THE UNIVERSITY OF MANITOBA

REGIME CONSEQUENCES OF THE ASSINIBOINE RIVER DIVERSION

by

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ABSTRACT

Key words - (Assiniboine River, diversion, regime, degradation, quantitative fluvial morphology).

Using Blench's empirically deduced modified regime theory equations, an attempt is made to quantify predicted changes in the reach of the Assiniboine River downstream of the diversion at Portage la Prairie. These empirical equations have been used because the conditions of the channels that provided data for the regime equations are similar to the conditions found in the Assiniboine River channel downstream of the Portage Diversion and because Blench's system appeared to provide the widest choice of relationships to be utilized in an attempted quantitative prediction of this kind. It is estimated the ultimate depth of degradation immediately downstream of the diversion site will be about three feet in a period of about 30 years and the downstream extent of degradation will be approximately 20 miles. Also, immediately downstream of the diversion site, the mean channel width is anticipated to increase up to 20%, the mean depth is expected to decrease about 5%, and the mean velocity is expected to decrease about 15%. However, the quantitative estimates of increased channel width, decreased depth, and decreased velocity are subject to doubt due to their computation being extremely sensitive to error propagation.

It should be noted that these results are theoretical maximums because in the analysis, no account has been taken of the sediment supplied to the downstream reach by the 13-foot conduit under the dam.

Unfortunately, no information regarding the effect of this conduit on sediment supply was available at the time of the study.

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PREFACE

Regime. The noun "regime" applied to a channel or channel reach, is analogous to "climate" since it implies a behaviour that is appreciated in terms of many fluctuating factors whose average values, over a sufficient period, are either steady or change relatively slowly. Such a slow change is called "secular" (Latin, saeculum, age, span of time). The mind finds no difficulty in visualizing a climate or a regime as a relatively steady state of large erratic fluctuations, though statistical technicalities are involved in defining, exactly, "a sufficient period" and "a secularly changing mean". THE WORLD BOOK DICTIONARY (1963) defines "climate" (Greek, Klinein, to incline) as "the kind of weather over a period of years, based on conditions of heat and cold, moisture and dryness, clearness and cloudiness, wind and calm....". So "regime" may be defined as "the behaviour of a channel, over a period, based on conditions of water and sediment discharge, breadth, depth, slope, meander form and progress, bar movement, etc..". Unconventionally, but descriptively, it could be called "the climate of a channel".

T. Blench, 1969.

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SYMBOLS

Unless otherwise specified in the text:

A	is cross-sectional area (ft.^{2})
C	is sediment charge (parts per 100,000)
F_b	is the bed-factor
F_{bo}	is the zero bed-factor
F_s	is the side factor
K	is a coefficient
L	is length (ft., miles)
Q	is discharge (cfs)
S,Sc, etc.	is slope (ft./ft)
T	is mean annual bed-load transport (tons/day, acre-ft./year)
V	is mean velocity or volume of sediment
b	is channel width (ft.)
d	is mean depth (ft.)
d_{50}	is the sediment size (diameter in mm.) at which 50% of the sample is smaller by weight
d_s	is depth of degradation (ft.)
f(c)	is the bed-material load transport function
g	is acceleration due to gravity (32.2 ft./sec.^{2})
k	is a meander correction coefficient
n	is Manning's n
q	is discharge per foot width (cfs/ft.)
σ	is standard deviation
ν	is kinematic viscosity

CHAPTER 1

INTRODUCTION

It is only in recent years that major engineering works have been constructed on the Assiniboine River. These works include dykes placed downstream of Portage la Prairie over the last 50 years, the Shellmouth Dam constructed in 1968, and the Portage Diversion built in 1969. The object of their construction is the alleviation of spring flooding along the Assiniboine Valley and at Winnipeg. As a side effect the diversion and storage works cause changes in the river's natural regime; flows are now regulated and sediment travel in the river channel is interrupted by the reservoirs and structures. In addition, the dykes downstream of Portage la Prairie have altered the sediment transport capability of that reach of the river.

A mean or average elevation of the bed of a river channel is maintained over a period of time because the inflow of sediment to the reach equals the outflow. The purpose of this report is to present a description of the Assiniboine River immediately downstream of the Portage Diversion, to comment on its present state, and assess quantitatively the expected changes due to the disruption of sediment flow.

Perhaps the best summary description of the Portage Diversion is to be found in a pamphlet published by the Prairie Farm Rehabilitation Administration, Government of Canada, and the Water Resources Branch, Province of Manitoba, during construction of the diversion at

Portage la Prairie and the Shellmouth Dam near Russell. This description is reproduced below:

> "PORTAGE DIVERSION
>
> A reservoir covering 1,610 acres with a storage capacity of 14,600 acre-feet is being created by the construction of an earthfill dam 1,400 feet long, and rising 35 feet above the bed of the Assiniboine River. A concrete spillway control structure equipped with two Bascule fish-belly type gates, 13 feet by 75 feet in size, is located on the south side of the dam. These gates are the largest of their type in North America.
>
> Also located in this structure is a low-level gate-controlled riparian outlet conduit. North and west of the dam at the upper end of the diversion channel, an inlet control structure regulated the flow to Lake Manitoba. This is accomplished by the use of four vertical lift gates each measuring 14½ feet by 40 feet. The diversion channel is designed to carry a flow of up to 25,000 cubic feet per second.
>
> As the Portage Diversion is an unlined channel, it was necessary to construct it in such a way as to keep water velocities below those which would cause erosion. When the diversion reservoir is full, there is a drop of 50 feet between the diversion channel inlet and the level of Lake Manitoba, over a relatively short distance of 18 miles. To keep velocities down to about 3½ feet per second, three drop structures along the diversion route are incorporated into the design. The channel has widths varying from 175 feet to 1,200 feet, with an average width of 600 feet. The greater width is through the Delta Marsh area adjacent to the lake. Dykes have been built along the entire length of the channel using much of the 10,000,000 cubic yards of excavated material. The design depth of water above the surrounding terrain will generally be equal to the ground water level when the channel is operating at peak discharge."

FIGURE 3 presents a site plan of the intake works.

When construction of a combined dam and diversion structure for flood-control purposes such as at Portage la Prairie is undertaken, two changes are immediately imposed upon the downstream reach. These imposed changes are as follows:

1. Water flow
    a. peak flows are reduced.
    b. mean flow is reduced due to diversion and reservoir losses.
    c. low flows may be increased somewhat depending upon storage characteristics and operation of the reservoir.
2. Sediment flow
    a. structure blocks bed-load transport in the natural channel.
    b. reduced water velocities in the reservoir allows suspended particles to settle. The inlet works were so designed that the diverted flow will be as sediment-free as possible to prevent deposition in the diversion channel.

Due to the nature of the Portage Diversion system, the imposed blockage of sediment flow is temporary; that is, when the reservoir completely fills with sediment, the bed-material load will be passed over the diversion spillway and sediment supply to the downstream reach will be restored as well as the original downstream slope. FIGURE 2 illustrates the above-mentioned changes.

It is very important to note that some sediment supply will be available to the downstream reach through the 13 by 13-foot low level outlet conduit under the dam. Due to a lack of data, no account of this supply is made in the following analysis.

As a consequence of the above imposed changes, other parameters of the river's regime are subject to change. These parameters and the expected consequent changes (Galay, 1966) are:

1. The most significant change in the downstream reach is its expected degradation and flattening of the river's profile when the sediment flow is interrupted.

2. The mean channel width for a specified discharge is expected to be reduced. Channel width varies directly with the square root of discharge ($b \propto \sqrt{Q}$); with a decrease in mean discharge due to the operation of the diversion the mean channel width will decrease.

3. The water surface elevation for a specified discharge is expected to be lowered due to degradation of the channel bed.

4. The mean depth for a specified discharge would probably be increased, however, this is not certain and requires verification.

5. Mean velocity at a specified discharge is expected to be reduced with a possible increase in cross-sectional area.

6. Hydraulic roughness is expected to increase with the "sorting" effect of degradation. Unfortunately, no information regarding bed configuration was collected in the field studies.

7. The consequences of diversion upon channel pattern is unknown.

8. The rate of lateral shift of the channel is expected to be reduced with the reduction in mean discharge.

In the following chapters the imposed changes are described more fully and consequent changes investigated with a view of assessing them quantitatively.

For the analysis used in this particular thesis, the "modified regime theory" equations advanced by Blench (1966, 1969) are utilized. It should be recognized that there are limitations to the use of the empirically deduced equations of regime theory; but, for the sand-bed channel of the Assiniboine River downstream of the Portage Diversion, the conditions of this natural channel and the natural and artificial channels used to determine the regime theory relationships are considered sufficiently similar to warrant use of these relationships in attempting quantitative predictions.

CHAPTER 2

THE DOWNSTREAM REACH

2.1 Discharge

There are four hydrometric gauges operated by the Water Survey of Canada in the vicinity of the Portage Diversion. FIGURE 6 shows, in schematic, the relationship of the gauges to the diversion system. The gauges are:

1. Assiniboine River at Portage la Prairie

Drainage Area:	approximately 62,140 square miles
Records Begin:	1923 (not continuous)
Mean Discharge:	(16 years) 1,800 cfs
Extremes Recorded:	Maximum daily discharge 32,000 cfs on April 21, 1974. Minimum daily discharge 25 cfs, January 21, 1963)

(Discharge partially regulated by reservoirs on tributaries)

2. Assiniboine River Diversion near Portage la Prairie

Records Begin:	April 19, 1970

3. Assiniboine River near Rossendale

Drainage Area:	approximately 62,100 square miles
Records Begin:	1970

4. Assiniboine River near Holland

Drainage Area:	approximately 61,980 square miles
Records Begin:	1954

The pre-diversion, or natural flow conditions, as well as estimated post-diversion conditions are summarized in TABLE 1. At this point in time, the estimates frequency curve for post-diversion flows (FIGURE 7) can only be considered as a best guess estimate based on available information.

The mean monthly flow-duration estimates (FIGURE 9), however, may be considered reasonably reliable. These were prepared by the Water Resources Branch assuming the following defined flow conditions:

1. Past - This shows the flow pattern that existed for the period of record 1921 to 1964 as recorded by the Water Survey of Canada. Missing records have been reconstructed by the Prairie Provinces' Water Board. The Water Resources Branch has adjusted these flows eliminating the regulatory effects of the Rivers Reservoir.

2. Present - This condition represents the operation of Shellmouth and Rivers reservoirs for both flood control and water supply. The flood-control aspect involves drawing down Shellmouth Reservoir from a normal live storage of 290,000 acre-feet in November, to 150,000 acre-feet by the end of March. The water-supply aspect involves guaranteeing a minimum flow of 300 cfs at Brandon and 50 cfs at Portage la Prairie. These are arbitrary minimum flows being well in excess of existing demands at these locations. In addition, if required, water is assumed to be diverted via the Portage Diversion to the Delta Marsh and to Lake Manitoba provided that the flow at Portage la Prairie is in excess of 800 cfs.

3. Future - This conditions represents the demands as envisioned in 10 to 20 years with flood control and conservation storage in the Assiniboine Basin as for item 2 above. However, the minimum flows at

Brandon and Portage la Prairie have been decreased to 207 cfs and 31 cfs respectively. These flows are based on providing the future thermal and municipal dilution requirements at Brandon and Portage la Prairie. In addition, minimum flows of 50 cfs and 880 cfs have been guaranteed for the Assiniboine River at Headingly and the Red River at Winnipeg respectively. Also, it has been assumed that 10,000 acres would be irrigated from the Assiniboine River in the Assiniboine River Valley flats above Portage la Prairie, 10,000 acres in the Portage la Prairie area, and 10,000 acres in the Morden-Winkler area. To satisfy these increased demands, it has been assumed that a diversion from the South Saskatchewan River at Lake Diefenbaker via the Qu'Appelle River System, having a capacity of 225 cfs, would be available to meet Manitoba's needs.

The bankfull discharge determined at different sections of the reach under consideration varies between 12,000 and 13,000 cfs. At this discharge, the river overtops the natural levees formed along the banks and spills over to flood the lower prairie level. Any flows exceeding this overtop the river banks and are lost for channel-forming purposes. Therefore, the bankfull discharge would be considered to be the "dominant discharge", that is, the constant discharge equivalent, for channel-forming purposes, of the variable river flow.[1,2] The construction of dykes along the reach complicates this assumption.

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[1] Henderson, F.M., 1966, p.464

[2] Blench, T., 1966, p.54.."It is to be noted there is no obvious reason to expect an equivalent uniform discharge calculated from one phenomenon - for example, meander formation - to be exactly the same as from another such as self-adjustment of slope."

Considering the dykes as banks of the river, albeit artificial, the new or artificial bankfull discharge is of the order of 22,500 cfs. This discharge event is considered too infrequent (9% natural conditions, 0.13% post-diversion conditions) for use as a dominant discharge.

A dominant discharge was calculated using the "sediment moment approach" outlined by Komura (1968). This method employs both the duration relationship of discharge and the duration relationship of sediment transported in calculating a dominant discharge. Data from the duration curve of mean monthly flows, FIGURE 9, and the sediment transport rating curve, FIGURE 13, was used in the computation. The calculated dominant discharges were for natural conditions and for future conditions, 12,000 cfs and 11,800 cfs respectively.

TABLE 1

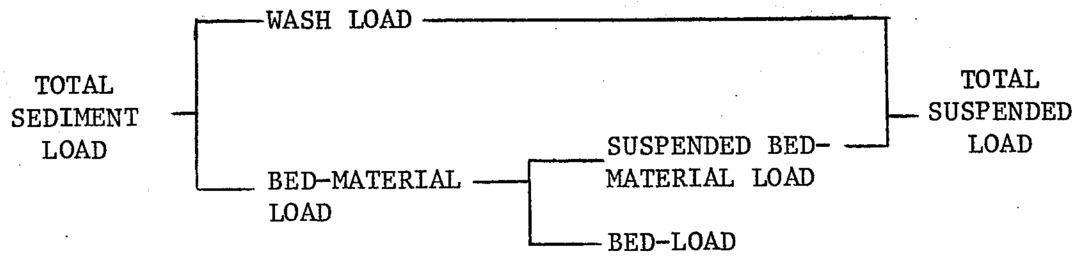
Discharge Summary
Assiniboine River at Portage la Prairie
(Period of Record 1921-1964)

	Natural Conditions		Post-Diversion Conditions	
	Discharge (cfs)	Probability of Exceedence in any year %	Discharge (cfs)	Probability of Exceedence in any year %
mean flow	1,650		1,380	
flow at which mean annual sediment transport occurs	2,100		1,800	
mean annual flood	11,000	40	8,000	40
natural bank-full	12,500	32	12,500	0.25
artificial bankfull	22,500	9	22,500	0.13
dominant discharge (computed)	12,100	34	11,800	0.28
1:10	21,500	10	8,800	10
1:100	42,500	1	9,600	1

2.2 Sediment Transport

As can be seen from TABLE 2, Summary of 1971 Bed-Material Sampling, and FIGURE 11, Wash Bore Logs, the bed and bank material of the channel is composed mainly of sands in the fine to coarse ranges. It appears from the wash bore data the sand bed-material is a thin veneer overlaying a stiff dark grey clay.

