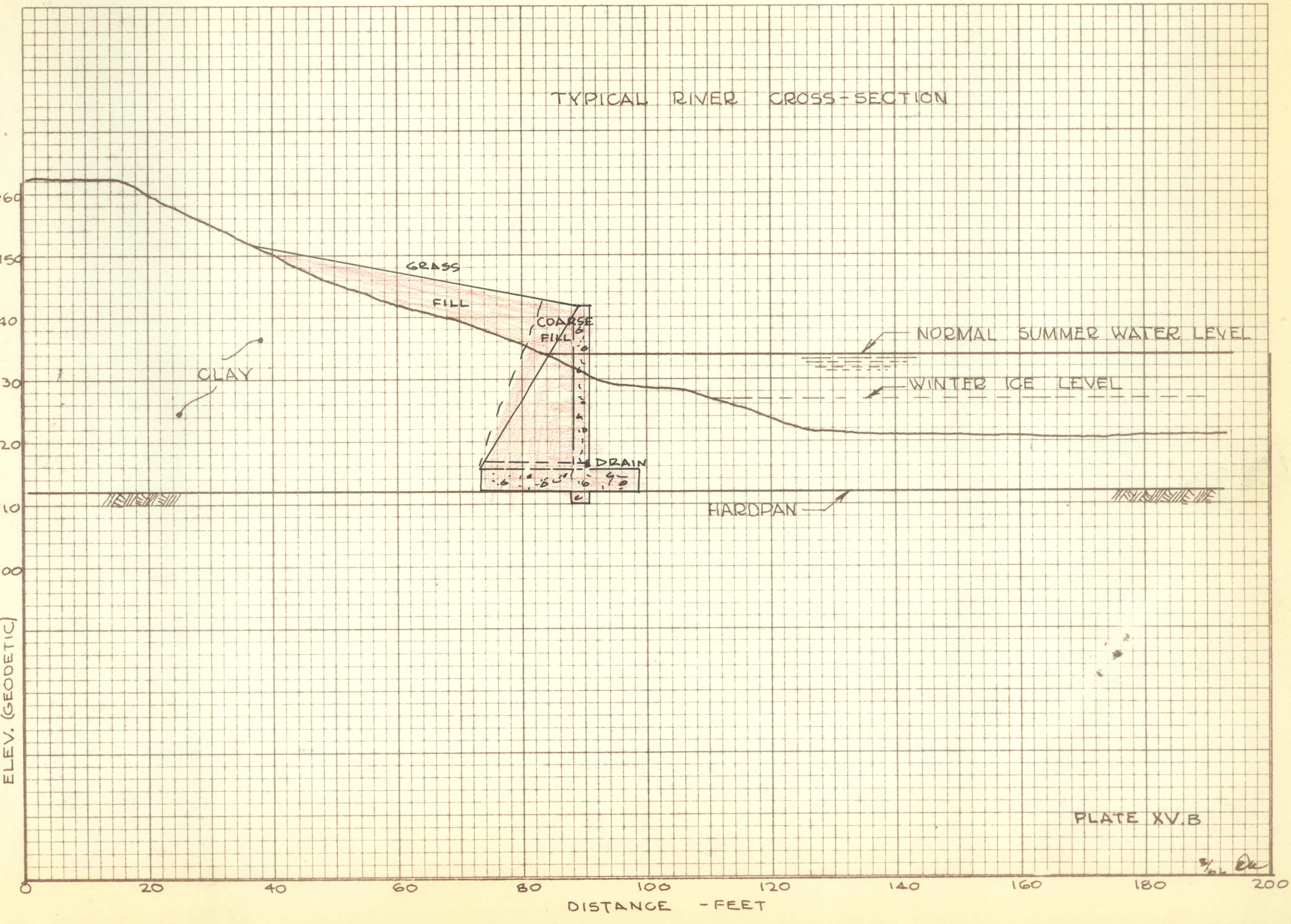
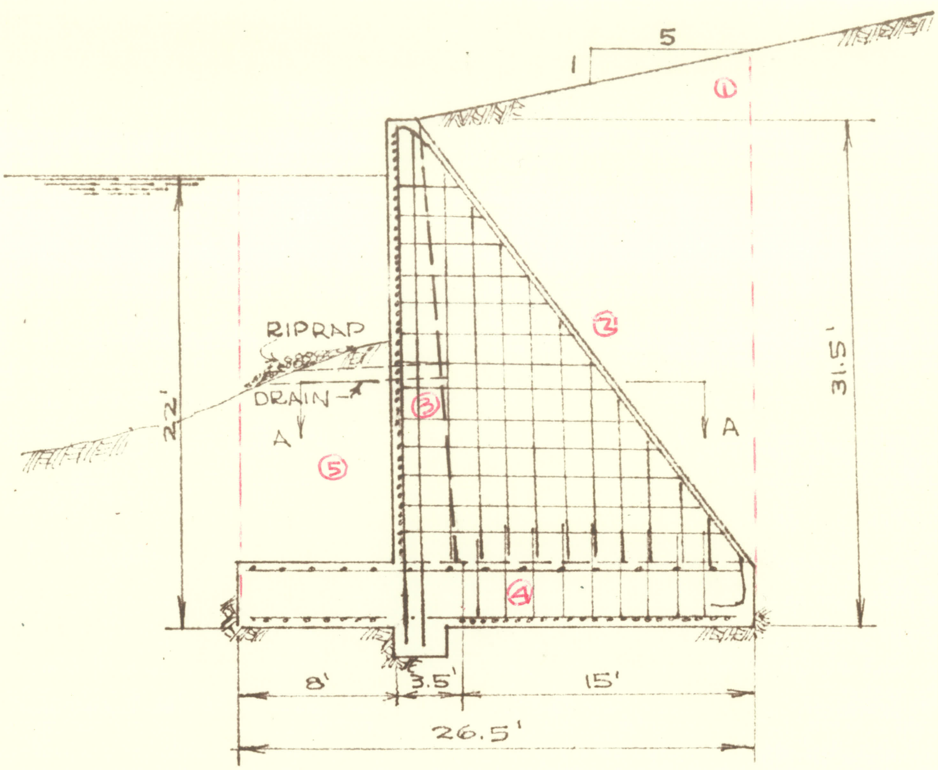

TYPICAL RIVER CROSS-SECTION
GRASS
ARTICULATED CONCRETE REVETMENT
ROCK FILL
NORMAL SUMMER WATER LEVEL
WINTER ICE LEVEL
CLAY
HARDPAN
760
750
740
730
720
710
700
ELEV. (GEODETIC)
0
20
40
60
80
100
120
140
160
180
200
DISTANCE -FEET

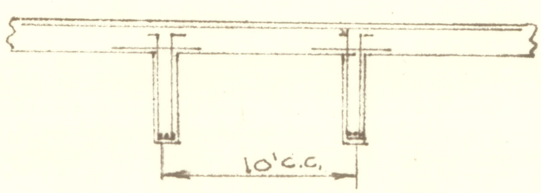
PLATE XV.A



TYPICAL RIVER CROSS-SECTION
GRASS
FILL
COARSE FILL
CLAY
DRAIN
NORMAL SUMMER WATER LEVEL
WINTER ICE LEVEL
HARDPAN
760
750
740
730
720
710
700
ELEV. (GEODETIC)
0
20
40
60
80
100
120
140
160
180
200
DISTANCE - FEET
PLATE XV.B



CROSS - SECTION - RETAINING WALL



SECTION A-A.

PLATE XVII

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section of the retaining wall is shown on Plate XV-B, page 85. The earth above the retaining wall will have a slope of 1 in 5. This will require also a hardy grass cover to prevent erosion. Some erosion may be evident at the toe of the wall, in which case some rock fill would be required. (Design section shown on Plate XVII Page 85A.)

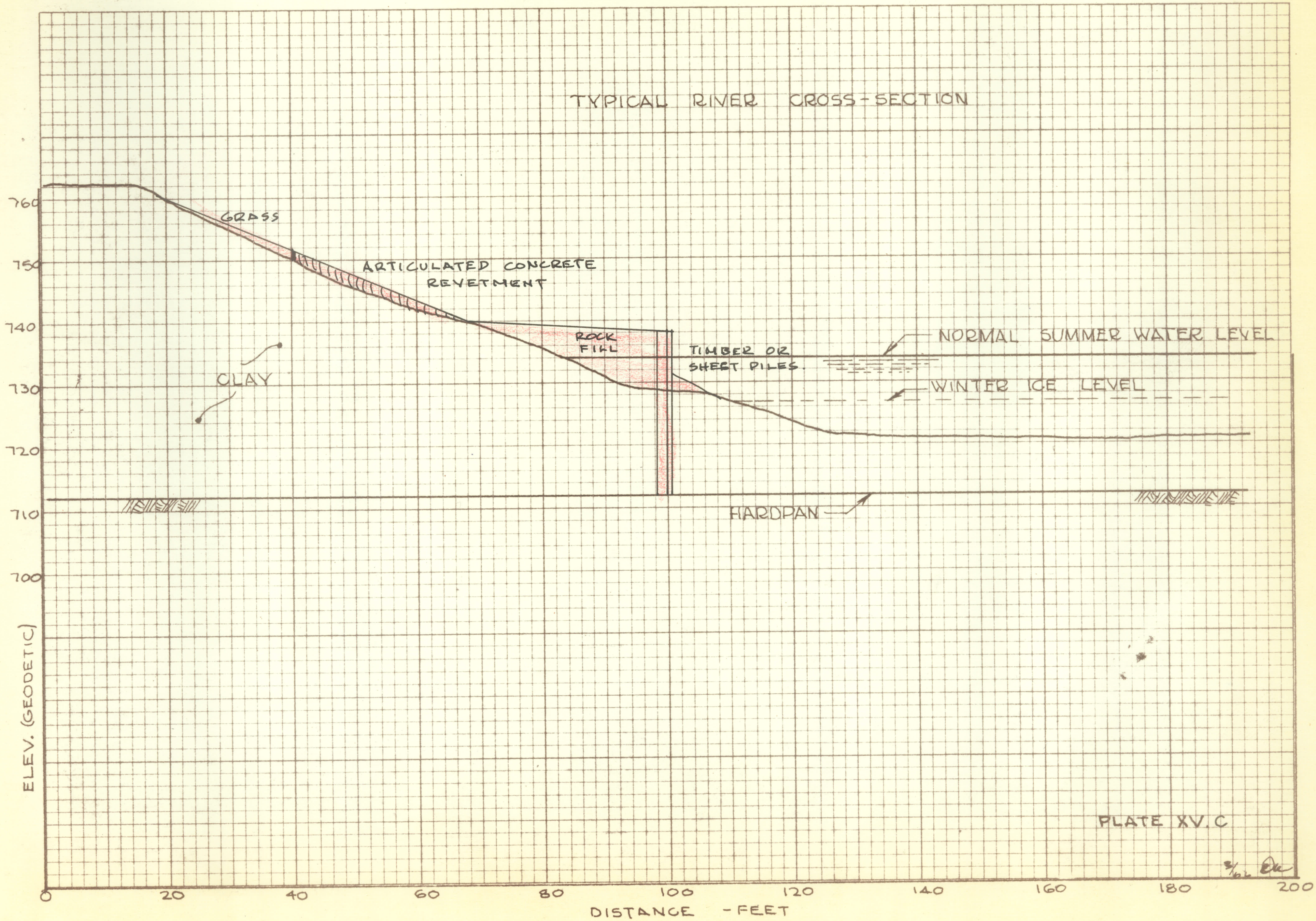
A sheet or timber pile retaining wall, together with some rock fill, shown on Plate XV-C, page 86A while being less expensive, is less effective. That is, it will protect banks against creep or minor slides which occur during relatively small floods, but will not protect against the larger floods. This wall requires 25 feet of revetment on the upper bank; either concrete mattress or asphalt. Again, where possible, slopes are to be flattened as much as possible.