Measurements of the suspended sediment load at Portage la Prairie for the period 1956 to 1968 were used to determine the Total Sediment Transport Rating Curve of FIGURE 13. The total sediment load in a river channel consists of a number of components transported in differing modes; these components are illustrated below:



The relationship between total suspended load and discharge was increased by 15% to account for the small amount of sediment that cannot be caught by the suspended material sampler (10%) and the bed-load (assumed to be 5%).[1] The Total Sediment Transport Rating Curve was then used with the flow duration curves of FIGURE 9 to construct the

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[1] Kuiper and Galay, 1971, p.48 ...... "An investigation of various sediment stations throughout the (Saskatchewan-Nelson) basin indicated that the measured load varied from 4% to 15% and 10% was chosen as being representative. Computations of bed-load using the Einstein bed-load function and the Meyer-Peter and Muller equation indicated that the bed-load varied from 3% to 6% of the measured suspended load and a value of 5% was adopted."

Sediment Transport Duration Curves for the Assiniboine River at Portage la Prairie, FIGURE 14.

The total mean annual sediment transported at Portage la Prairie was calculated as being, for natural conditions, 485 acre-feet per year; for present conditions, 409 acre-feet per year; and for future conditions, 372 acre-feet per year.

Prior to construction of the Portage Diversion, this sediment passed through the reach. Now, with the flood-control works in place and operational, this sediment is trapped by the reservoir. Near sediment-free water is released to the diversion channel during high flow periods and to the natural channel year-round. This condition will continue until the reservoir is completely filled. As mentioned previously, the Portage Diversion has a storage capacity of approximately 14,600 acre-feet. From the sediment transport duration curve (natural conditions - upstream of Portage la Prairie) on FIGURE 14, the mean annual amount of material to be deposited in the reservoir would be in the order of 485 acre-feet per year. Using these data, the reservoir is estimated to be filled in 30 years. During this period of reservoir filling, the near sediment-free water released to the natural channel will "pick up" sediment approximating the load trapped by the dam and cause a flattening of the downstream profile by a degrading of that reach. This loss of material would presently appear to average about 110 tons per day, or about 20.5 acre-feet per year, using 5% of the present mean annual sediment transport. Over 30 years, this totals a loss of about 615 acre-feet of bed-material in the reach downstream of the dam. The length of reach from which this sediment can be expected to be removed and the estimated depth of degradation will be discussed in the following section.

When the reservoir filling is completed, and the sediment flow is restored to the downstream reach, the process of degradation will cease. As the diversion continues to operate, deposition of sediment will occur both upstream and downstream of the diversion point. The amount of deposition will depend upon the quantities of near sediment-free water diverted from the natural river channel to the diversion channel during the flood conditions.

Using the natural mean annual bed-load transport $T_n = 24.25$ acre-feet per year, and the present mean annual bed-load transport, $T_p = 20.5$ acre-feet per year, and subtracting indicates about 3.75 acre-feet per year of bed material could be expected to be deposited in the vicinity of the diversion after the completion of reservoir filling. That is, assuming the downstream reach is competent enough to transport the present mean annual bed-load with its degraded profile. It is more likely that greater amounts of material would be deposited, especially downstream of the diversion, immediately after the initial reservoir filling and degradation phase.

As sediment is not conveniently measured as a volume, the proportionate discharge of sediment in the water-sediment complex is expressed for use in the regime equations as a ratio of weight discharges. The "charge" of any portion of the sediment load (e.g., bed-load charge) is defined as the weight (in air) of that portion of the sediment flow per second, divided by the weight of the water flow per second.

The bed-load charge (C) at Q = 2,100 cfs, the discharge at which mean annual sediment transport occurs under natural conditions, was calculated as being equal to 2.3 parts per 100,000 by weight. Using Blench's relationships (1966, FIGURE 7.2) the bed-material load transport functions $f^{1}(c)$, $f^{11}(c)$ and $f^{111}(c)$ equal 1.26, 1.29 and 1.32 respectively for C = 2.3 parts per 100,000. With construction of the diversion structure, the bed-load charge (C) will tend to zero. As C tends to zero, f(c) will tend to 1.0, and the zero bed-factor (F_{bo}) will equal to the value for the bed-factor (F_b), (see EQUATION A.9).

TABLE 2

Summary of 1971 Bed-material Sampling
Assiniboine River at Portage la Prairie

	Size - % Finer by Weight			
	d_{16} mm.	d_{50} mm.	d_{84} mm.	d_{90} mm.
1971 Data				
at XS-1	.33	.42	.79	.97
mean of XS-1, 2 and 3	.4	4.9	4.9	6.2
mean of reach XS-1 to 15[1]	.34	.63	1.8	3.0

[1] Standard deviation (σ) equal to 0.66 mm.

2.3 Channel Profile

In attempting to predict an ultimate degraded profile and the downstream extent of degradation, the most important factors to consider are the type of material forming the bed of the channel and the depth of this material. The range of sizes of the bed-material dictates whether an ultimate degraded profile will be achieved by a reduction of the slope (and consequently a reduction in sediment transporting capability) or by forming an "armour" protection of the bed (Komura and Simons, 1967).

Investigation of the bed-material indicated an insufficient fraction of coarse particles for an armouring of the bed to occur; that is, it was found, using Shields' Diagram, virtually all the material forming the bed was capable of being transported by the expected discharges. In the vicinity of cross-section 1, however, referring to the wash-bore logs of FIGURE 11, one foot below the present thalweg elevation enough coarse material may be found to provide some protection against further degradation in the vicinity of that cross-section.

Following are two methods of calculating the degraded profile and extent of degradation below dams; they are the stable-slope approach, and the prototype approach.

The stable-slope approach: the stable-slope approach (USBR, 1963) of predicting the degraded profile assumes the character of the bed-material does not change as degradation progresses (i.e., the amount of coarse material in the bed is negligible) and that the depth of the bed-material is greater than the expected degradation.

Computation of a stable-slope using Blench's regime slope formula follows:

$$S_c = \frac{kF_{bo}^{11/12} f^{111}(c)}{Kb^{1/6}Q^{1/12}} \qquad (A.5)$$

Where S_c	is the computed stable slope
k	is the meander correction coefficient, approximately equal to 1.62 (found by solving EQUATION A.3 with $S = 1.9 \times 10^{-4}$, $F_{bo} = 0.785$, $F_s = 0.036$, $f^1(c) = 1.26$, $Q = 2{,}100$ cfs, and $K = 1{,}900$).
F_{bo}	is Blench's zero bed-factor, equal to 0.785 (see following section).
$F^{111}(c)$	is the bed-material load transport function, equal to 1.0 as bed-load becomes vanishingly small.
K	is a constant equal to $3.63/\nu \; 1/4$, where ν is kinematic viscosity and g is acceleration due to gravity. ($K \simeq 1{,}900$).
b	is the channel width, 240 feet (FIGURE 20).
Q	is the discharge (1,800 cfs) at which mean annual sediment transport is expected to occur under post-diversion conditions.

Solving the equation yields a stable-slope of 1.45×10^{-4} feet per foot. The average slope over the length of channel (FIGURE 19) downstream of the diversion is 1.9×10^{-4} feet per foot. This gives a difference (ΔS) of 0.45×10^{-4} feet per foot between the computed stable-slope and the existing slope.

It can be seen that the computation of a stable-slope using EQUATION A.5 is relatively insensitive to errors in the selection of width (b) and discharge (Q). In the equation, the sixth root of width and the twelfth root of discharge are used thereby virtually eliminating practical error on their account.

Combining the maximum length of degradation (100 miles to confluence with Red River), the mean channel width (b = 240 feet), and $\Delta S = 0.45 \times 10^{-4}$ feet per foot, with the following two equations (USBR, 1963) will yield the expected depth of degradation at the dam (d_s) and the expected volume of sediment (V) removed.

$$d_s^2 = \frac{64}{39} \frac{V\Delta S}{b} \qquad (2.1)$$

and

$$L = \frac{13}{8} \frac{d_s}{\Delta S} \qquad (2.2)$$

Solving EQUATION 2.2 yields an ultimate depth of degradation (d_s) of 15 feet. Solving EQUATION 2.1 then yields $V = 702.3 \times 10^6$ cubic feet = 16,120 acre-feet. At a mean rate of 20.5 acre-feet per year, it would take 786 years to achieve this degradation.

In 30 years, the loss would total 615 acre-feet of bed-material. Solving EQUATION 2.1 with V - 615 acre-feet yields $d_s = 2.9$ feet. The length of reach degraded (L) then equals 105,000 feet or 19.8 miles.

TABLE 3

Comparison of Degraded Profile Estimates

	Stable-Slope Approach Q=1,800 cfs	Prototype Approach Q=4,500 cfs	Prototype Approach Q=1,800 cfs
degradation at dam	2.9 feet	18 feet	9 feet
time of degradation	30 years	6.6 years	2.8 years
length of degraded reach	105,000 feet	48,000 feet	24,000 feet

<u>The prototype approach</u>: a prototype approach (Priest and Shindala, 1969) to degradation below dams based on laboratory data and field data from Fort Randall, Fort Peck and Hoover dams in the United States, is currently used by some agencies in that country. The dimensionless parameters representing the functions and their relationships are presented on FIGURES 15, 16, and 17.

Applying these relationships to the downstream reach yields a predicted degradation at the dam of about 18 feet with the required time being 6.6 years and the ultimate length of the degraded reach being 46,800 feet. The Assiniboine River discharge which would be the equivalent of those used in the construction of this relationship (mean annual mean monthly peak) was estimated as 4,500 cfs.

Applying these relationships using Q = 1,800 cfs, the flow at which mean annual sediment transport is expected to occur under post-diversion conditions, yields a predicted degradation at the dam of about nine feet with the required time being 2.8 years and the ultimate length of the degraded reach being 24,000 feet.

It can easily be seen that a predicted degradation of the extent calculated by the prototype approach, as it presently exists, would require a rate of removal of bed-material far in excess of the Assiniboine River's capabilities; therefore, the results of the prototype approach analysis must be rejected in this instance.

TABLE 3 provides a summary of the predictions provided by both of the foregoing approaches.

Cross-sections at 15 selected locations downstream of the dam have been surveyed by the Water Resources Branch, Province of Manitoba, as part of a program to monitor the expected degradation of the channel. The locations of these cross-sections are shown on FIGURE 4. At the time of writing, the results of surveying undertaken in March 1971, (discharge approximately 600 cfs) and May 1972, (discharge approximately 8,000 cfs) are available.

Also available are cross-sections surveyed approximately 25,000 feet downstream of the dam-site by the Prairie Farm Rehabilitation Administration, Government of Canada, in 1961, 1964, and 1971. This organization also undertook a comprehensive mapping (one inch equals 400 feet) of this reach of the river in 1951.

Comparison of the PFRA cross-sections indicates, for that period, the river channel was relatively stable, neither measurably aggrading or degrading. A comparison of the more recent Water Resources Branch data was also made. Caution should be exercised in the discussion of results of this comparison because of the great difference in discharges at the times of the surveys.

The comparison between the 1971 and 1972 data indicates a slight net aggradation at cross-section 1, but, the channel bottom was lowered in 1972 about one-half foot for a width of about 200 feet. At cross-section 2, a degradation averaging one foot over about 200 feet in width was observed with a slight aggradation extending up the banks. Cross-section 3 indicates a degradation of the channel averaging six feet over a 280-foot width with slight aggradation on the sides of the channel. Cross-section 4 showed a slight net aggradation over the year. At cross-section 5, degradation was observed to average about four feet over a 150-foot width with the thalweg itself being lowered in excess of seven feet. According to the wash-bore log information at cross-section 5 (FIGURE 11) the channel appears to have eroded over five feet through a stiff, highly plastic, dark grey clay. This development should be investigated further in the monitoring program. At cross-section 6 and beyond, a net aggradation is indicated at all cross-sections with the exception of cross-section 11. The degradation at cross-section 11 is discounted due to its proximity to a bridge site and the great difference in discharges at the times of the survey.

From early observed evidence, it would appear the process of degradation may have begun. The reach showed a lower bed for a distance of about 12,000 feet below the dam with a higher bed downstream on the observed reach. The ultimate depth and downstream extent of degradation calculated by the stable-slope approach ($d_s \simeq 3$ feet;

$L \simeq 20$ miles) appears, at this time, to be the best probable estimate of the extent of this phenomena for the 30-year period during which the reservoir is filling with sediment. Caution should be exercised in any assumptions, however, due to the wide variance in discharge at the times of the surveys. What may appear to be significant degradation may only be local scouring at high flow.

2.4 Channel Geometry

From the available cross-section and stage information for the downstream reach, a mean cross-section versus discharge relationship array was derived and is shown on FIGURE 20. These relationships are considered to describe the mean channel for natural conditions.