With approximately the same protection, an alternative to the pile retaining wall, might be a sheet pile retaining wall also shown on Plate XV-C , page 86A.

Another alternative is the combination of retaining wall and earth, gravel and rubble fill, While earth fill does not have the draining qualities of rock, it will be considered as having beneficial effects.

The final alternative is a variation of the first method mentioned, except that toe fill is somewhat heavier.

The projects for which analysis will be made are then as



TYPICAL RIVER CROSS-SECTION
GRASS
ARTICULATED CONCRETE REVETMENT
ROCK FILL
TIMBER OR SHEET PILES.
NORMAL SUMMER WATER LEVEL
WINTER ICE LEVEL
CLAY
HARDPAN
760
750
740
730
720
710
700
ELEV. (GEODETIC)
0
20
40
60
80
100
120
140
160
180
200
DISTANCE -FEET
PLATE XV. C

follows:

1) Extensive rock fill, articulated concrete revetment, sod.
2) Concrete counterfort retaining wall and sod.
3) Pile retaining wall, articulated concrete revetment, sod.
4) Sheet pile retaining wall articulated concrete revetment, sod.
5) Sheet pile retaining wall, earth and rubble fill, sod.
6) Replacing all buildings by roadways and parks.
7) Heavier rock fill than (1), articulated concrete revetment, sod.

For all the above major projects, it will be assumed that minor protection for minor projects is included. It is assumed that minor protection to prevent toe erosion consists of rock riprap or very light pile walls, which can be installed for approximately the same price.

D. Purchase of All River Property

The purchasing of all river bank property and its conversion to parks has been suggested and discussed by many individuals. This is a suggestion requiring serious consideration. Indeed, none of the aforementioned methods of improvement would alleviate the problem as satisfactorily as this method. The presence of the structures, poses the main difficulty in arriving at a practical and economical solution of the bank stabilization problem. Without their presence, banks could be graded to stable slopes, proper surface drainage could be easily initiated, and such structures as may be required could be installed with freedom of movement, interference and inconvenience. The construction of roadways and parks following removal of existing houses and industry would undoubtedly add to the aesthetic value of the city as a whole. With public parks bordering the river, the pleasures provided by parks in these locations could be enjoyed by all city residents and tourists alike, and not only by the relatively small number of river property owners.

With all due consideration given to aesthetic qualities, it must be realized, however, that economics normally dictate the feasibility of a project such as this. The estimated cost given in the following cost study, of purchasing existing property is, in the author's opinion, a conservative one. The costs, therefore, of property, excluding major bank improvement are already substantially high.

CHAPTER V

COSTS

The following unit prices have been obtained from local authorities who have had experience in the type of work concerned.

TABLE IV

UNIT PRICES

Treated pile and timber retaining wall (pile lengths 25' - 30')	$ 40/l.f.
Sheet pile retaining wall (pile length 25' to 30')	$150/l.f.
Cofferdam	$100/l.f.
Reinforced Concrete (including steel and forming)	$ 65.00
Earth Excavation	$ 2.50/cu. yd.
Bank Grading	$ 10.00/l.f.
Hardpan Excavation	$ 10.00/cu. yd.
Rock Fill (in place)	$ 3.00/cu. yd.
Trench Excavation	$ 2.50/cu. yd.
Gravel and Sand (in place)	$ 2.50/cu. yd.
Earth and Rubble Fill	$ 2.00/cu. yd.
Articulated Concrete Revetment	$ 0.25/sq. ft.
Grass	$ 0.02/sq. ft.

QUANTITIES AND COSTS PER LINEAL FOOT OF BANK

PROJECT I (Plate XV, A) Extensive Rock Fill, Revetment, Sod

Fill	20.5 cubic yards	@ $3.00	$ 61.50
Revetment	25 square feet	@ $ .24	$ 6.25
Grass	35 square feet	@ $ .02	$ .70
		Total	$ 68.45

PROJECT II (Plate XV, B.) Concrete Retaining Wall, Sod

Excavation	18.5 cubic yards	@ $ 2.50	$ 46.30
Concrete	6.7 cubic yards	@ $65.00	$435.00
Cofferdam	(2 rows - 1 each		$200.00
Backfill	side of excavation)	@ 2.50	$ 14.80
		Total	$696.10

PROJECT III (Plate XV, C.) Timber, Pile Retaining, Revetment, Sod

Rock fill	2.8 cubic yards	@ $ 3.00	$ 8.40
Revetment	30 square feet	@ $ .25	$ 7.50
Pile walls			$ 45.00
Grass	25 square feet	@ $ .02	$ .50
		Total	$ 61.40

PROJECT IV (Similar to Plate XV, C) Sheet Pile Retaining Wall, Revetment, Sod

Rock fill	2.8 cubic yards	@ $ 3.00	$ 8.40
Revetment	30 square feet	@ $ .25	$ 7.50
Sheet pile			$150.00

Project IV - continued

Grass	25	square feet	@ $ .02	$ .50
			Total	$166.40

PROJECT V (Similar to Plate XV, C) Sheet Pile Retaining Wall, Rubble Fill, Sod

Fill	2.8	cubic yards	@ $ 2.00	$ 5.60
Revetment	30	square feet	@ $ .25	$ 7.50
Sheet Pile				$150.00
Grass	25	square feet	@ $ .02	$ .50
			Total	$163.60

PROJECT VI - Replacing Buildings with Parks

Costs	Houses and property 850 @ $20,000.00	$17,000,000.00
	Blocks 4,000 @ $1,200.00	$ 4,800,000.00
	Industry 15,000 @ $1,200.00	$18,000,000.00
	Grading 337,000 ft. @ $10.00	$ 3,370,000.00
	Revetment and Sod 337,000 feet @ $ .27	$ 100,000.00
	Total	$43,270,000.00

PROJECT VII (Plate XV, D) Similar to I, Heavier Rock Fill

Fill	27.5	cubic yards	@ $ 3.00	$ 82.50
Revetment	25	square feet	@ $ .25	$ 6.25

Project VII - continued

Grass	35	square feet	@ $ .02	$ .70
			Total	$ 89.45

GRAVEL FILTER DRAINS (Plate IX)

Excavation	48 cubic yards per 100 feet = .48 cubic yards per lineal foot@ $2.50	$ 1.20
Fill	48 cubic yards per 100 feet = .48 cubic yards @ $2.50	$ 1.20
	Total	$ 2.40

GRAVEL FILTER DRAINS (To Hardpan Plate IX)

Excavation	1.2 cubic yards	@ $2.50	$ 3.00
Fill	1.2 cubic yards	@ $2.50	$ 3.00
Shoring			$ 2.00
		Total	$ 8.00

MINOR PROTECTION

Rock riprap	1.5 cubic yards	@ $3.00	$ 4.50

TABLE FOR TOTAL COSTS

All projects will have an assumed useful life of 50 years. The total costs are the sum of the total capital costs of the completed projects plus the cost of operation and maintenance in perpetuity. Operation and maintenance has been estimated at 1 per cent of the capital cost and interest is estimated to be a rate of 4 per cent.

The capital costs include an engineering charge of 15 per cent. Length of major improvements - 28 x 5280 = 148,000 feet

Length of minor improvements - 36 x 5280 - 189,000 feet

For major and minor improvements, see Plate I, page 141

TABLE IV

TOTAL COSTS

Project Number	Cost Per Lineal Foot of Bank	Total Capital Cost	Present Value O. and M.	Total Cost
I Major (Rock Fill)	$ 68.45	$ 12,000,000	$ 3,000,000	$ 15,000,000
Minor	$ 4.50			
II Major (Retaining Wall)	$696.10	$117,000,000	$29,000,000	$147,000,000
Minor	$ 4.50			
III Major (Piling)	$ 61.40	$ 11,000,000	$ 3,000,000	$ 14,000,000
Minor	$ 4.50			
IV Major (Piling)	$166.40	$ 29,000,000	$ 7,000,000	$ 36,000,000
Minor	$ 4.50			
V Major (Piling)	$163.60	$ 28,000,000	$ 7,000,000	$ 35,000,000
Minor	$ 4.50			
VI Major (Purchase Property)		$ 43,000,000	$10,000,000	$ 53,000,000
Minor				
VII Major (Rock Fill)	$ 89.45	$ 16,000,000	$ 4,000,000	$ 20,000,000
Minor	$ 4.50			

CHAPTER VI

BENEFITS

It is, to say the least, not a simple matter to estimate accurately benefits accrued from bank protection works. First it is required to estimate the damage that will result in the future from unstable and caving river banks. It must then be estimated what reduction in damage will be obtained by the various protection works. This saving in damage is considered to be the benefit of the project.

In this study, the method of obtaining information of past river bank failures was simple, though tedious. A survey was made by the author, of river bank landowners asking questions regarding the past activity of the river bank. A copy of the questions asked is shown on Plate IV. The survey, while not being exceptionally rewarding, gives a relatively good insight into the behaviour of river banks in the past. One of the reasons for the survey not being more successful was the fact that many of the landowners, indeed the greater percentage, had lived in their dwellings for only a few years.

For the information required, the desired owners would be those who have resided at their location for not less than 5 years;

and the longer the more helpful. Therefore, while well over one hundred owners were interviewed, only that information which, in the author's opinion was representative, was selected. The areas chosen for the survey were such areas as were thought reasonably "normal" with regards to having an average population and home density, and the appropriate river conditions. (See Plate I page 141) Locations on both the Red and Assiniboine Rivers were chosen. A simple graphical representation of the results of the survey was shown on Plate V, page 14.

A study was also made of the distances between existing houses and top of bank. While it was not practical to do a survey of all houses on the river banks, it is thought the 60 homes surveyed and approximately 100 houses studied on contour maps would be a relatively representative sample. It appears that the mean distance from the houses to the top of the river bank is 44 feet. Also, two thirds of the houses are closer than 50 feet to the top edge of the bank, while approximately seven-eighths are closer than 70 feet. It was also estimated that three quarters of the residential dwellings were located on banks susceptible to slides. The average cost per river lot plus dwelling is estimated at $20,000.

The damage to industry on river banks is more difficult to estimate. In a comprehensive study, each industry would require individual attention regarding costs due to damages, due to moving, or due to renewal and renovation of the river bank. For industry, therefore, costs were simplified by assuming the cost per lineal foot of bank as 4 times that of residential areas. That is, if a home has an average 60 foot lot, and the destruction of the home by flood is estimated at $20,000 the cost per lineal foot of bank for industry is approximately $1,200.00.

In the final analysis, the overall average annual bank recession was found from the information obtained by survey, using nineteen figures chosen from the survey. These figures while being of questionable accuracy are shown in the following table:

TABLE VI

ANNUAL RECESSION

Item No.	Bank Recession Feet	Period of Years	Recession Per Year Feet
1	35	4	8.7
2	75	11	6.8
3	38	6	6.3
4	100	17	5.9
5	8	2	4.0
6	40	11	3.6
7	40	14	2.9
8	10	6	1.7
9	25	21	1.2

Table VI - continued

Item No.	Bank Recession Feet	Period of Years	Recession per Year Feet
10	30	26	1.2
11	10	9	1.1
12	5	5	1.0
13	10	17	0.6
14	10	19	0.5
15	5	12	0.5
16	3	12	0.2
17	0	5	0.0
18	0	10	0.0
19	0	10	0.0
			46.2

Mean annual bank recession $\frac{46.2}{19} = 2.4$ feet

At first glance, it would appear that locations with zero bank recession are not well represented. Since the main concern in the bank stability study is the concave bank, where the majority of bank failures occur, it is of assistance to investigate the report published by the Government of Manitoba, Water Control and Conservation Branch (21). Of the 136 river bank locations studied in this report, 113 were at concave banks. In the total number of sites, only 6 were classified as being stable, these being at concave banks. According to the report, therefore, only 5.3 per cent of all concave banks were found to be stable; the majority had failed and remained unstable, had failed and had become stable, or were unstable and deteriorating due to toe erosion.

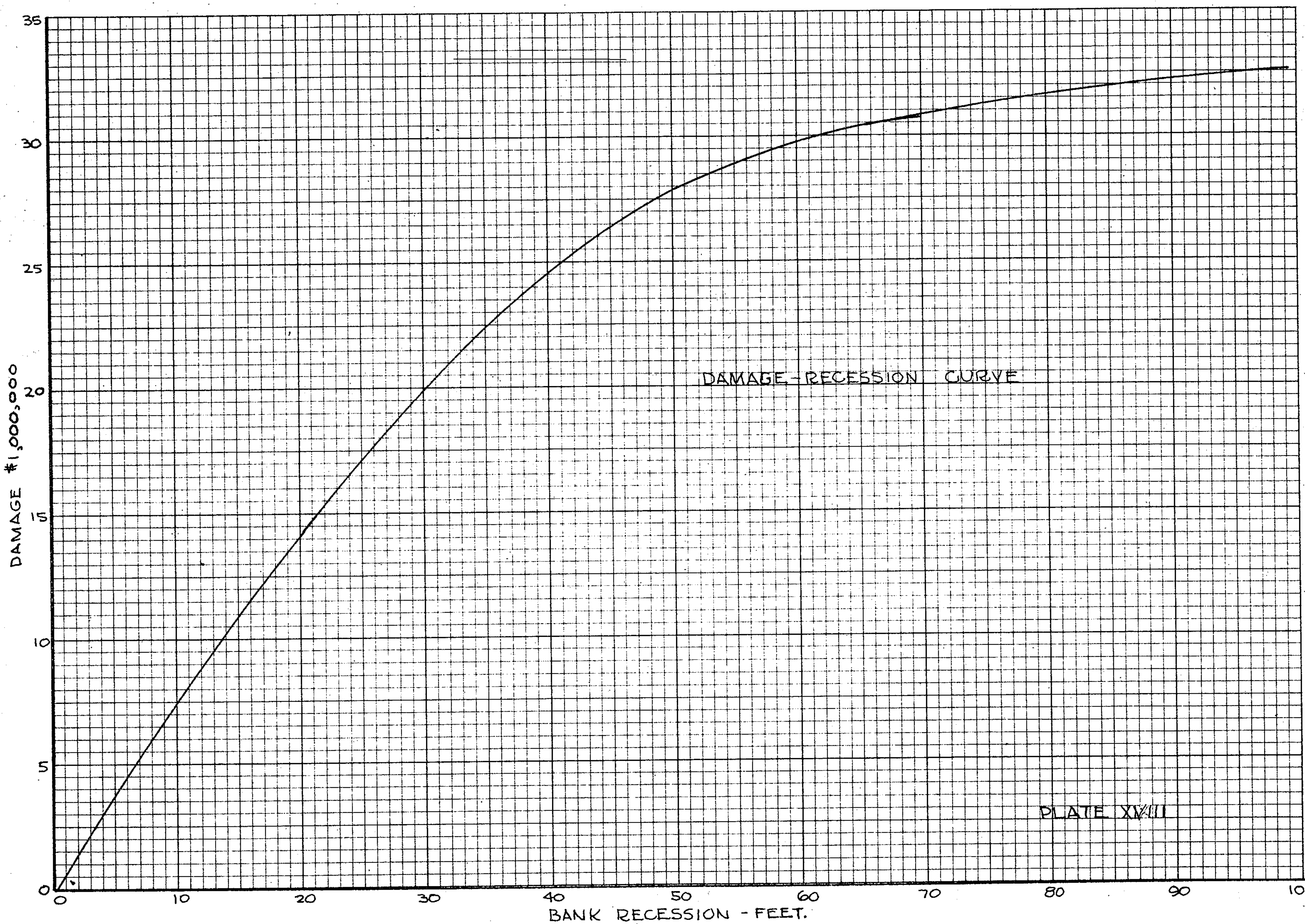
The above nineteen figures from the author's survey were all made on concave banks and among them 16 per cent showed no bank recession. It is the author's opinion that the nineteen figures, while being limited in accuracy to the estimates made by river property owners, offer a satisfactory representation of the rate of occurrence of bank recession. The actual amounts of recession, while being more speculative than the rate of occurrence, are considered to represent severe conditions, such as could occur at almost any concave bank in the Winnipeg area.

Some consideration was given to studying each concave bend separately with regard to rates of bank recession. Preliminary work showed that this would be far too extensive a project to encompass in this study. Rather, it was thought that since practically all concave river banks within the area studied had been or were at present unstable, only a limited number of severe conditions should be chosen. It was further considered that, pending the occurrence of optimum conditions, the severe bank recession portrayed by the selected locations, could occur at any and all concave river banks. The yearly bank recession of the nineteen selections was therefore, averaged to give a mean annual bank recession of 2.4 feet.

With the use of aerial photographs (1961), the number of structures along river banks was obtained. These amounted to approximately 850 houses within the improvement area considered, and 41 apartment dwellings. In the benefit study it is assumed that if the top of bank receded to the house limit, the house will be lost. Therefore, if for example the banks receded 10 feet, 16.7 per cent of the houses would be lost. For apartment blocks, as for industry, losses will be estimated at $1,200.00 per linear foot of river frontage. Because of the expense of the blocks and industry it is assumed they would be reinforced and renovated and the $1,200.00 would cover this.

Table VII shows the computations for damages to dwellings, blocks and industry. This table is represented on a graph, shown on Plate XVIII, page 100.

The average annual bank recession, from Table VI before floodway construction, is estimated at 2.4 feet. It is estimated that the floodway, the construction of which will be assumed completely operation in 10 years, will reduce the probability of large magnitude floods by nearly 5/12, and will consequently reduce average



DAMAGE-RECESSION CURVE
PLATE XVIII
DAMAGE #1,000,000
BANK RECESSION - FEET.
0
5
10
15
20
25
30
35
0
10
20
30
40
50
60
70
80
90
100

annual bank recession from 2.4 feet per year to 1.0 feet per year.

The damage therefore, will be calculated on this estimated recession. The total damage will be the present value of the damage increment taken at 8 year intervals at an interest rate of 5 per cent. The computations are shown in Table VII.

Often for hydraulic projects, a benefit cost study is made on the basis of average annual benefits and costs. Charles Senour, in his paper "Economics of River Bank Stabilization" states the following:

> It is suggested that the true effect of a bank-stabilization project on the economy might better be gaged by comparing total costs of its life with total earnings; in each case using the applicable interest rates, compounded as a means of compensating for deferment of benefits". (19)

It will be considered therefore that comparision of total benefits during the life of a bank stabilization project with total costs during the same period is a better criterion of economic merit than the ratio between annual "equivalent" benefits and annual costs since interest rates pertaining to benefits differ materially from those pertaining to costs. If the total cost of a project, therefore, is in excess of $17,480,000.00, it will be considered as an uneconomical project.