It was also considered that sufficient observations were made in the construction of these curves, and the channel sufficiently approximates a trapezoidal section, to determine the bed-factor (F_b) and side-factor (F_s) values of the natural channel using Blench's (1966, 1969) modified regime equations:

$$b = \sqrt{\frac{F_b \, Q}{F_s}} \qquad \text{(A.1)}$$

$$d = \sqrt[3]{\frac{F_s \, Q}{F_b^{2}}} \qquad \text{(A.2)}$$

Solving for F_s and F_b with Q = 2,100 cfs, the discharge at which mean annual sediment transport occurs under natural conditions, b = 225 feet, and d = 4.2 feet (FIGURE 20) yields the following:

for Natural Conditions $F_s = 0.036$

$F_b = 1.00$

The problem we must now address is the relationship of these descriptive regime parameters before and after construction of the Portage Diversion. In a sand bed river, such as the Assiniboine, with a small bed-load and with the virtual removal of that bed-load by river closure, the sediment charge (c) will tend to zero. Using Blench's definition of zero bed-factor (F_{bo}), we find F_b tends to F_{bo} as C tends to zero, for post-diversion conditions. For natural conditions, F_{bo} can be derived from EQUATION A.9, using $F_b = 1.0$ and $C = 2.3$ parts per 100,000.

$$F_b = F_{bo}\ (1 + 0.12\ C) \qquad (A.9)$$

Hence for natural conditions, $F_{bo} = 0.785$ and for post-diversion conditions, $F_b = F_{bo} = 0.785$. Using the regime slope formula:

$$S = \frac{F_{bo}^{5/6} F_s^{1/12} f^1(c)}{KQ^{1/6}} \qquad (A.3)$$

and designating the present natural slope of the channel $S_n = 1.9 \times 10^{4}$ and the computed post-diversion stable slope $S_p = 1.45 \times 10^{-4}$, then

$$\Delta S = S_n - S_p = 0.45 \times 10^{-4} \qquad (2.3)$$

and

$$\Delta S \doteq \frac{kF_{bo}^{5/6}}{K} \left[\frac{F_{sn}^{1/12} f^1(c)n}{Q_n^{1/6}} - \frac{F_{sp}^{1/12} f^1(c)p}{Q_p^{1/6}} \right] = 0.45 \times 10^{-4} \qquad (2.4)$$

Substituting $K = 1{,}900$, $k = 1.62$, $F_{bo} = 0.785$, $Q_n = 2{,}100$ cfs, $Q_p = 1{,}800$ cfs, $f^1(c)_n = 1.25$, $F_{sn} = 0.035$, and $f^1(c)_p = 1.0$ (as C tends to zero) into EQUATION 2.4 yields a post diversion side-factor (F_{sp}) value of 0.017.

It should be noted that in the solution of EQUATION 2.4, the value of the twelfth root of the post-diversion side-factor (F_{sp}) is found; then, that value is raised to the twelfth power. This procedure also raised possible error in the result to the twelfth power. Even though in subsequent calculations the post-diversion side-factor value square and cube root is used, substantial error may still exist in the value. This would appear to be the major obstacle in obtaining reliable quantitative estimates of changes in channel width, flow depth, and velocity using the modified regime theory equations.

In summary, for post-diversion conditions $F_s = 0.017$ and $F_b = F_{bo} = 0.785$.

Substituting these post-diversion condition values and Q = 1,800 cfs into Equations A.1 and A.2 gives a channel width (b) of 288 feet and a mean depth (d) of 3.7 feet. Solving EQUATION A.8 with the above values for F_b, F_s, and Q gives a mean velocity (V) of 1.7 feet per second.

Summarizing the foregoing as a consequence of the diversion, it is estimated using the modified regime theory equations:

a. mean channel width (using b = 240 feet) will increase approximately 20%

b. mean depth (using d = 3.9 feet) will decrease approximately 5%

c. mean velocity (using V = 2 feet per second) will decrease approximately 15%. This implies cross-sectional area (A) will increase by 15% for a specified discharge (equation of continuity, Q = VA).

In recent years, a number of studies concerning the meandering of channels have been conducted (Ackers and Charlton, 1970; Leopold and Wolman, 1957). However, it was found (Zimmerman, 1971) the relationships derived from these observations did not yield satisfactory results when applied to the Assiniboine River.

There appears to be little hope at the present time of quantifying the change in meander wave length and meander shift rate. However, if it is accepted these two parameters vary directly with discharge, then the imposed reduction in discharge would effect a consequent reduction in the meander wavelength and meander shift rate.

CHAPTER 3

SUMMARY AND CONCLUSIONS

As a consequence of the Assiniboine River Diversion at Portage la Prairie, the reach downstream is estimated to degrade up to three feet (near the diversion site) over a period of about 30 years. The downstream extent of the degradation is estimated to be approximately 20 miles. The degradation in the vicinity of cross-section 1 may be less than this estimate due to a coarse sand and gravel layer two feet below the 1972 observed channel bottom. That is, there may be a "local armouring" of the bed at that cross-section.

The method utilized in this thesis to provide quantitative estimates of the increased channel width, decreased depth, and decreased velocity is extremely sensitive to error propagation and the results therefore are suspect. However, these quantitative estimates can indicate the order of magnitude of the expected regime consequences.

The mean channel width for a specified discharge is estimated to have a tendency to increase up to approximately 20% immediately downstream of the diversion site with this consequence being less pronounced further downstream.

The mean depth for a specified discharge is estimated to have a tendency to decrease approximately 5% immediately downstream of the diversion site and the mean velocity for a specified discharge is estimated to have a tendency to decrease approximately 15%. These consequences, as well, will be less pronounced further downstream.

The above consequences imply a lowered water surface elevation for a specified discharge on the degraded reach due to the degradation of the channel bed and an anticipated net increase (slight) in the hydraulic roughness of the reach due to the "sorting" effect of degradation.

With an imposed reduction in discharge, the meander wavelength and meander shift rate are anticipated to be reduced.

Due to the consequent degradation of the bed and the tendency to increase channel width, the stability of the existing banks is expected to be reduced and local scour at bridge sites is expected to be increased. Subsequent monitoring should be directed to the investigation of the consequences at specific locations and measures should be suggested to enhance the integrity of endangered structures.

It should be noted that these results are theoretical maximums because in the analysis no account has been taken of the sediment supply provided to the downstream reach by the 13- by 13-foot conduit under the dam. Unfortunately, no information regarding the effect of this conduit on sediment supply was available at the time of the study.

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APPENDIX A

MODIFIED REGIME THEORY EQUATIONS

For the analysis used in this particular thesis, the empirically deduced "modified regime theory" equations advanced by Blench (1966, 1969) are utilized. The equations derived by Blench for design and analysis are:

$$b = \sqrt{\frac{F_b Q}{F_s}} \qquad (A.1)$$

$$d = \sqrt[3]{\frac{F_s Q}{F_b{}^2}} \qquad (A.2)$$

$$S = \frac{k\ F_{bo}{}^{5/6}\ F_s{}^{1/12}\ f^{1}\ (c)}{KQ^{1/16}} \qquad (A.3)$$

$$S = \frac{k\ F_{bo}{}^{7/8} f^{11}\ (c)}{K\ b^{1/4}\ d^{1/8}} \qquad (A.4)$$

$$S = \frac{k\ F_{bo}{}^{11/12}\ f^{111}\ (c)}{K\ b^{1/6}\ Q^{1/12}} \qquad (A.5)$$

$$K = 3.63\ g/\nu^{1/4} \qquad (A.6)$$

$$d = (q^2/F_b)^{1/3} \qquad (A.7)$$

$$V = (F_b\ F_s\ Q)^{1/6} \qquad (A.8)$$

$$F_b = F_{bo}\ (1 + 0.12\ C) \qquad (A.9)$$

and for sand-bed channels, the rough relation:

$$F_{bo} = 1.90\ d_{50}(\text{mm}) \qquad (A.10)$$

where F_b is the bed factor

F_s is the side factor

k is a meander correction coefficient

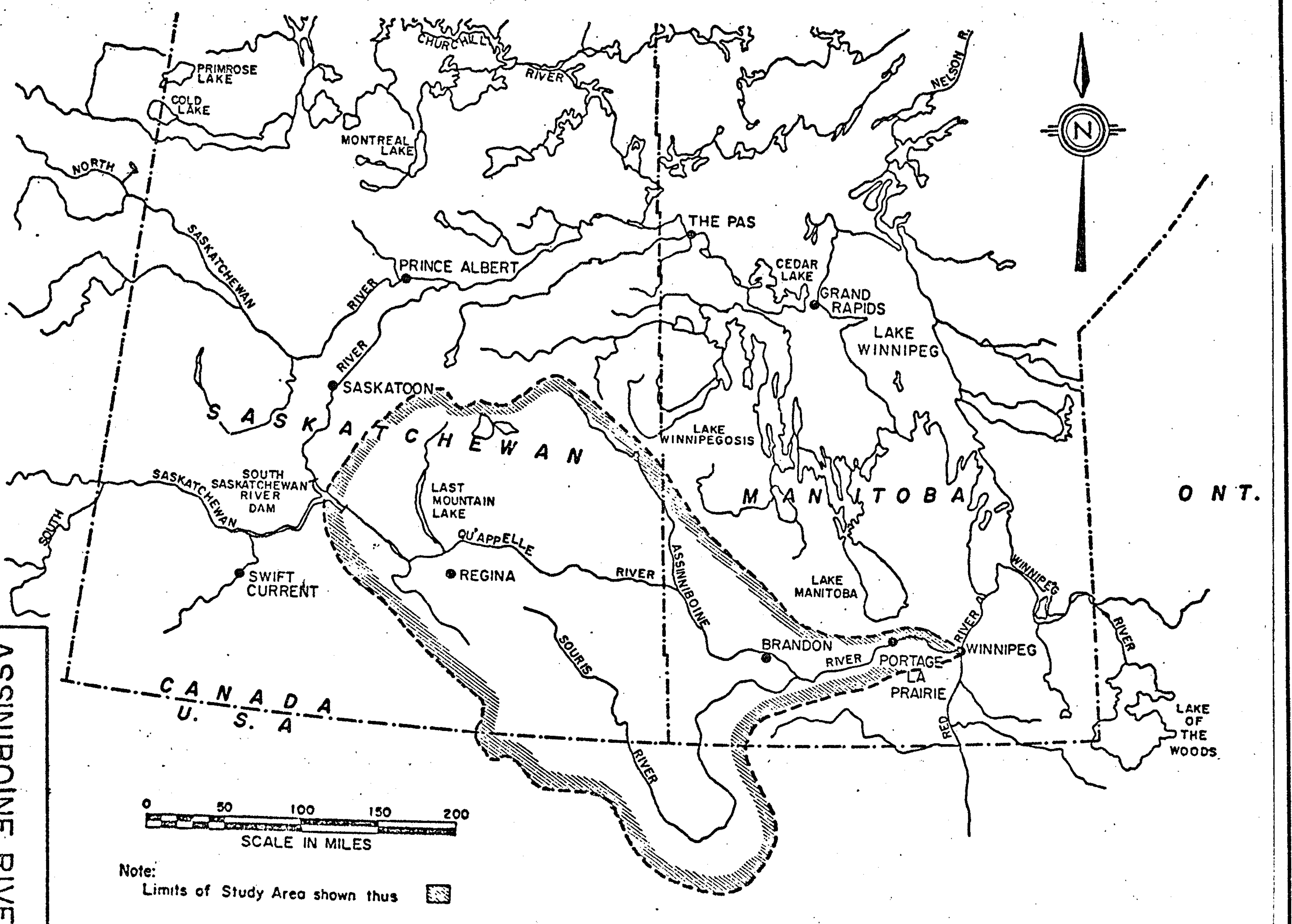
F_{bo} is the zero bed-factor

$f(c)$ is the bed-material load transport function

q is the unit water discharge per foot width

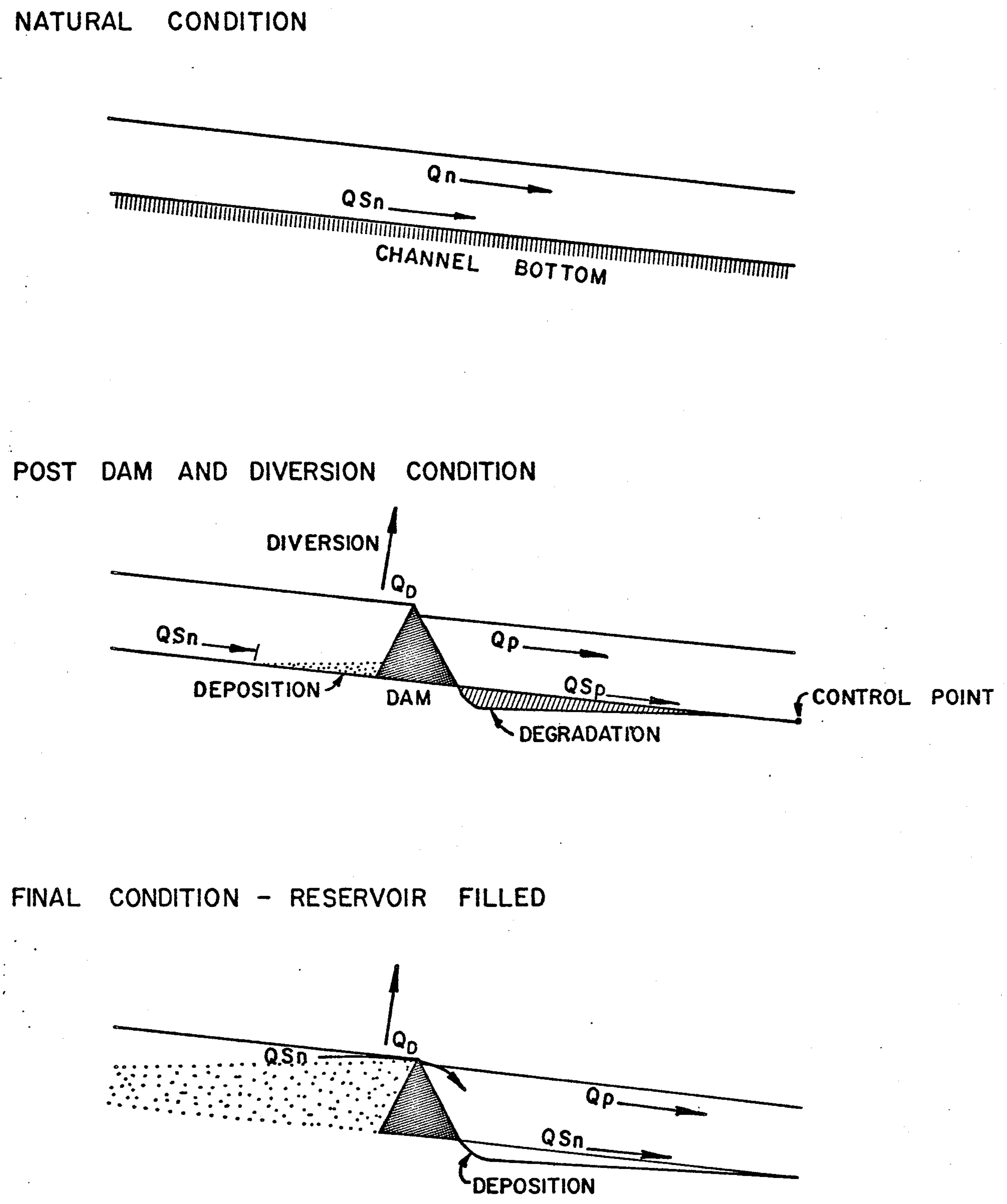
C is the sediment charge (parts per 100,000)

d_{50} is the mean sediment particle diameter (mm)



ASSINIBOINE RIVER DRAINAGE BASIN

FIGURE 1.