TABLE VII

DAMAGES

	HOUSES			BLOCKS		INDUSTRY		TOTAL
Bank Recession	Percent of Structures Lost **	Number of Structures Lost	Damage @$20,000 per House	Length **	Damage @ $1,200 per foot	Length ***	Damage @ $1,200 per foot	
10	16.7	107	2.41 m	670	0.80 m	2500	3.00 m	5.94 m
20	41.7	267	5.34 m	1670	2.00 m	6400	7.20 m	14.54 m
30	50.0	320	6.20 m	2000	2.40 m	7500	9.00 m	17.60 m
40	67.0	430	8.60 m	2700	3.24 m	10,000	12.00 m	23.84 m
50	79.0	505	10.10 m	3150	3.80 m	11,900	14.30 m	28.20 m
60	83.0	530	10.60 m	3320	4.00 m	12,500	15.00 m	29.60 m
70	85.0	545	10.90 m	3400	4.10 m	12,800	15.40 m	30.40 m
80	87.0	557	11.14 m	3480	4.20 m	13,000	15.60 m	30.94 m
90	89.0	570	11.40 m	3560	4.30 m	13,400	16.10 m	31.80 m
100	92.0	590	11.80 m	3700	4.45 m	13,800	16.60 m	32.85 m

* Percent times 640 houses (640 = 75 per cent ot total of 850 houses)

** (Estimated length of river banks housing apartment dwellings) x per cent lost = 4,000 x per cent lost

*** (Estimated length of river bank housing industry) x per cent lost = 15,000 x per cent lost

TABLE VIII

BENEFITS

Period	Bank Loss		Damages*	Present Value	
Years	Increment	Cum. Total	Increment from Plate XVIII	Rate	Total
0 - 8	19.2	19.2	13,500,000	.823	11,100,000
8 - 16	10.8	30.0	6,500,000	.557	3,600,000
16 - 24	8.0	38.0	3,500,000	.377	1,300,000
24 - 32	8.0	46.0	3,000,000	.255	800,000
32 - 40	8.0	54.0	2,500,000	.173	400,000
40 - 48	8.0	62.0	1,500,000	.117	200,000
48 - 56	8.0	70.0	1,000,000	.080	80,000
				Total	$17,480,000

* The damages as given here vary from those of Table VII, page 102, since the values from Table VII were plotted on Plate XVIII, and adjusted to arrive at a more or less smooth curve, that is, making the consecutive incremental increases in benefits less erratic.

CHAPTER VII

BENEFIT - COST ANALYSIS

The benefit study has shown that the total benefits accrued from complete bank stabilization amount to approximately 17.5 million dollars. It follows that any project which gives this complete bank protection and has a total cost of 17.5 million dollars or less may be a worthwhile undertaking. Two of the projects studied have total costs of less than the stated benefits. Three projects in the group of seven yield the desired results; namely the retaining wall, which may cost up to $147,000,000, the extra heavy rock fill, which has a total cost of $20,000,000, and the replacing of buildings with parks costing $53,000,000. It is seen that only the rock fill approaches in cost the total potential benefits acquired. It must be noted that the quantity of rock used in this project is an absolute minimum to give permanent protection, according to above calculations. It appears, therefore, that on the basis of the primary assumption that protection of all banks is necessary for any project, it is uneconomical to initiate an overall bank stabilization project.

From a preliminary study of comprehensive river bank stabilization, it can be said, at first glance, that there exists no satisfactory solution. It is true that the costs of a concrete retaining wall, even if considerably refined, would be insurmountable. Also timber or sheet pile retaining walls, while being in the acceptable price range, do not provide satisfactory protection. However, it appears that if a more detailed study were made of rock fill, a more plausible cost figure may be arrived at since the above approach has been approximate and may be too conservative.

To this may be added a refinement in benefits. The benefit study has dealt only with the existing structures and industries. Although a considerable amount of work would be involved, a further study should be made of future construction along river banks. It may be stated that if banks were made permanently stable, there would be a large movement of industry, and particularly homes, to the river banks. Many vacant lots still exist along the banks in Metropolitan Winnipeg. Past records of annual rates of construction along river banks would be required to be interpolated to the future, with a factor added to allow for extra construction due to better bank conditions. A comparison of 1949 maps with the 1962 tally made by the author from aerial photographs, shows that the number of houses has increased by almost 300 over these 13 years. With the above mentioned and other alterations it seems possible that the heavy rock fill method of stabilization may have merits. It certainly merits further study.

Observations

While no overall scheme from the above is acceptable in its present state, it is thought that the system of surface drainage proposed be experimented with. At a cost of approximately $2.40 per lineal foot of bank or about $13,000 per mile, a test area of one concave bank may cost approximately $8,000 to $10,000, excluding the costs of such soil testing as may be required.

Also, it is proposed that a local authority, such as a Rivers and Streams Authority, should have the facilities and the ability to demonstrate to and instruct any and all riparian land owners, free of charge, on methods of improving the conditions of river banks. Only such an authority, which is fully aware of the problems at hand and the most recent developments of improvements, should be allowed to instruct owners as to actions to be taken. Poor or incorrect information frequently creates adverse affects.

There are several items which require specific mention here. There has been some discussion regarding constriction of the channel due to improvements. It is probably true that local improvements may cause minor local effects such as slight deepening. However, constriction due to improvements is considered to be negligible. The rock fill which was used for bank protection in project 7 occupies only 1,7 per cent of the total Red River channel capacity. This would amount to 1,700 cfs. for a flood such as the 1950 flood. On the Assiniboine the flow is reduced proportionally. This should be of no concern when considering improvements.

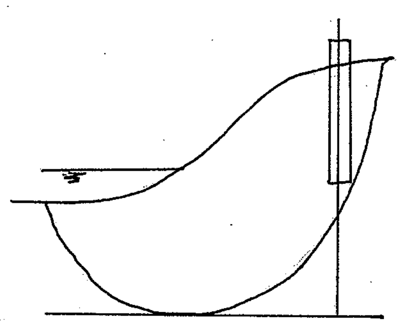
Often in interviews with riparian landowners, the topic of conversation came to the event of the late fall drawdown of water due to opening of the gates at Lockport. The majority of landowners complained about the rapidity of drawdown. A comment from several owners expressed the general feeling on this topic. "Some years drawdown is normal. Other years drawdown occurs in a matter of two to three days and "overnight" the bottom four to five feet of the river bank, and frequently a larger portion of the bank slough into the river". It is probably true that the small zone, between winter and summer water levels, is the most susceptible to annual build up of sediment and that the loss due to sloughing may well be built up again in the following spring. However, this is not likely to occur at all locations. Where sediment does not have an opportunity to accrete, the entire bank can conceivably become unstable in several years and may fail. It is, therefore, of utmost importance that the lowering of the river level to winter level be a slow process (that is, in a minimum time period of three to four weeks,) every year. One year of rapid drawdown, when other conditions are favorable, may well damage the bank severely at many locations.

While relatively low stages can be regulated by the dam at Lockport, it is stipulated that overall regulation of river stages during floods will be more feasible with the use of the floodway. Firstly, the floodway will relieve the main river channel of excessive flows during high floods, resulting in lower stages. Also, rates of flow may be controlled, after the peak of the flood is past, by some storage behind the dam. Care should be taken in controlling the flow, in order to obtain the optimum possible beneficial effects from the floodway. The desirable effect sought in every case is to obtain as slow a fluctation of water level as is possible.

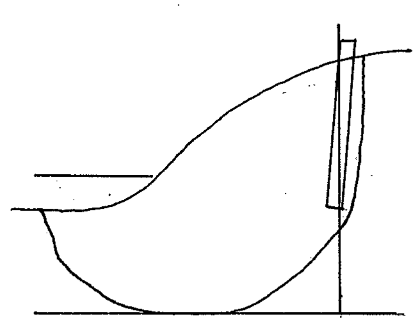
As mentioned under "Re-sedimentation" above, during high flows a considerable amount of sediment is transported by the stream. Considerable space has been allotted to this subject with the thought that, some methods may be available for advantageously utilizing the sediment. It is suggested that for the sediment that is available, test strips be initiated for accretion. Use could be made of any of the methods mentioned but it must be considered that spring ice conditions may to an extent dictate the methods to be used. It is suggested that at a location such as the Canoe Club Golf Course, trials could be made with low, permeable timber pile dikes of stone groins. With adequate sediment, re-sedimentation could be of assistance in improving banks as well as realigning the channel at necessary locations. Realigning of channels by the use of dikes can well be experimented with at several bridge locations, of which the Provencher Bridge is the most desirable. Here the current from the upstream bend has gouged a bay of 40 feet by 500 feet just upstream of the bridge. This bay will coninue to grow to more dangerous proportions unless halted by some means.

As at this particular location, erosion of banks by wave and current is evident almost everywhere. Many signs such as fallen trees, sheer banks with abandoned houses, local bays, and others, are indications that the constant process of erosion is occurring. In a study of river bank conditions on the Red River made by the Manitoba Government, one of the items studied was toe erosion. Of the 131 locations investigated, only 9 percent of the locations had no noticeable toe erosion, 27 percent had "light" erosion, 17 percent had "medium" erosion, and 47 percent of the locations were classified as having "severe" toe erosion. Elimination of erosion at the toe of banks is thus an important feature of bank improvement. Light rubble toe fill has been placed in a concave bend in North Winnipeg recently and, according to adjacent owners, has been highly successful in halting toe erosion. Higher flows may still erode the bank behind the fill. This will be shown when floods occur.

While erosion is a never ending occurrence and visible to the naked eye, this is not true for bank slides. Often a slide is preceded by creep. This can only be detected by field measurement. If creep is detected, measures may in some instances be taken to alleviate the condition before a severe slide occurs. The bank can be partially unloaded, for example. This would, in any case, be more desireable to the owner, than loss of the entire bank. Several methods are available for detecting bank movements. The following is perhaps one of the simpler methods available. A hole is drilled several feet back of the top of bank, in a susceptible slide area. This hole is approximately 6 inches in diameter, drilled almost to hardpan, and lined with steel tubing. A metal rod is then driven into the clay at the bottom of the drilled hole, (See diagram 1) so that the rod is centered in the pipe. When movement occurs, it can be detected by the relative location of rod and tubing. (See diagram 2)



DIAG. 1



DIAG. 2

If movement occurs, and if the rate of movement increase, without any change in external conditions, the slope must be reduced by excavation. (The Wilson slope indicator has also seen much use.)

It was mentioned above that fallen trees are an indication of erosion. At many locations, evidence exists of trees moving slowly towards the water, either dropping vertically due to slides or tilting and falling suddenly due to erosion. Many locations show all stages of this seemingly general movement of trees from top of banks to the river level. Indeed, some locations have only tilting trees left and only adjacent to the water's edge. It is possible that another method of estimating bank recession may be the correlating of the ages of the trees with the observed tree movement over a period of say 5 to 10 years, and then interpolating.

In the above improvements, trees have not been mentioned. Trees, of course, have much intangible value in beautifying an environment, particularly near the river. It is thought that, where trees do not interfere with improvements they may remain. They are at very few locations dense enough to be major contributing factors to bank failure, although some locations exist where wind blowing against the trees may cause a horizontal force. At most locations buildings prevent direct force of wind against trees. However, where they interfere with whatever improvements, if any, are initiated, they must be removed without regret.

Where steps are taken to prevent erosion, it is not unlikely that much of the erosion would be caused by water craft. Indeed, if there has been an increase in erosion in past years, one of the main contributing factors has been the waves created by boats. The author has had the opportunity to observe the action of waves against several portions of bank where boat traffic was relatively heavy. One such location was adjacent to Churchill Drive, on the opposite side of a marina. Here was a constant lapping of waves against shore. Earth was observed peeling from the sheer bank of 5 to 15 feet in height. The number of craft has been and is increasing at such a rapid pace that, should some bank improvements be initiated, it would not be unfair to establish a license charge for all boats, charge depending on length of boat, to assist in paying for maintenance of improvements.

CHAPTER VIII

CONCLUSIONS

As in any preliminary studies, the accent can not be placed on details. Here, then, a broad view has been obtained of the river bank problem and some of its possible solutions. The study has shown that most engineering solutions that cope with the situation are not economical. For a furtherance of this report, the following proposals are offered. A search should be made for new solutions. From the above projects, more research should be directed to the solution by heavy rock fill, possibly resulting in lower costs. Further investigations should be made into the increase in benefits accrued from the added construction on river banks, instigated by bank improvement. As stated in the discussion, a preliminary study could be initiated to determine the possibility of evaluating bank recession from the gradual movement of trees down the bank and the age of the trees.

In regards to re-sedimentation, the information from the Dominion Government on suspended silt, warrants the setting up of test areas. The concave banks at the Canoe Club, opposite North Drive, or immediately upstream from the confluence of the rivers, are excellent locations for testing of pile dikes, fascine boxes, groins or fabric fences. Here, however, provisions must be made for protection against ice.

If after further studies, the principle of overall protection, yields no economical results, and tests show that sediment accretion is light, smaller scale projects could be reviewed. If it can be shown that no adverse affects result from piecemeal projects, improvements could be made on a curve to curve basis, depending on the interest shown and support given by the residents of those concave bends.

While on the survey of river property owners, the author presented questions regarding desired cost allocations of improvement projects. An analysis of the answers received shows the following breakdown; Federal Government 48.8 percent, Provincial Government 35.1 per cent, Metropolitan Government 10.5 per cent, and owner 5.6 per cent.

In the author's opinion, this breakdown is not acceptable. The main benefit is received by the owner, while secondary benefits are received by the municipality concerned. It is therefore, suggested that the owner pay between 30 and 50 per cent of the cost while Metropolitan Government, with considerable municipal contributions, pays the remainder.

In answer to questions 4, 5, and 6 the following results were obtained. For question 4, 72 per cent of the people stated that they should not be reimbursed for property lost over the past years. They believed, as does the author, that they should have or did understand some of the consequences arising out of unstable river banks. In answer to question 5, 60 per cent stated that they would not object to large scale bank improvement even if it meant altering up to 15 feet of their property. A review of the banks has shown that if improvements were made this would be the case at all concave bends. Question 6 states "Would you agree to increased taxation to cover bank improvement?" Sixty-five per cent of the owners answered negatively. The author believes that an extra tax to river property owners should be considered, since they are the main recipients of the benefits. The charge of a proposed boat licence would be a reasonable division of costs between river property owners and others, who live elsewhere but use the river for pleasure purposes.

APPENDIX "A"

DETERMINATION OF DEPTH OF HARDPAN OR BEDROCK BELOW SUMMER WATER LEVEL

RED RIVER

Depth Ft.	Length This Depth Mi.	Units	Depth X Units
22	0.5	5	110
24	1.0	10	240
26	0.2	2	52
30	0.4	4	120
27	0.5	5	135
24	0.2	2	48
21	0.6	6	126
18	0.7	7	126
20	4.0	40	800
25	9.2	92	2300
	TOTAL	173	4057

Average depth = $\frac{4057}{173}$ = 23.5 feet

ASSINIBOINE RIVER

21	1.0	10	210
18	1.3	13	234
11	0.9	9	99

Depth Ft.	Length This Depth Mi.	Units	Depth X Units
9	0.7	7	63
1	0.4	4	4
		43	610

Average Depth = 14.2 ft.

Overall Average Depth = 22.0 feet.

APPENDIX "B"

DESIGN OF RETAINING WALL

After several trials, the cross section shown on Plate XVII, Page 85A was found to have the required stability.

Data

Allowable bearing pressure on hardpan is 10 kips per square foot. Friction between base and hardpan = 0.60

f_s = 20 ksi	K = 197 psi
f_c = 1200 psi	v = 75 psi
n = 10	u = 240 psi
j = 0.87	

STABILITY COMPUTATIONS

Moments about A

Area No	Area Sq. Ft.	Weight Kips.	Mom Arm Ft.	Moment Ft - Kips
1	22.5	2.25	5	11.2
2	420.0	42.0	7.5	315.0
3	40.0	3.6	16.0	57.6
q	16.0	2.4	16.0	38.4
4	50.0	4.5	12.5	56.2
5	160.0	9.6	21.0	201.0
	160.0	9.6	11.0	599.0
	160.0	9.6	7.1	103.1
			ΣM_A	1165.3 Ft-Kips.

Stability Computations - continued

$$P_v = \frac{5.44}{69.79} \text{ kips}$$
$$V$$

Resisting Force = 14.52 kips

Sliding Force = 54.45 kips.

Location of Resultant

From point A $\qquad \frac{1165.3}{69.79} = 16.65$ ft.

$$= 16.65 - \frac{25}{2} = 4.15 = \frac{25}{6}$$

Soil Pressure at Base

At toe, $q_{max} = (41 - 6a)\ \frac{R_{v2}}{1} = (4 \times 25) - (6 \times 8.32)\ 69.79 = \frac{5.58}{625}$ K/sq.ft.

At heel $q\ min = \frac{69.79}{25}\ (1 - \frac{6 \times 4.15}{25}) = + \ .01$ K

Shear available along base = $69.79 \times .6 = 41.9$ Kips

Passive force at toe:

$$F = 1/2 \ {}_sH^2$$

$$= 1/2 \times 60 \times 22^2$$

$$= 14.52^K$$

Total resisting force = 41.9 + 14.52

= 56.42 Kips

Total earth pressure = 54.42 Kips

Factor of safety against sliding $= \frac{56.42}{54.45} = 1.04$

This is less than 1.5 which is the minimum required. Therefore, a key is required at the base. Shear required to increase the safety factor to 1.5 = 54.45 x 1.5 - 56.42

$= 81.5 - 56.42 = 25.08^{K}$ per foot

Require 25,080 pounds per lineal foot of wall.

v - allowable unit shear stress

= 75 psi Area required $= \frac{25{,}080}{75} = 335$ sq. in.

Width (top of key) $= \frac{35\ 5}{12} = 28$ inches, say 2.5 feet

Depth of Key = 1.5 feet required for proper grip.

A panel of the vertical wall between two counterforts is a slab acted upon by the horizontal earth pressure and supported along three sides, i.e., at the two counterforts and the base slab, while the fourth side, the top edge, is unsupported. The earth pressure, of course, increases with the distance from the free surface. The exact determination of moments and shears in such a slab supported on three sides and nonuniformly loaded is rather involved. It is customary in design of such walls to disregard the support of the vertical wall by the base slab and to design it as if it were a continuous slab spanning horizontally between counterforts. This procedure is conservative, because the moments obtained by this approximation are larger than those corresponding to the actual conditions of support, particularly in the lower part of the wall. Hence, in a final design for large installations, significant savings may be achieved by a more accurate analysis.

Slab moments are determined for strips 1 foot wide vertically, usually for the strip at the bottom of the wall and for three or four equally spaced additional strips at higher elevations. The earth pressures on the different strip decreases with increasing elevation and is determined by $p = \gamma H.K.$

Moments and shears cannot be determined from the recommendations of the A. C. I. Code since the entire horizontal load, i. e. earth pressure, must be regarded as live load. On the other hand, an exact analysis by moment distribution or some other method, is not warranted in view of (a) the approximate assumptions of design and (b) the uncertain magnitude of the earth pressure, which may vary from panel to panel. It is considered adequate to assume the positive moments midway between and the negative moments at the counterforts to be equal and to use for these moments $M = PL^2/12$ for the two lowermost strips and $M = PL^2/10$ for the strips at higher elevations. The smaller moment value for the bottom strips accounts for the fact that the additional support provided by the base slab decreases the moments in the bottom portions of the slab. The shear force at each support of a strip is simply computed from $V = pL2$. Horizontal bars are provided as required by these moments, with increased spacing or decreased diameter at higher elevations. Alternate bars are bent up to provide for the negative moments at the counterforts.

The heel slab is supported in a manner similar to that of the wall slab, i.e. by the counterforts and the wall. It is loaded downward by the weight of the fill resting on it and its own weight. This load is counteracted by the bearing pressure. Here also the support along the third side is neglected. Moments and shears are determined for strips parallel to the wall, each strip representing a continuous beam supported at the counterforts. The above formulae for shears and moments are used.

Assume 4 strips will be used, one at the top and bottom and two strips between these.

<u>Bottom Strip</u>

Force per lineal foot = $100 \times 28 = 2.8^{K}$ / ft.

$K = 197$ $b = 12$ inches (assumed)

$M = Kbd^2$

$D = \frac{M}{Kb}$

After several trials a counterfort spacing of 10 feet was found to be ample.

$$M = \frac{2800 \times 120^2}{12}$$ Vermels

$$d = \frac{2800 \times 1200}{12 \times 197}$$

$$= 37.7'' \text{ say } 38 \text{ inches}$$

With cover thickness of wall at base = 42''

For shear, $V = \frac{pL}{2} = \frac{2800 \times 10}{2} = 14{,}000$ pounds

$$v = \frac{V}{bjd} = \frac{14000}{12 \times .87 \times 38} = 32 \text{ psi}.$$

Allowable = 75 psi

No web reinforcement is required.