Qn is natural discharge spectrum

QSn is natural sediment flow

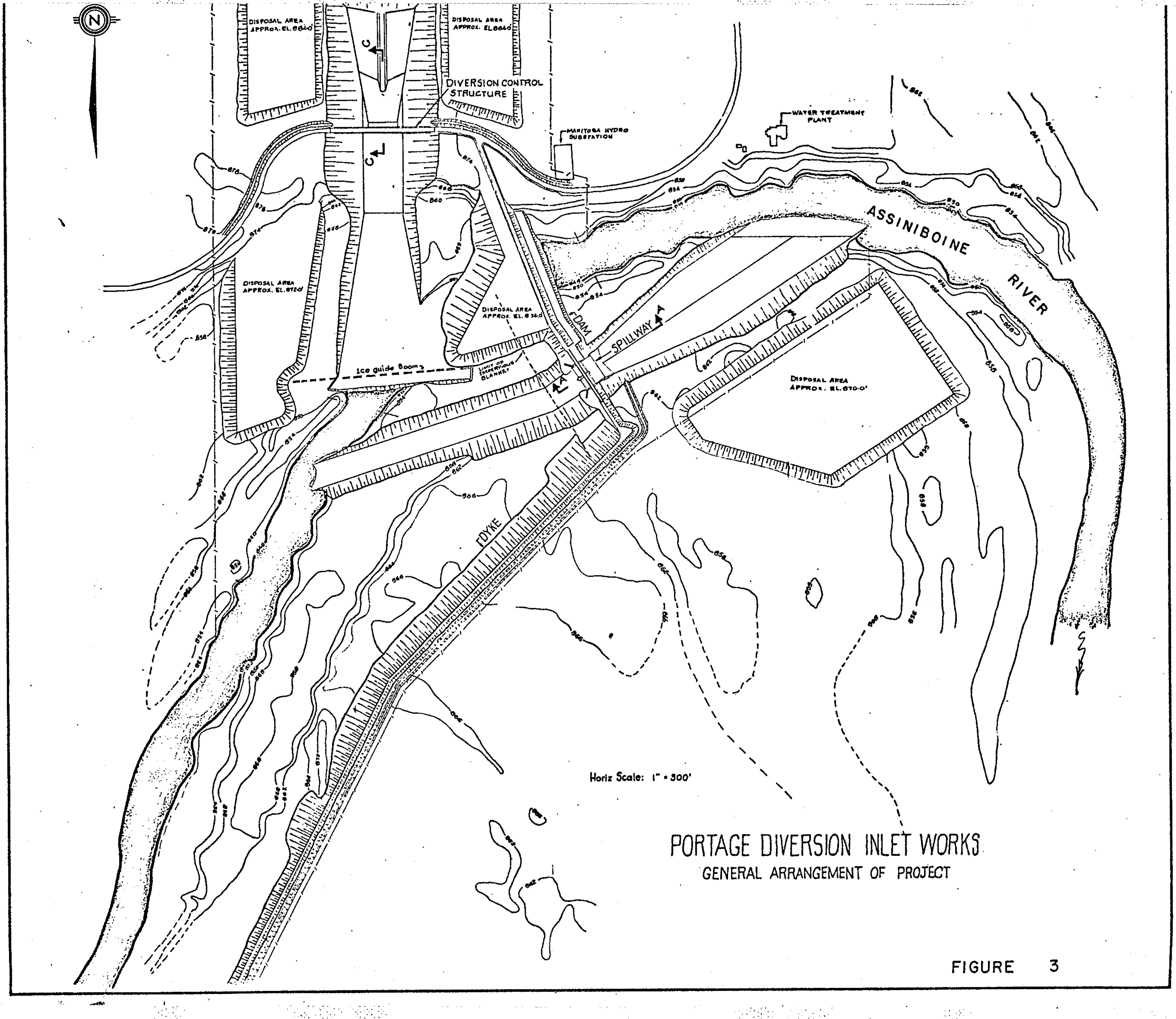
Q_D is diversion discharge

Qp is Qn minus Q_D

QSp is altered sediment flow

ILLUSTRATION OF DIVERSION CONSEQUENCES

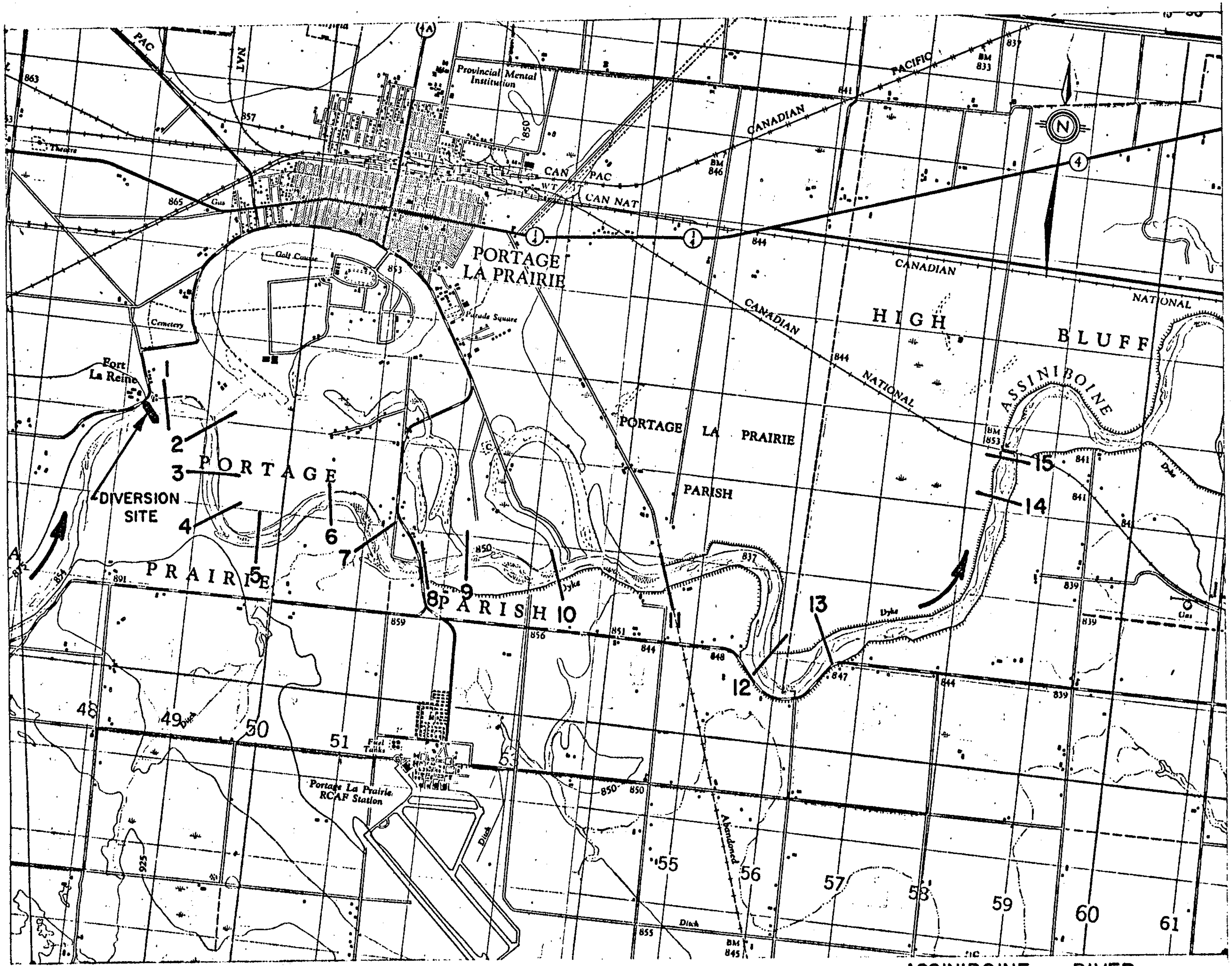
FIGURE 2



PORTAGE DIVERSION INLET WORKS

GENERAL ARRANGEMENT OF PROJECT

FIGURE 3



ASSINIBOINE RIVER

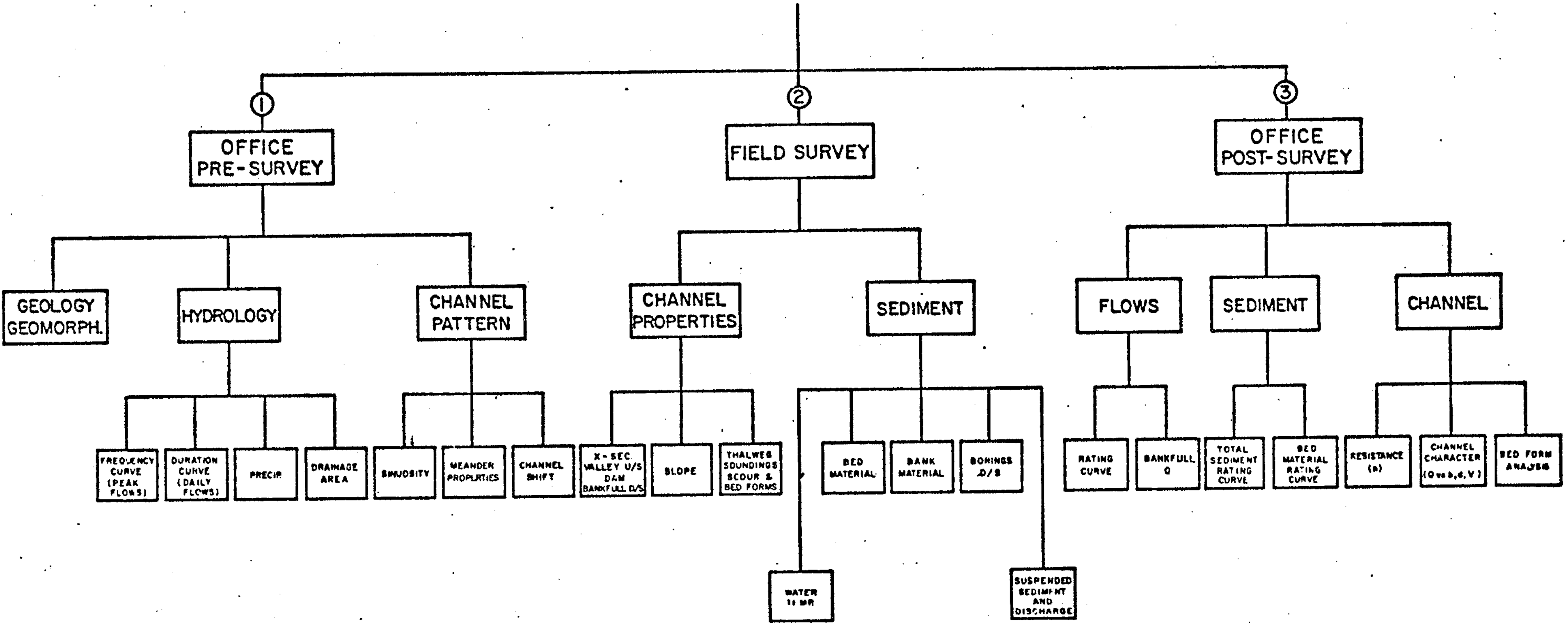
THE DOWNSTREAM REACH

SCALE : 1.25" = 1 mi.

FIGURE 4

RIVER INVESTIGATION PROGRAM

WHAT IS THE RATE AND EXTENT OF DEGRADATION?