Tension Steel

From "Reinforced Concrete Design Handbook"

Per cent steel = p = .0113

Compression area = Kbd

= .375 x 38 x 12

= 171 sq. in.

$A_s = pA_c = .0113 \times 171 = 1.93$ sq. in.

Use 2 - #9 bars.

Lower 9 foot strip (Calculations same as above)

Wall thickness = 33 in. d = 30 in.

No web reinforcing is required.

Steel - use 3 #7 bars.

Upper 9 foot strip (Calculations same as above)

d = 23 in. wall thickness 26 in.

No web reinforcing required

Steel - use 3 #6 bars.

Top strip (At 5 ft. from top) (Calculations same as above)

d = 16 in wall thickness = 19 in

No web reinforcing required

Steel - use 3 - #5 bars

Heel Slab

Consider a longitudinal strip 1 foot wide. Since 20 feet of height of the wall has been given a submerged unit weight, the maximum moments will be produced directly at the heel of the heel slab portion.

Load per foot at heel = 100 x 28 = 2,800 lbs.

Assume 3 foot slab thickness. Then weight of slab = 3 x 1 x 150 = 450 lb per ft Total load = 3,250 lbs per ft.

As above, maximum positive and negative moments are considered equal. $M = \frac{pl^2}{12} = \frac{3250 \times 120^2}{12} = 3,900,000$ in - lbs

$$M = Kbd^2$$

$$d = \frac{M}{Kb} = \frac{3,900,000}{197 \times 12} = 39 \text{ in.}$$ Then slab is 42" thick.

For shear, $V = \frac{pl}{2} = \frac{3250 \times 10}{2} = 16,250$ lb.

$$v = \frac{V}{bjd} = \frac{16250}{12 \times .87 \times 39} = 40 \text{ psi}$$

No web reinforcement required.

A depth therefore of 42 inches could be assumed for the heel slab However, when placing the wall on solid hardpan, a large measure of regidity is attained and if designed accordingly, the slab thickness required would be reduced considerably. It will therefore, be assumed for this preliminary design that a slab thickness of 36 inches will suffice. Therefore, d = 33".

Steel requirements:

$A_c = kbd = A_s = pAc = pkbd = .0113 \times .375 \times 12 \times 33 = 1.69$ sq. in.

Use 3 #7 bars.

Toe slab.

The toe slab is subject to moment from the earth above it, and also moment, near the wall, due to the earth pressure on the wall.

The moments right at the wall are somewhat larger than for the heel slab but this can be taken up by some additional steel. The thickness of the toe slab will therefore be assumed as 36 inches.

Counterforts

The counterforts are wedge-shaped cantilevers built in at the base slab. They support the wall slab and are, therefore, loaded by the total soil pressure over a length equal to the center distance between counterforts. They act as a T-beam of which the wall slab is the flange and the counterfort the stem. For this reason concrete stresses are always low and need not be checked. The maximum bending moment is that of total earth pressure, taken about the bottom of the wall slab. Since the moment decreases rapidly in the upper parts of the counterforts, some of the bars can be discontinued.

Maximum moment = 54450 x 11 x 12 = 7,200,000 in.-lbs.

$$d = \frac{7,200,000}{197 \times 12} = 55 \text{ ins.}$$

Width of counterfort at base is 15 feet.

A thickness of counterfort of less than 12 inches would be possible but because of the 28 foot height it is thought that a 12 inch thickness is required for rigidity.

APPENDIX "C"

SWEDISH CIRCLE ANALYSIS

The bank, which may be considered typical for this study is shown on Plate XIX, page 129. For this typical bank the following information is obtained:

Slope = i = 25° $\qquad \phi = 0$

Unit weight = w = 110 lb. per cu. ft.

Height of bank = H = 36 ft.

Height of firm layer = DH = 50 ft.

Depth factor = D = 1.39

From Figure 16-27 of Taylor's "Fundamentals of Soil Mechanics" for the above conditions, the stability number is found to be 0.154.

Stability number = $N = c_d \div wH$

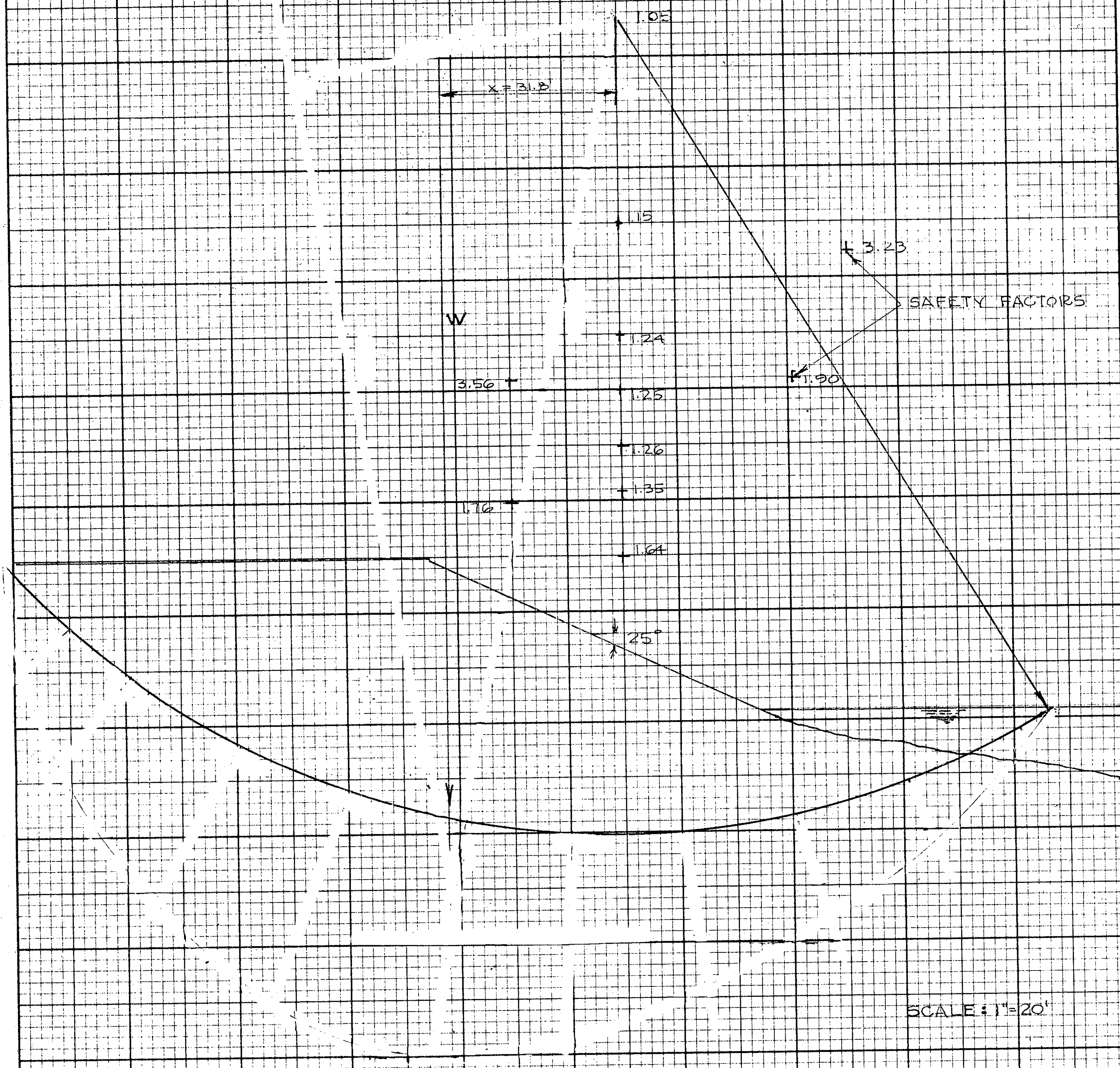
Therefore $c_d = N\,wH$

$= .154 \times 110 \times 36$

$= 611$ psf.

On Plate XIX, page 129, are shown a number of safety factors. Eleven trials in all were made resulting in these safety factors.

The circle which will be considered in this study is the one shown, having the lowest factor of safety. By scaling, this circle has a radius of 146 feet and an arc length of 186 feet, which excludes the portion of arc projected through the water.



SWEDISH CIRCLE METHOD

PLATE XIX

For location of centre of gravity a table was set up:

Take moments about A.

Point	Moment Arm FT.	Mid-ordinate FT.	Moment FT^2
1	10	10	100
2	20	20	400
3	30	26	780
4	40	32	1280
5	50	37	1850
6	60	42	2520
7	70	44	3080
8	80	46	2880
9	90	42	3780
10	100	39	3900
11	110	34	3740
12	120	30	3600
13	130	24	3120
14	140	19	2660
15	150	17	2550
16	160	13	2080
17	170	-9	-1530
18	180	-4	- 720
		462	36070

Units

Distance from A to centre of gravity $= \frac{36070}{462} = 78.2$ ft.

Total weight $= W = W_s \quad _s + 62.4 \times 13 \times 10$

$= 495{,}000 + 3{,}000$

$= 503{,}000$ lb.

Distance from centre of gravity of centre of circle $= x = 31.8$ feet.

Disturbing moment $= Wx$

$= 503{,}000 \times 31.8$

$= 16{,}000{,}000$ ft-lbs.

Resisting moment = radius x arc length x cohesion
+ wt. of water (slice 18) x moment arm
= 146 x 184 x 600 + 13 x 10 x 62.4 x 65
= 16,300,000 + 530,000
= 16,830,000 ft.-lbs.

Safety factor = $\frac{16830}{16000}$ = 1.05

At impending failure, the safety factor should be 1.0. The above figure is assumed to be relatively accurate in this respect.

To increase the safety factor, rock fill is added at the toe as shown in Plate XV-A. The weight of fill, therefor, is:

Submerged fill: 1/2 x 45 x 14 x 90 = 28,400 lbs.

Other: 46 x 5 x 150 = 34,500 lb.

Added resisting moment = 28,400 x 53 = 1,500, 00 ft. lb.

34,500 x 43 = 1,480,000 ft. lb.

2,980,000 ft. - lb.

Total resisting moment = 16,830,000 + 2,980,000 = 19,810,000 ft-lb.

New safety factor = 1.20

This factor, even for soil, is considered too low. The minimum safety factor should be 1.25 or higher. Plate XV-D shows heavier rock fill than above resulting in a factor of 1.26.