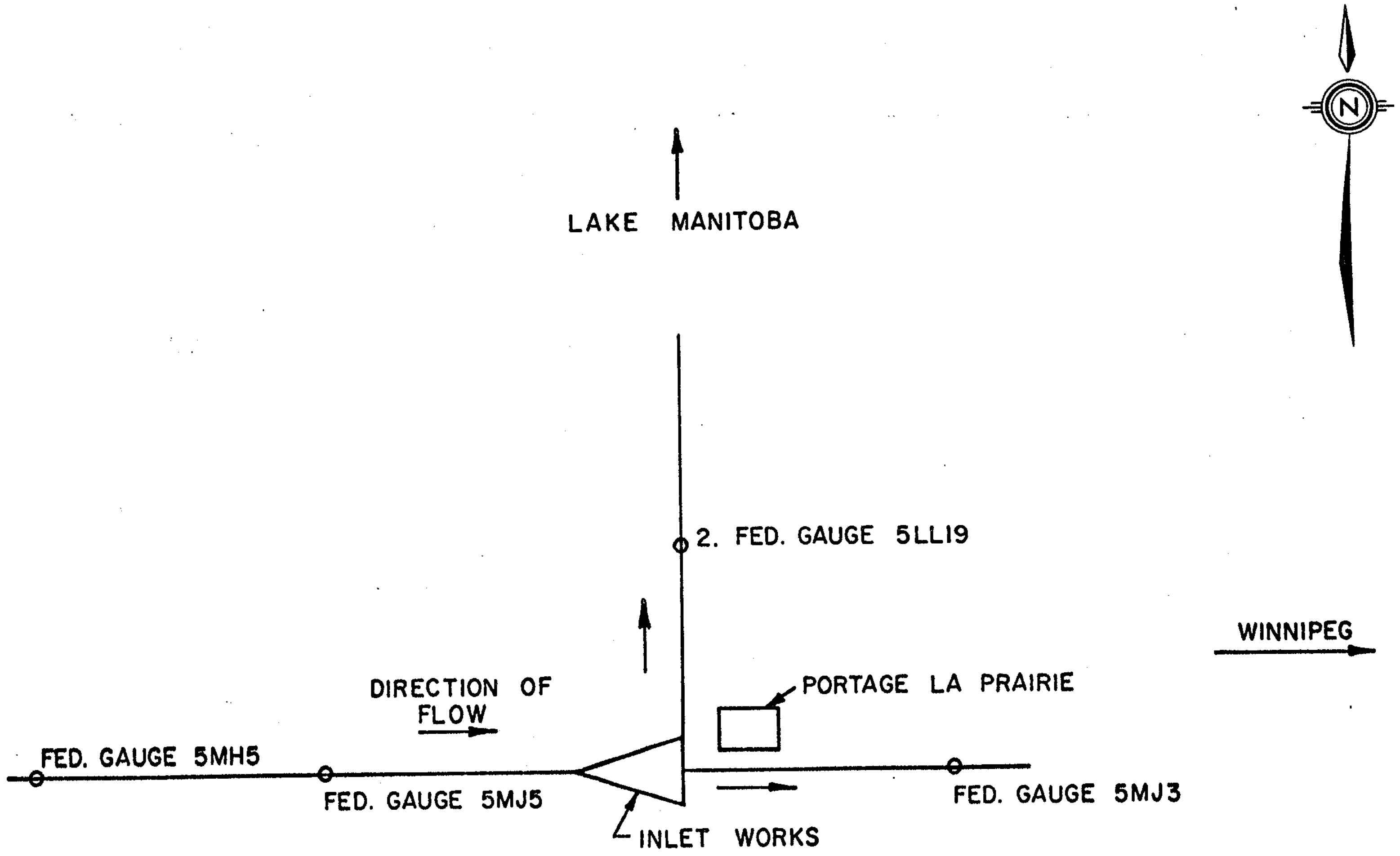


LEADS TO

FUTURE PREDICTIONS

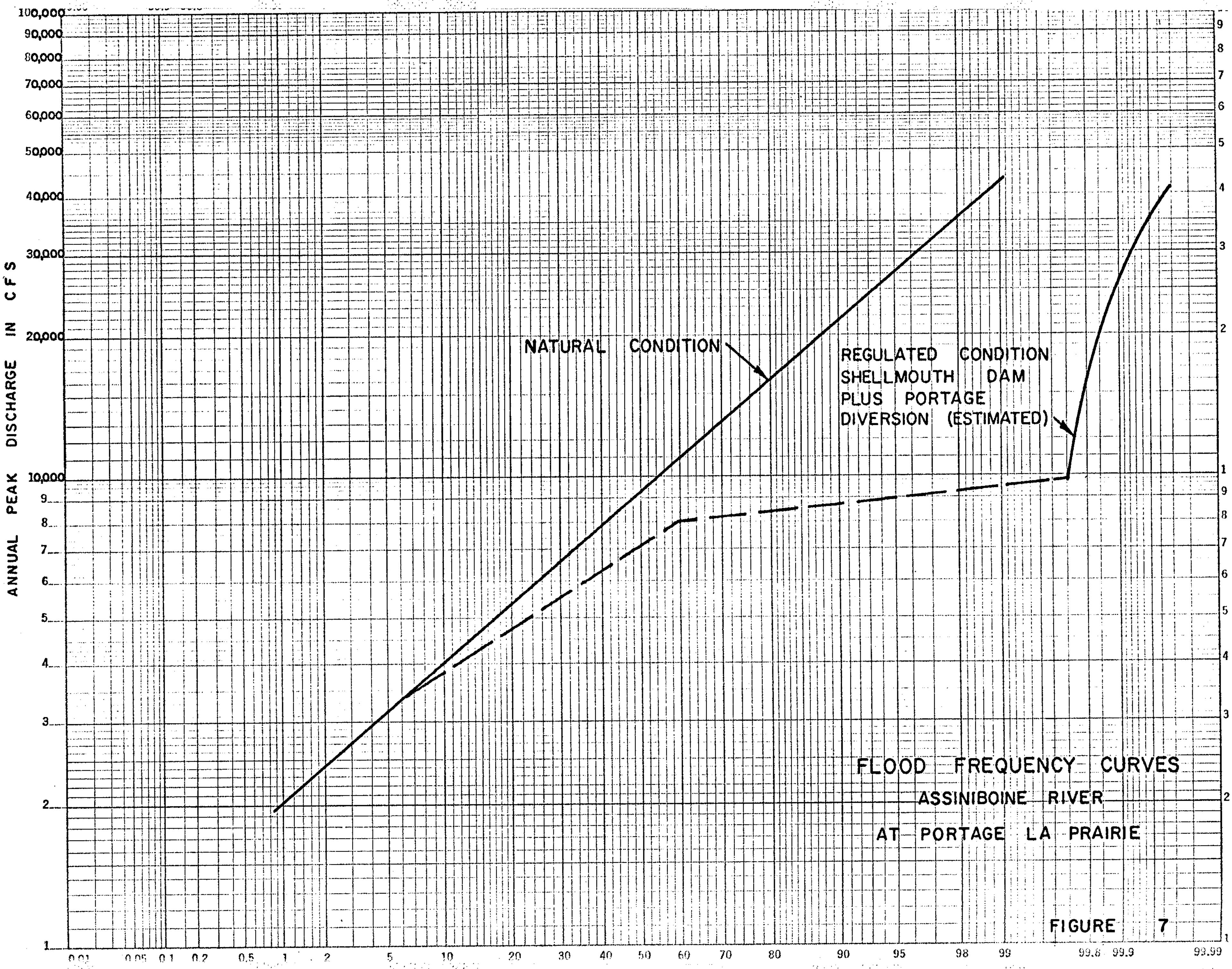
(DANGER TO D/S STRUCTURES)

FIGURE 5



ASSINIBOINE RIVER
GAUGING STATIONS IN THE
VICINITY OF THE PORTAGE
DIVERSION

FIGURE 6



FLOOD FREQUENCY CURVES
ASSINIBOINE RIVER
AT PORTAGE LA PRAIRIE

FIGURE 7

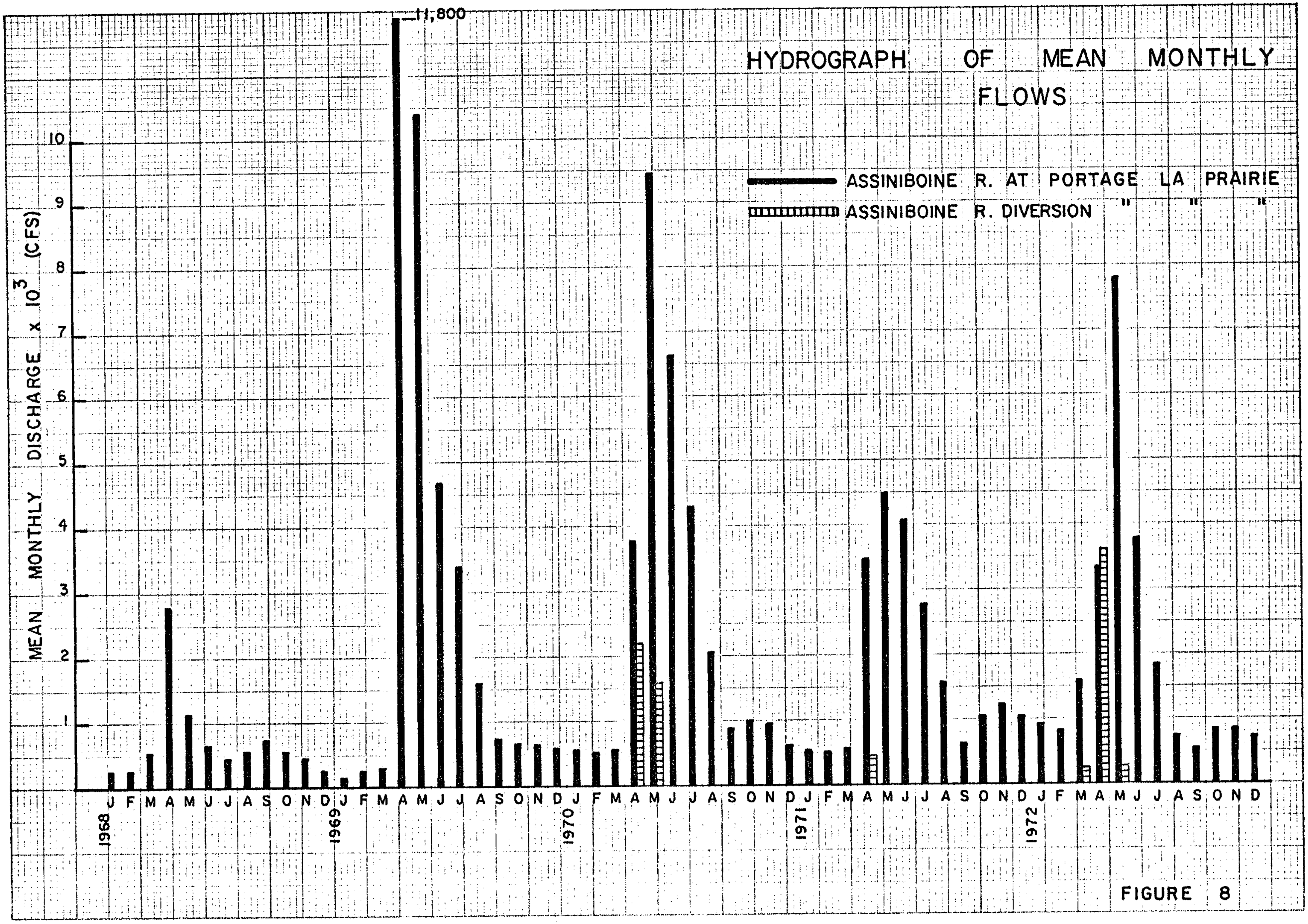
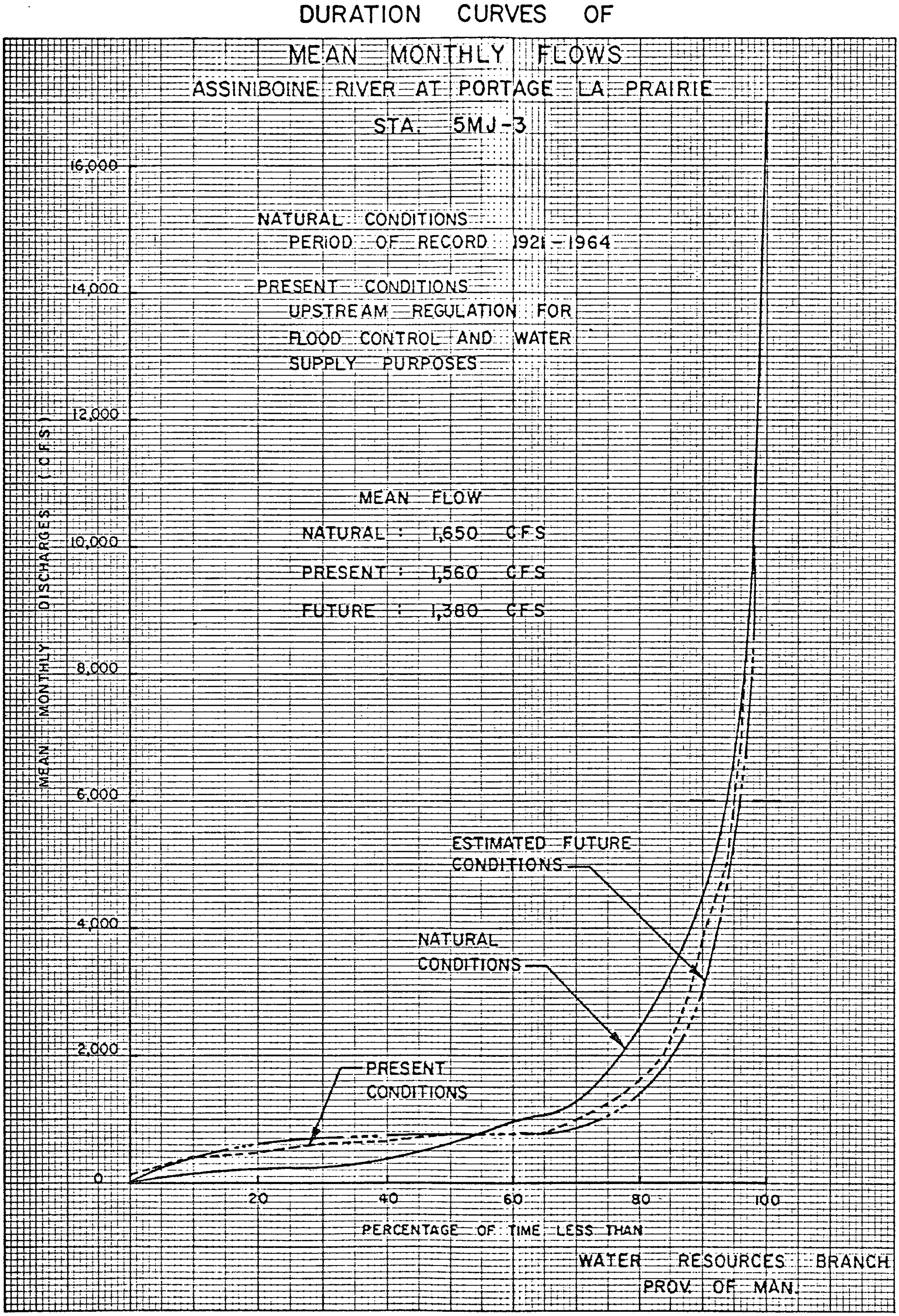

HYDROGRAPH OF MEAN MONTHLY FLOWS
ASSINIBOINE R. AT PORTAGE LA PRAIRIE
ASSINIBOINE R. DIVERSION " " "
MEAN MONTHLY DISCHARGE x 10^3 (CFS)
11,800
10
9
8
7
6
5
4
3
2
1
J F M A M J J A S O N D
J F M A M J J A S O N D
J F M A M J J A S O N D
J F M A M J J A S O N D
J F M A M J J A S O N D
1968
1969
1970
1971
1972

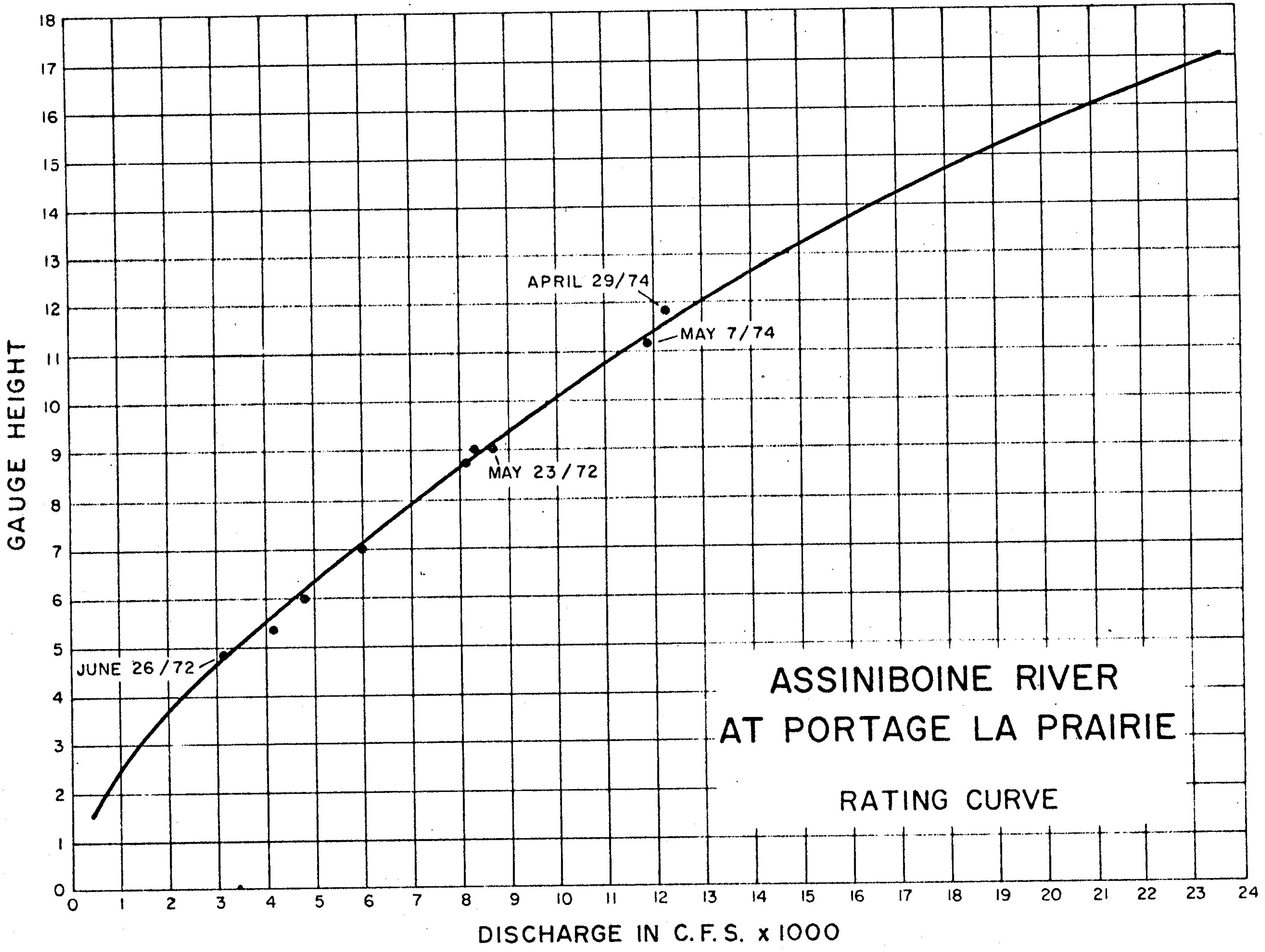
FIGURE 8

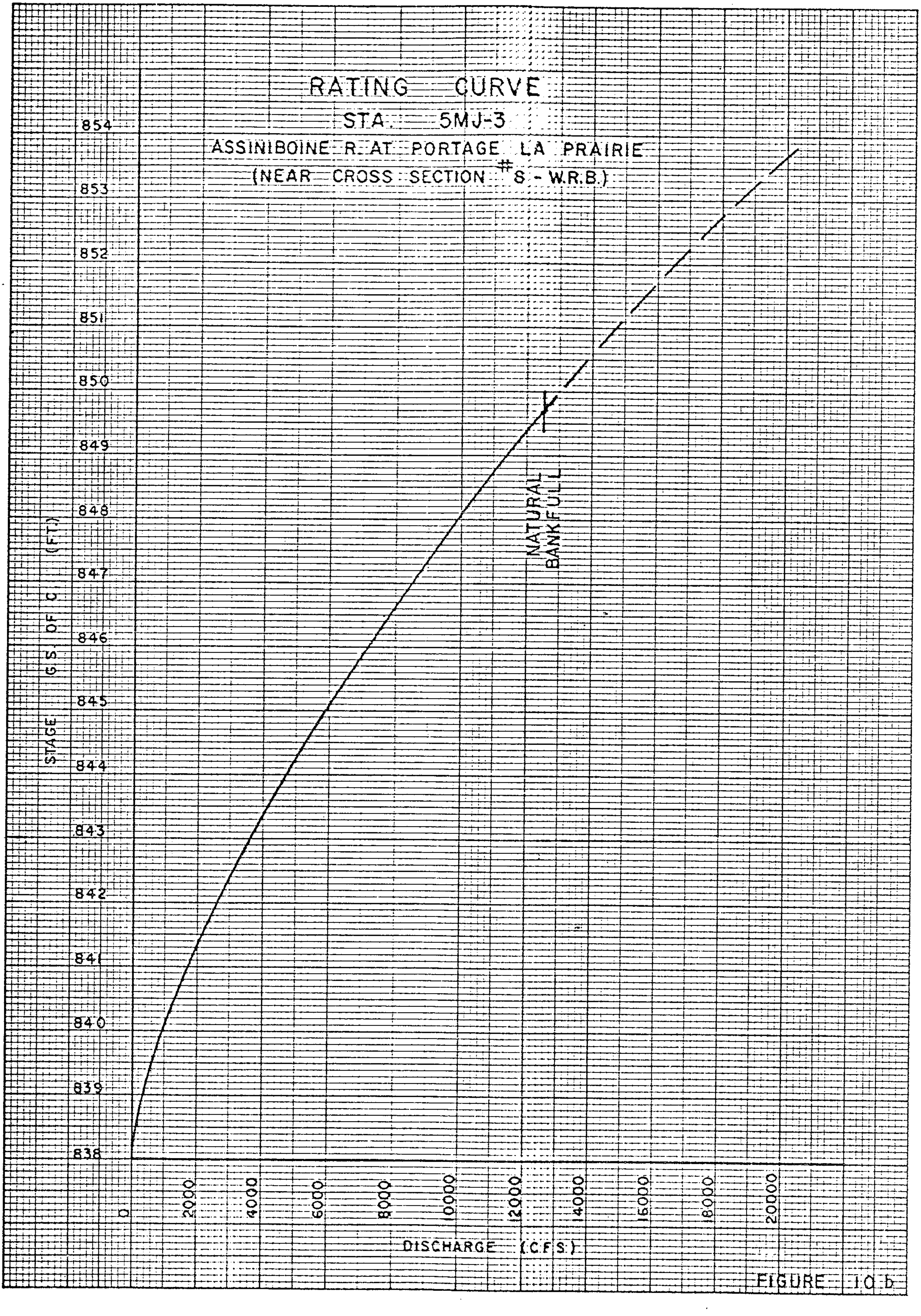


DURATION CURVES OF MEAN MONTHLY FLOWS ASSINIBOINE RIVER AT PORTAGE LA PRAIRIE STA. 5MJ-3

FIGURE 9

FIGURE 10a
PLATE 8





RATING CURVE

STA. 5MJ-3

ASSINIBOINE R. AT PORTAGE LA PRAIRIE

(NEAR CROSS SECTION #8 - W.R.B.)

FIGURE 10 b

WASH BORE LOGS

ASSINIBOINE RIVER AT PORTAGE LA PRAIRIE

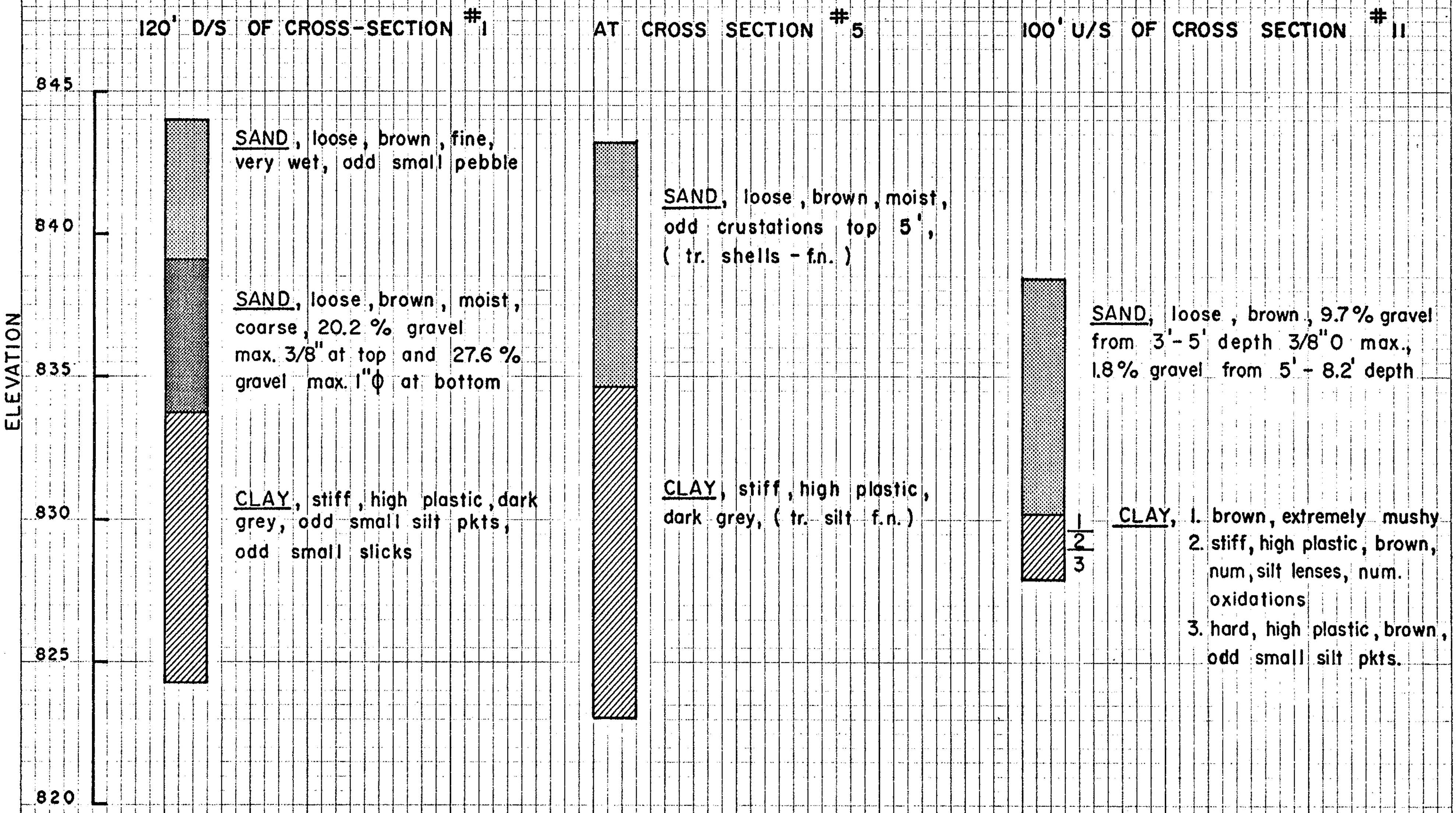
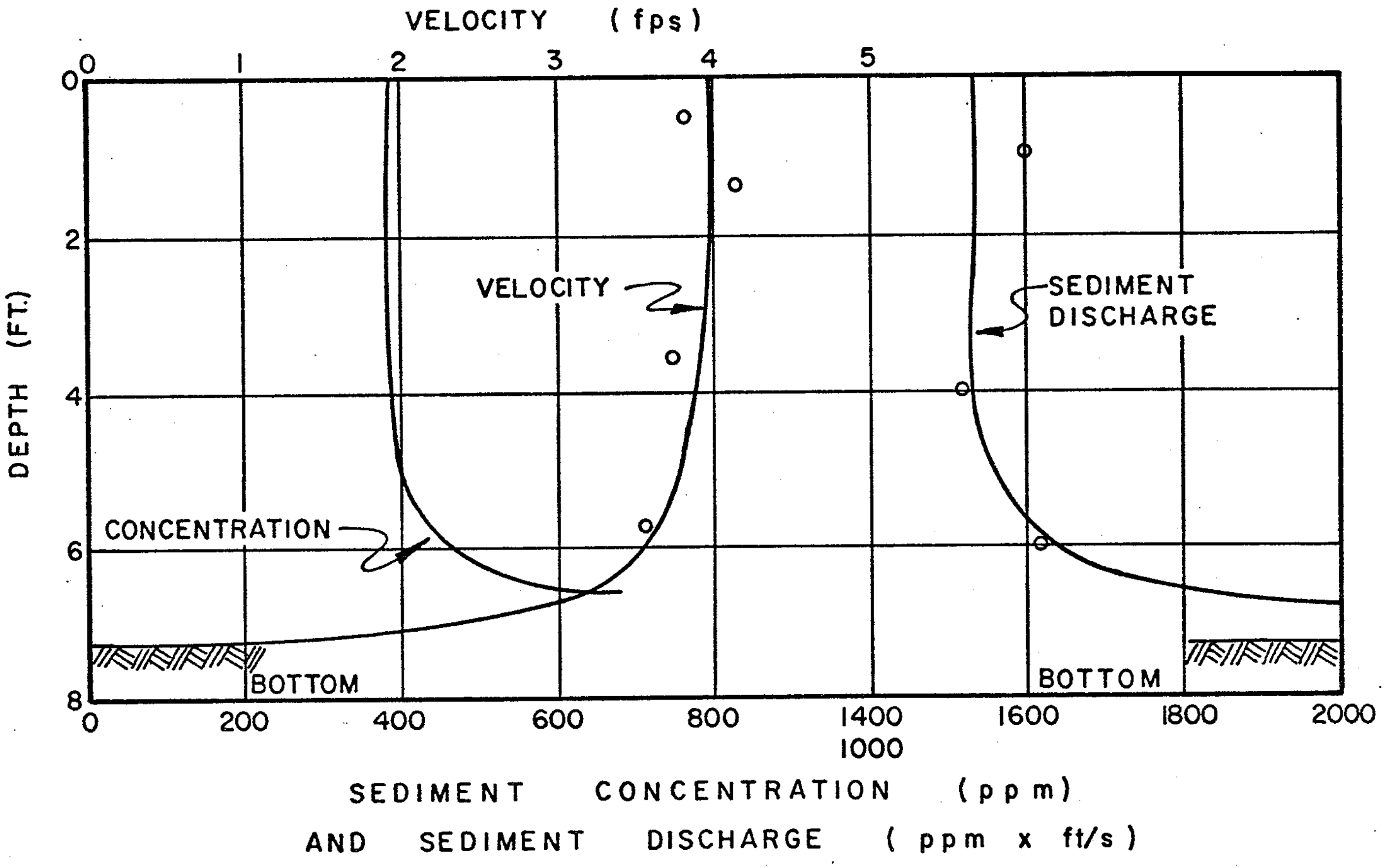