APPENDIX "D"

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HARVARD UNIVERSITY
Division of Engineering and Applied Physics

Arthur Casagrande
Gordon McKay Professor of Soil Mechanics
and Foundation Engineering

Pierce Hall
Cambridge 38, Massachusetts
June 28, 1961

Mr. W. D Hurst
City Engineer
223 James Avenue
WINNIPEG 2, Manitoba

Dear Mr. Hurst:

The purpose of this letter is to try to answer the questions which you prepared for me on June 16, 1961. For convenience, I quote also each of your questions.

> Question No. 1: "What factors natural or artificial in your opinion cause the typical river bank failures observed in this area? (Drawdown, surcharge, ponding, frost action, erosion, presence of large trees, etc etc)"

Analysis of the river bank failures in the Winnipeg area is a difficult problem involving many variables. To do it justice would require the writing of a sizeable doctor's thesis.

All major bank failures in the Winnipeg area seem to have in common that they occurred during or after subsidence of the river after a severe flood condition. This indicates that the clay experienced temporary swelling and loss of strength along interfaces with silty layers or partings, and particularly along the contact with the more pervious underlying stratum. During falling river stages, the drop in piezometric pressures and the reconsolidation of the clay lags behind.

When a previously undisturbed bank is subjected to a major slide, this bank will be permanently weakened because drainage parallel to stratification is blocked. Such slopes then become vulnerable to further sliding under the effect of relatively minor causes such as application of a small surcharge fill, a minor drawdown condition after a period of higher river stages, minor river erosion at the toe of the bank, effects of seepage from a leaking water pipe or a sewer or even from liberal watering of lawns, or from ponding due to inadequate surface drainage conditions.

> Question No. 2: "What measures may usefully be undertaken to prevent river bank failures? (Removal of surcharge, sloping, toe loading, driving of piles, planting of vegetation, etc etc)"

I assume that this question refers to slopes which have not failed previously. The most important measure for such a slope is protection of the toe against river erosion. This could be accomplished by placing sufficient riprap or by constructing a bulkhead. In addition, it may be necessary to flatten the average slope in case it is steeper than slopes which empirically were found to be stable in the Winnipeg area. From slope studies now in progress, it is tentatively concluded that slopes with a height of 30 to 40 ft. and which have not failed before, should be not steeper than 1 on 6 to assure safety under conditions such as existed during the 1950 flood.

> Question No. 3: "What remedial measures in your opinion are suggested to prevent further failure of the banks where failure has already commenced? (Rock placed at toe of slope, grouting, trimming, removal of trees, driving of sheet and timber piling, erection of retaining walls, etc etc)"

Also for banks which have experienced sliding, protection of the toe against river erosion is the most important protective measure. In addition, flattening of the overall slope is a desirable precaution. This can be accomplished by construction of a horizontal berm which is combined with elimination of any depressions that cause ponding of water. Any fill which has been added on such a slope is an open invitation for additional sliding and should be removed. I wish to emphasize that even after such precautionary measures are carried out, one cannot be certain whether such a slope would remain stable when subjected to a repetition of flood and drawdown conditions similar to those of 1950. Therefore, construction of the proposed floodway should be mentioned as necessary for the purpose of protecting river banks in Winnipeg. Until the time when that protection becomes effective, one must assume that any slope which has experienced sliding will be subject to additional serious movements unless its average slope is substantially flattened and its toe protected against erosion. Based on empirical data, it is estimated that in order to achieve a slope which may be considered to be entirely safe for a 1950 flood condition, it should be not steeper than 1 on 6 if it has not experienced sliding, and 1 on 9 if it has been subjected to substantial sliding.

> Question No. 4: "What dimensional limitations should be placed on the construction of buildings on or near the banks of the rivers in this area? (For example, the permission of the River and Streams Protection Authority is required when any additional load is to be imposed on the bank within 150 feet in a horizontal distance from summer elevation of the river. Should for example a distance of 150 feet back from the top of the bank be required in place of the present regulation, or should this limitation be contingent on the height of the bank? Average prairie elevation in this area is about 32 - 35 feet above datum. The river level is maintained at about 6.5 feet above datum when the Lockport Dam gates are in place during the summer season. The Elevation of the bed of the river averages about -7.0 feet below datum.)"

Comparing the slopes of river banks adjacent to a number of apartment buildings which I inspected, with a statistical plot of height versus slope for river banks in the Winnipeg area, I have arrived at the opinion that a repetition of the 1950 flood condition would cause failures that might encroach dangerously upon these buildings.

For most slopes which I have inspected, I would consider the restriction of "150 ft in a horizontal distance from summer elevation of the river" as grossly inadequate. Until such time when the floodway will protect the City, safe distances for buildings supported by piles or caissons on the hard stratum underlying the clay, would be the tops of the 1 on 6 and 1 on 9 gradients as given in my answer to the preceding question. In addition, protection of the toe against erosion should always be made an essential.

My answers to your questions are subject to modification on the basis of the detailed investigations of the stability of these river banks which the Manitoba Water Control and Conservation Branch is undertaking in cooperation with P.F.R.A.

Sincerely yours,

AC:mma

(Sgd.) A. Casagrande

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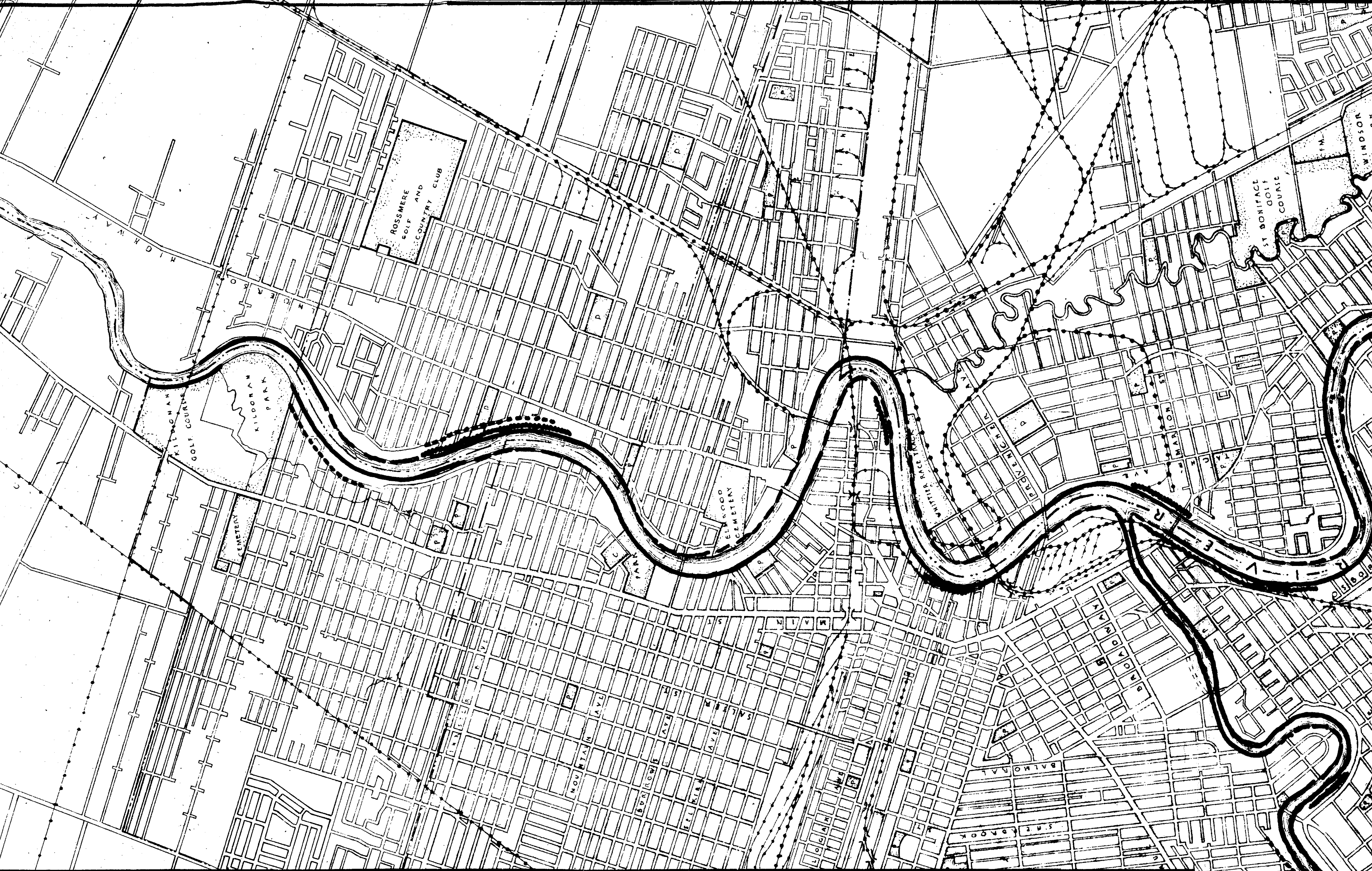