FIGURE 11.a



UNMEASURED SEDIMENT = 15 %

ASSINIBOINE RIVER AT PORTAGE LA PRAIRIE

SUSPENDED SEDIMENT CONCENTRATION

(AFTER KUIPER AND GALAY, 1971)

FIGURE 12

SEDIMENT TRANSPORT
RATING CURVE
ASSINIBOINE RIVER
AT
PORTAGE LA PRAIRIE

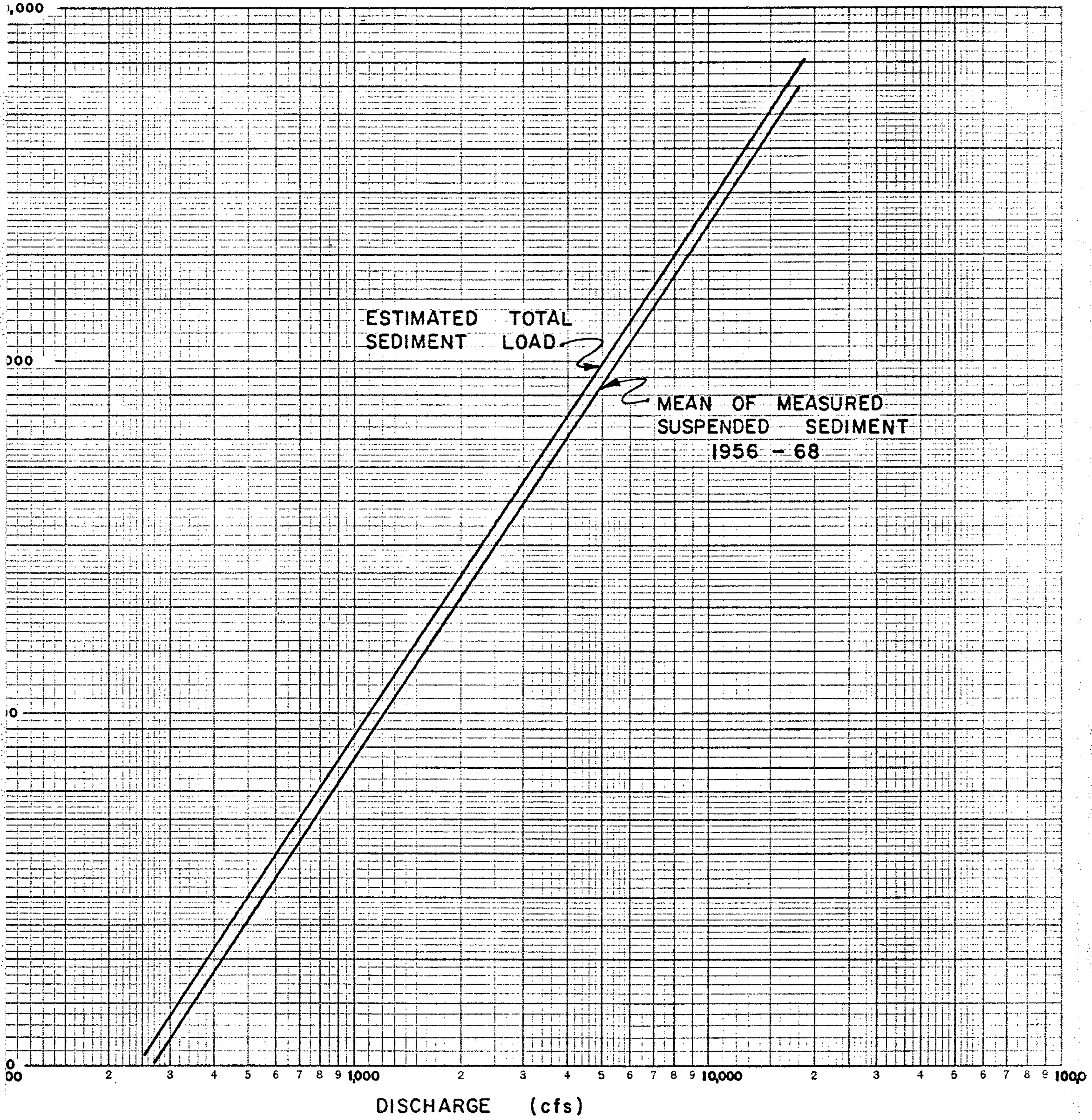
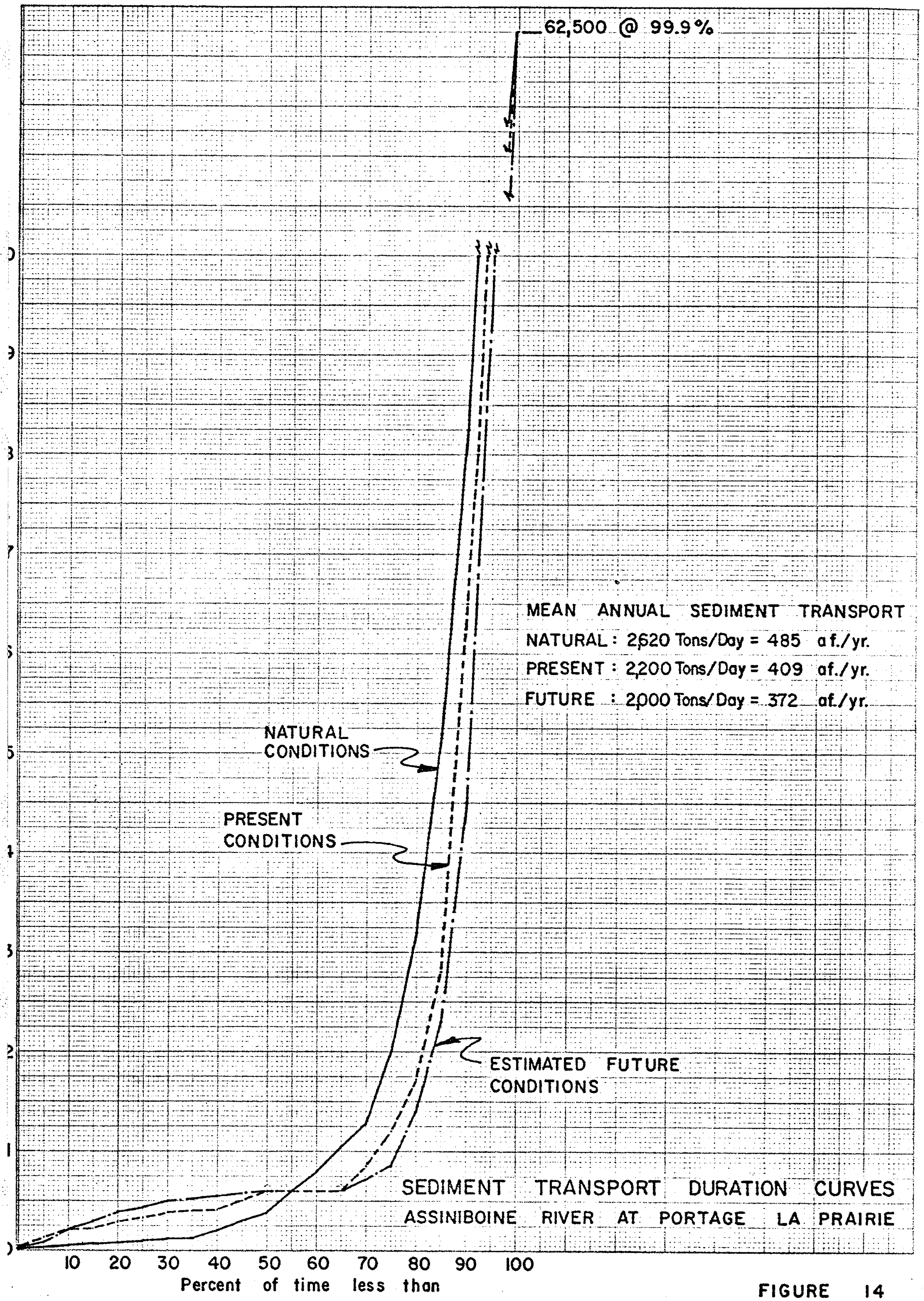


FIGURE 13



MEAN ANNUAL SEDIMENT TRANSPORT

NATURAL : 2,620 Tons/Day = 485 af./yr.

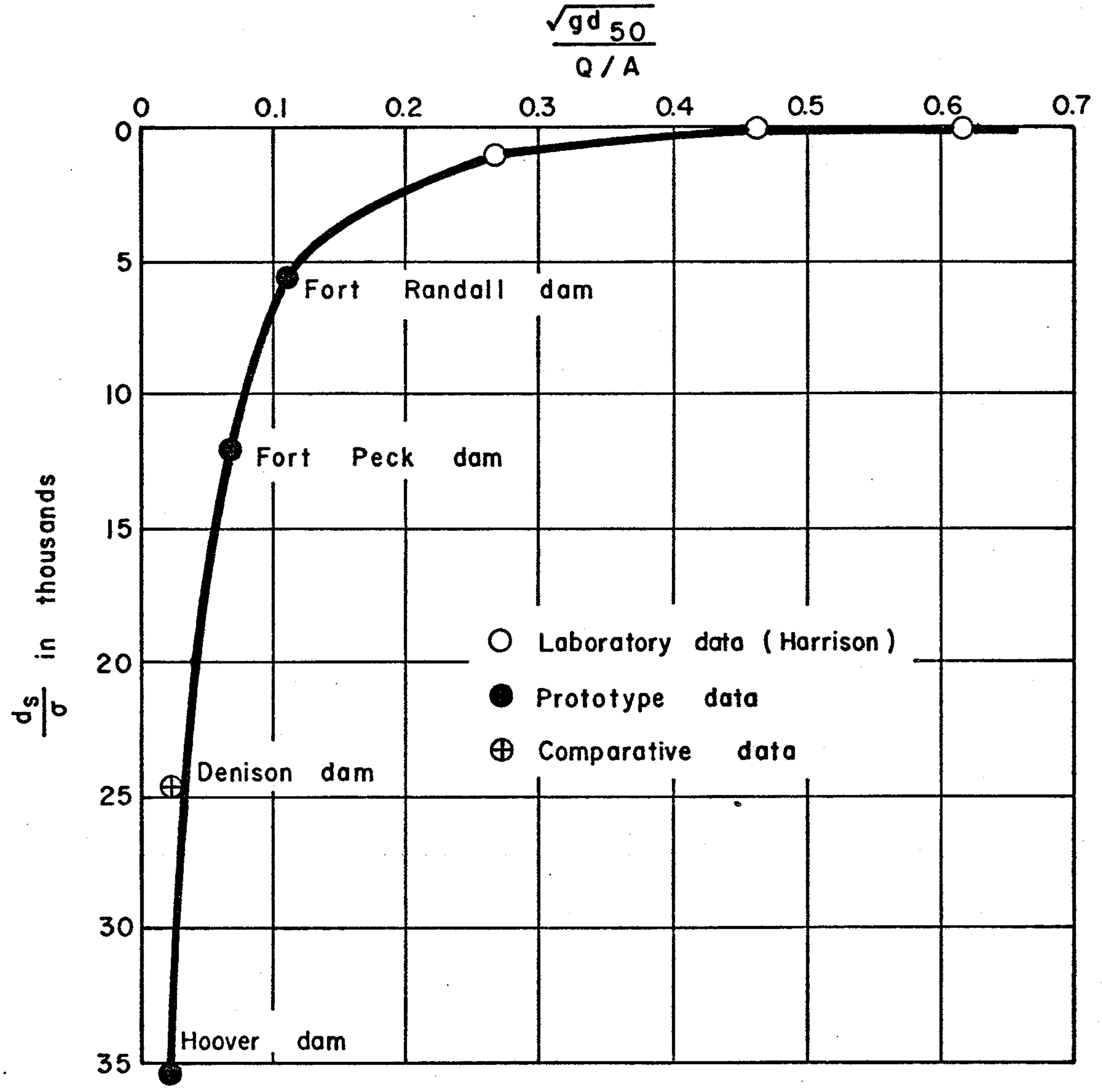
PRESENT : 2,200 Tons/Day = 409 af./yr.

FUTURE : 2,000 Tons/Day = 372 af./yr.