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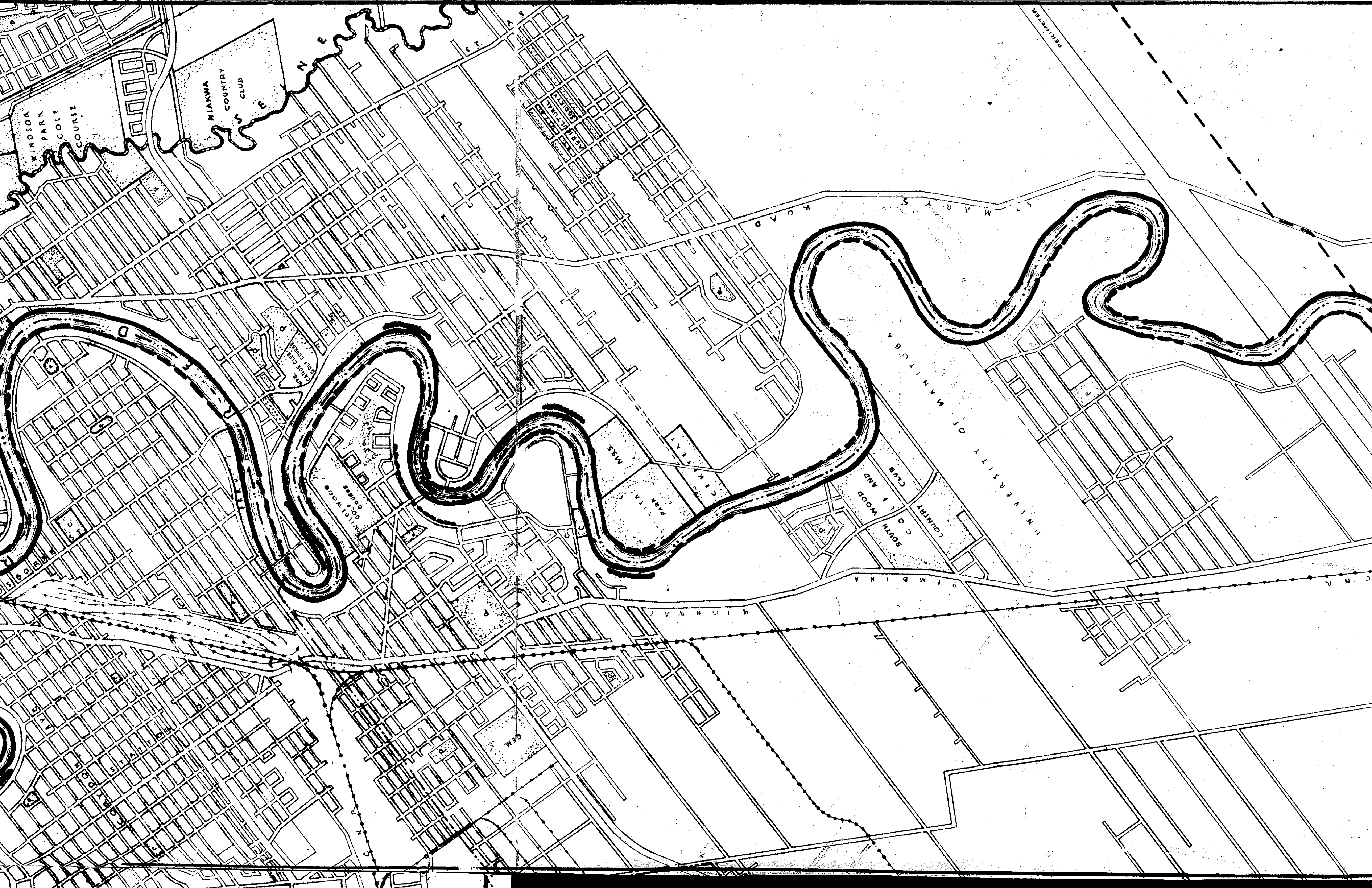
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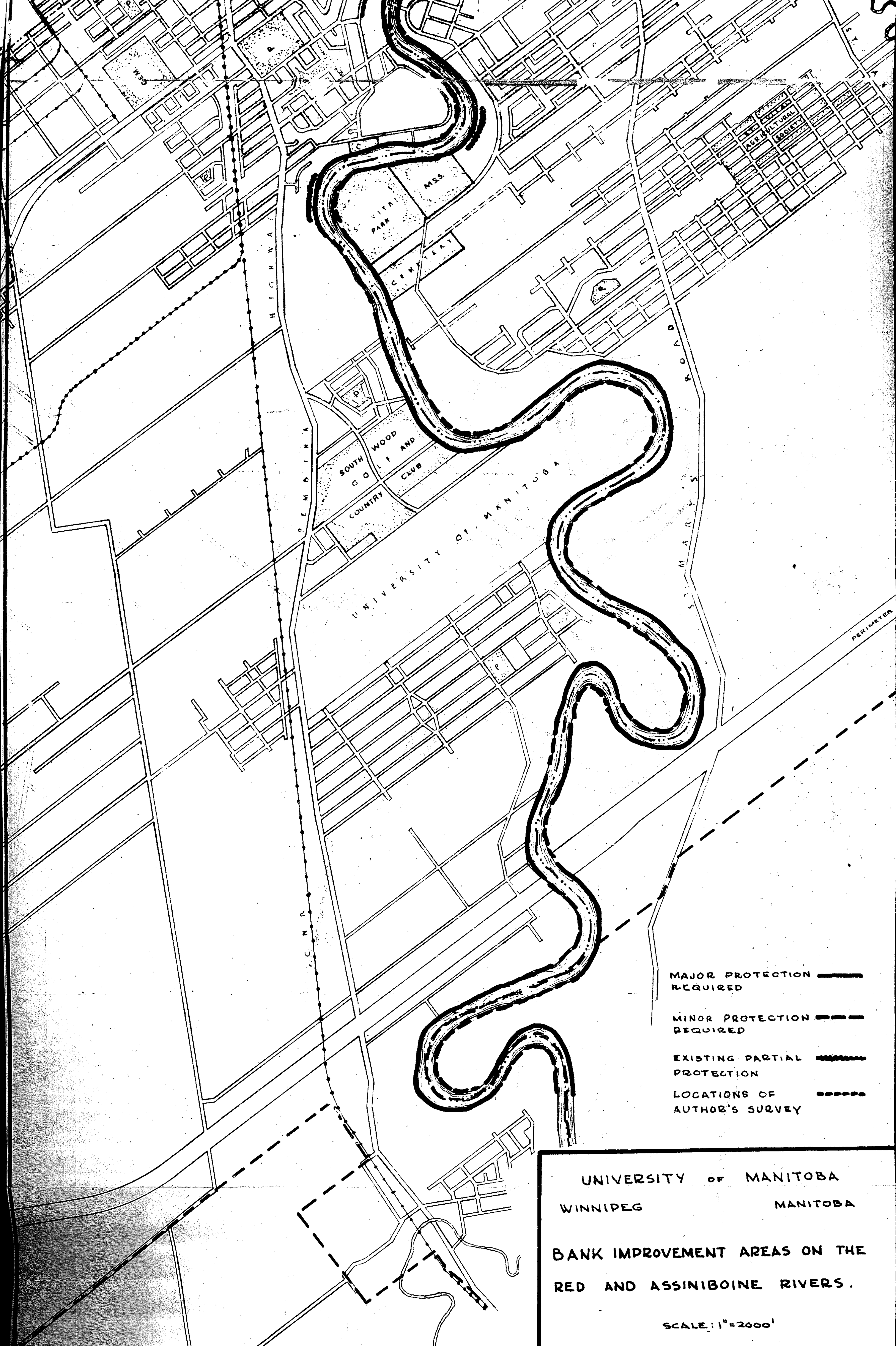
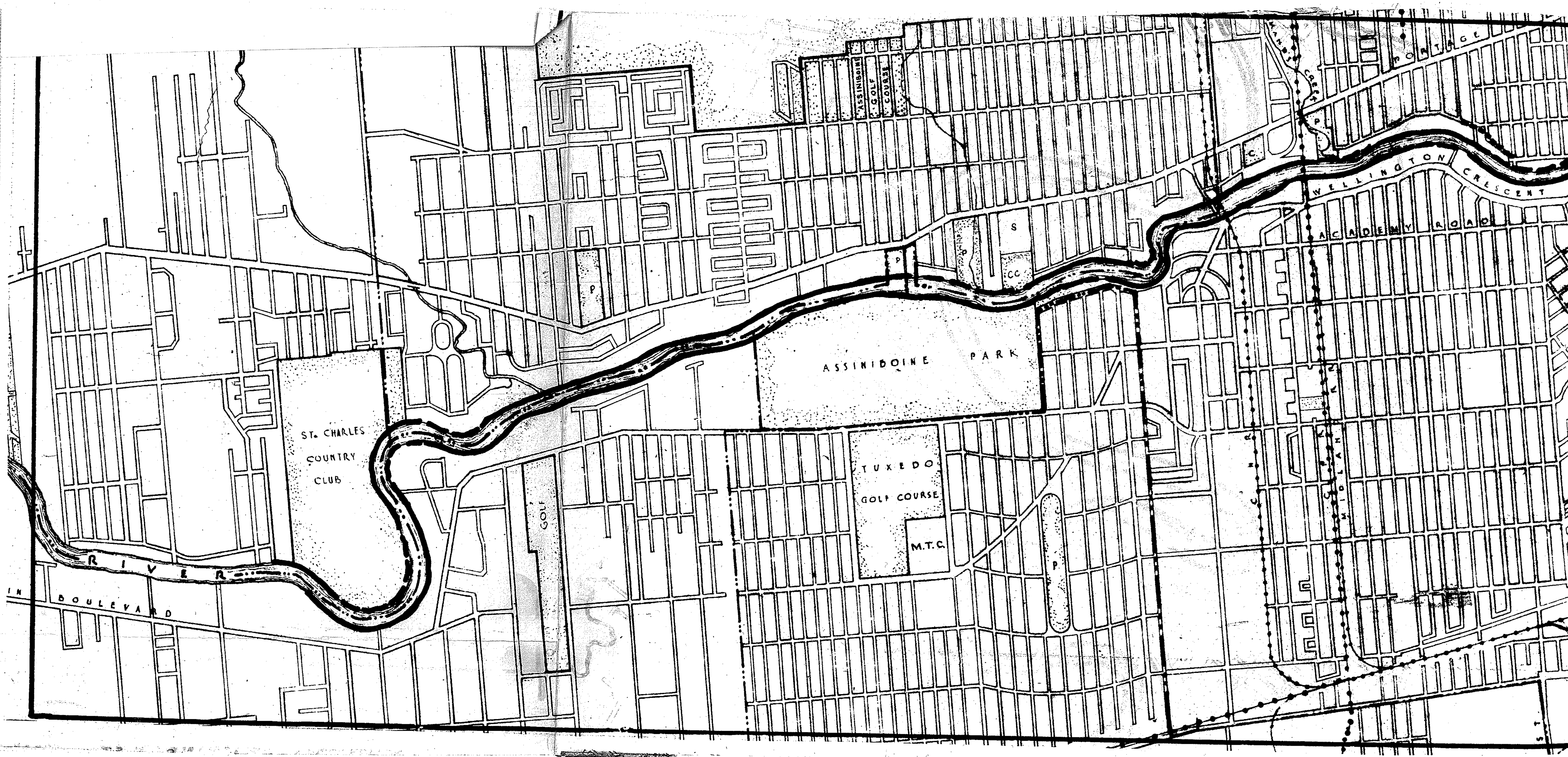


BASE PLATE: COURTESY METROPOLITAN CORPORATION

PLATE I



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