SEDIMENT TRANSPORT DURATION CURVES

ASSINIBOINE RIVER AT PORTAGE LA PRAIRIE

FIGURE 14



ULTIMATE CHANNEL DEGRADATION

(AFTER PRIEST AND SHINDALA, 1969)

WHERE : d_s is the ultimate degradation

d_{50} is a measure of the grain size taken as the grain diameter d at 50% finer

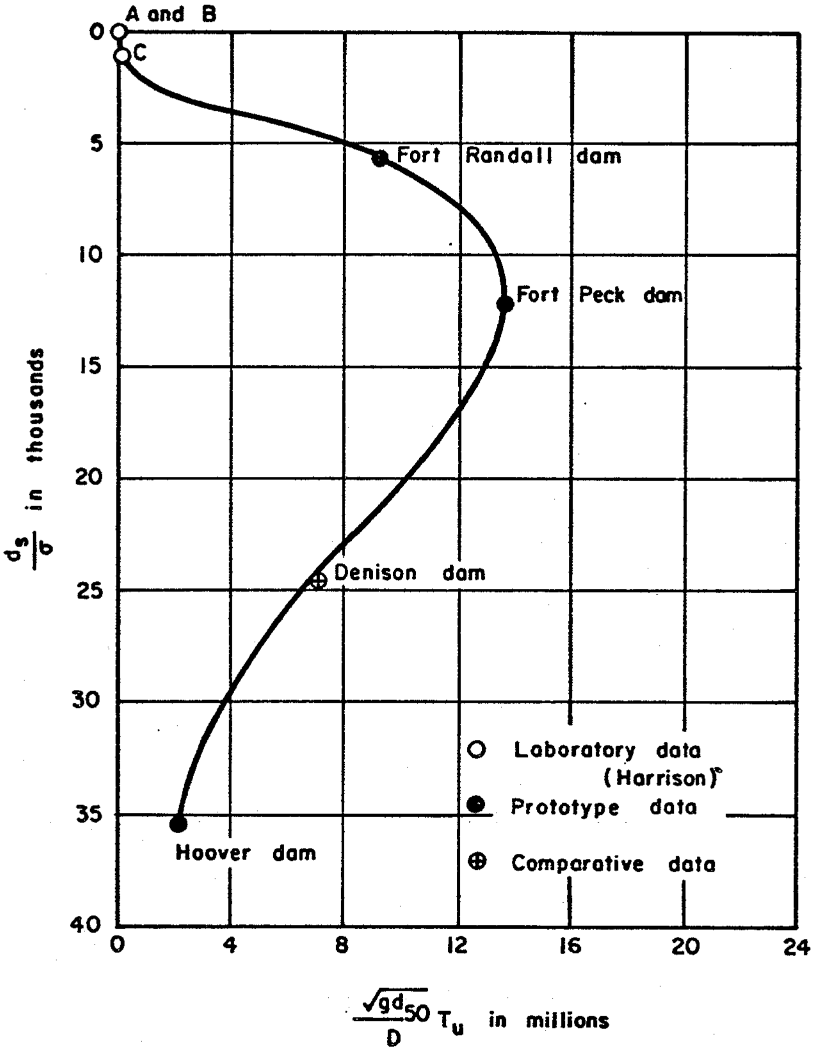
σ is the standard deviation from diameter d_{50} between 10 and 90% finer

Q discharge (an average of annual peak values of monthly averages over period of degradation)

A is cross-sectional area of the channel

g is acceleration due to gravity

FIGURE 15

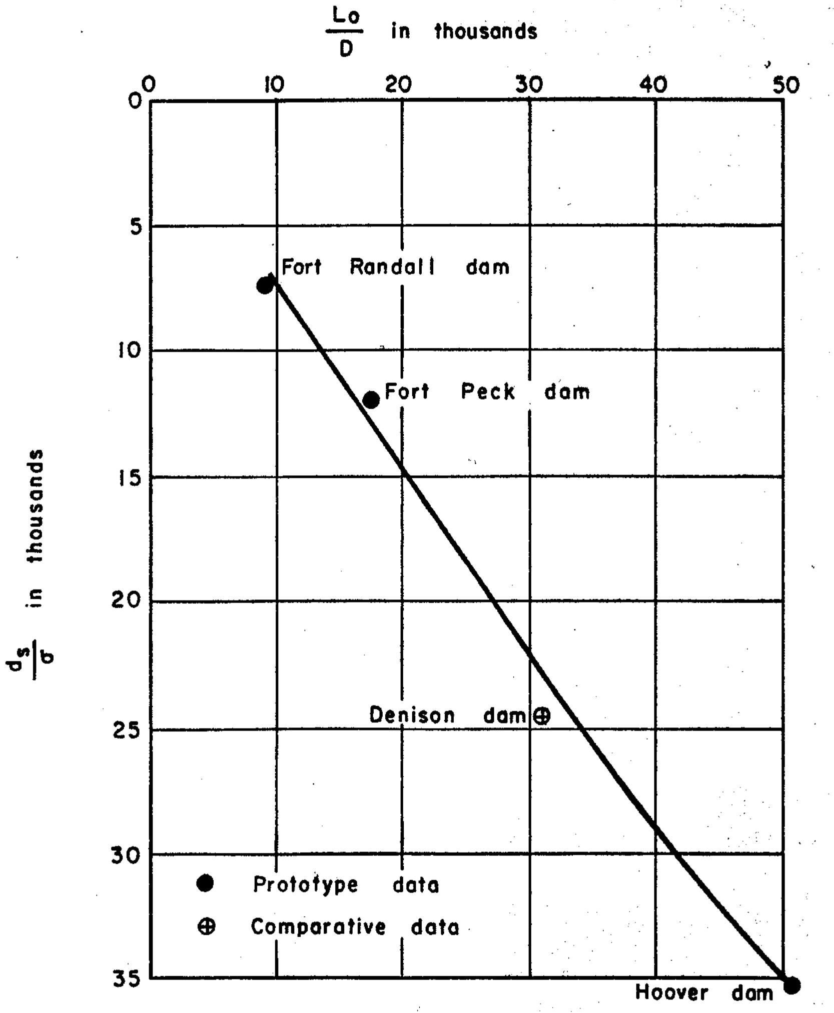


TIME REQUIRED FOR ULTIMATE DEGRADATION
(AFTER PRIEST AND SHINDALA, 1969)

WHERE: T_u is required time for ultimate degradation

D is average depth of flow for Q

FIGURE 16



THE DOWNSTREAM LIMIT OF CHANNEL DEGRADATION

(AFTER PRIEST AND SHINDALA, 1969)

WHERE : L_o is distance downstream to limit of channel degradation.

FIGURE 17

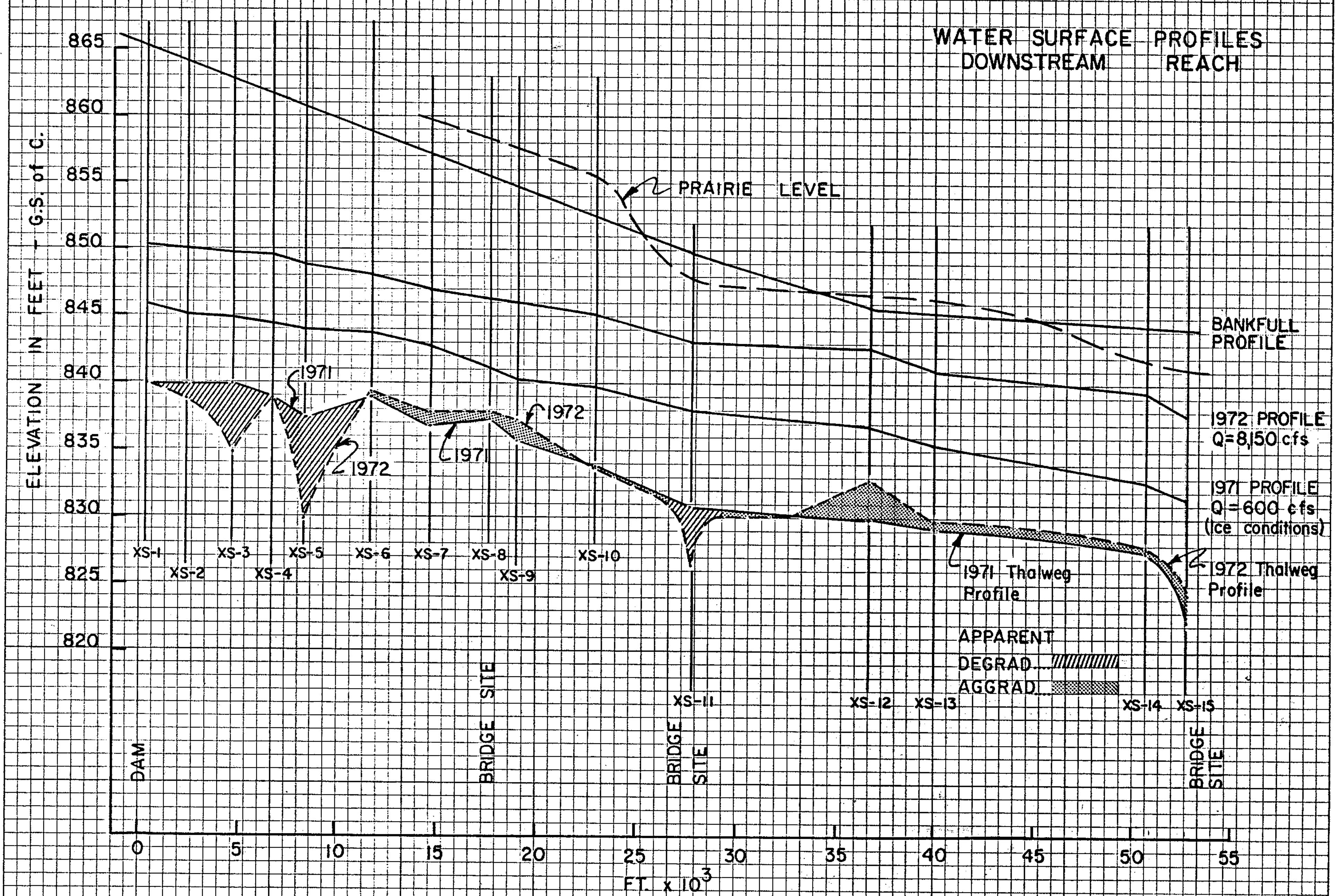
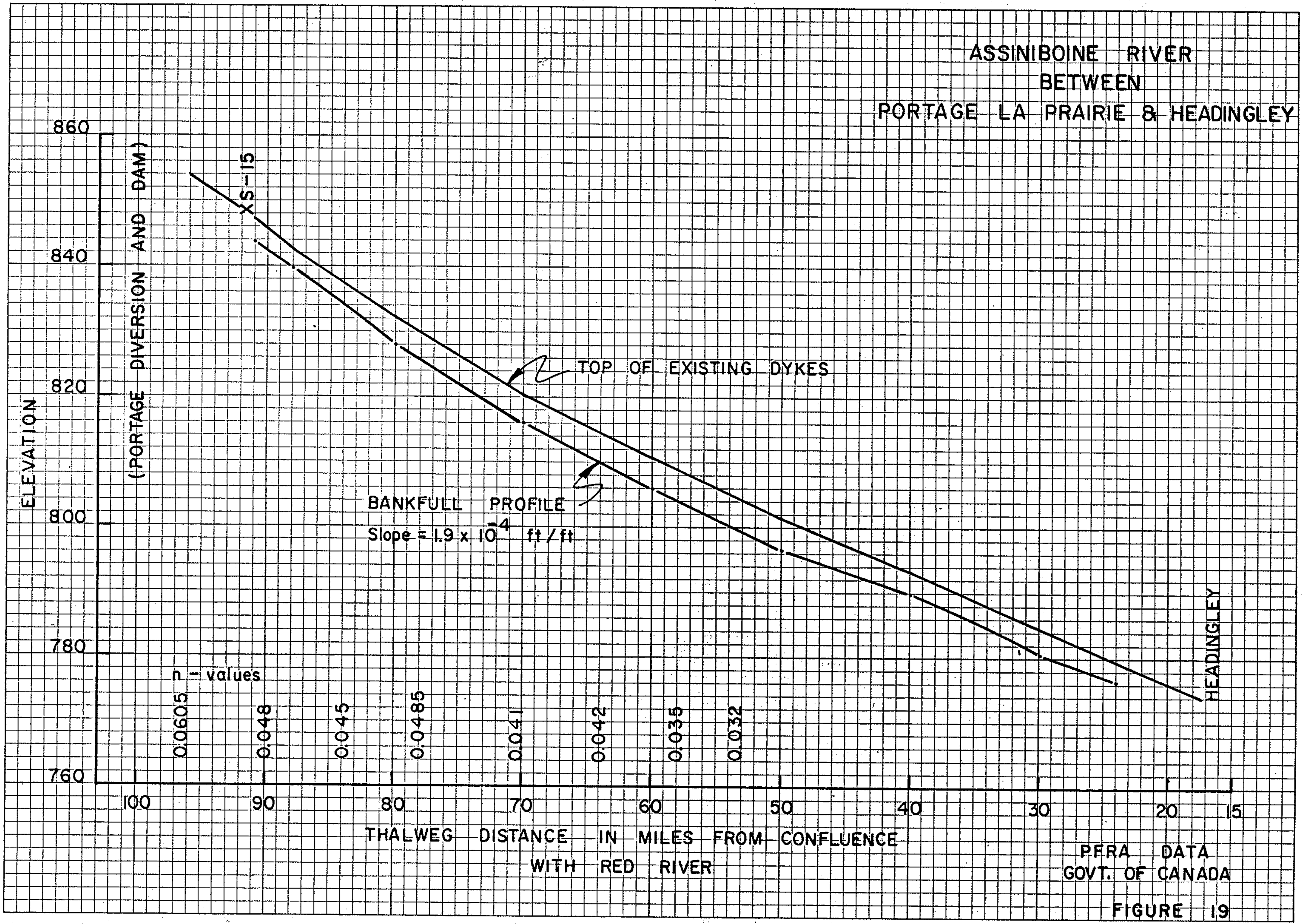

WATER SURFACE PROFILES
DOWNSTREAM REACH
ELEVATION IN FEET - G.S. of C.
865
860
855
850
845
840
835
830
825
820
PRAIRIE LEVEL
BANKFULL PROFILE
1972 PROFILE Q=8,150 cfs
1971 PROFILE Q=600 cfs (Ice conditions)
1971
1972
1972
1971
1972
1971 Thalweg Profile
1972 Thalweg Profile
APPARENT
DEGRAD. ▨
AGGRAD. ▩
XS-1
XS-2
XS-3
XS-4
XS-5
XS-6
XS-7
XS-8
XS-9
XS-10
XS-11
XS-12
XS-13
XS-14
XS-15
DAM
BRIDGE SITE
BRIDGE SITE
BRIDGE SITE
0
5
10
15
20
25
30
35
40
45
50
55
FT. x 10^3

FIGURE 18 a



ASSINIBOINE RIVER
BETWEEN
PORTAGE LA PRAIRIE & HEADINGLEY

PFRA DATA
GOVT. OF CANADA

FIGURE 19

THE DOWNSTREAM REACH
CROSS - SECTIONS 1 TO 15
ASSINIBOINE RIVER AT PORTAGE LA PRAIRIE
MEAN CROSS-SECTION vs DISCHARGE CURVE
(FROM 1971 AND 1972 W.R.B. CROSS-SECTION DATA)

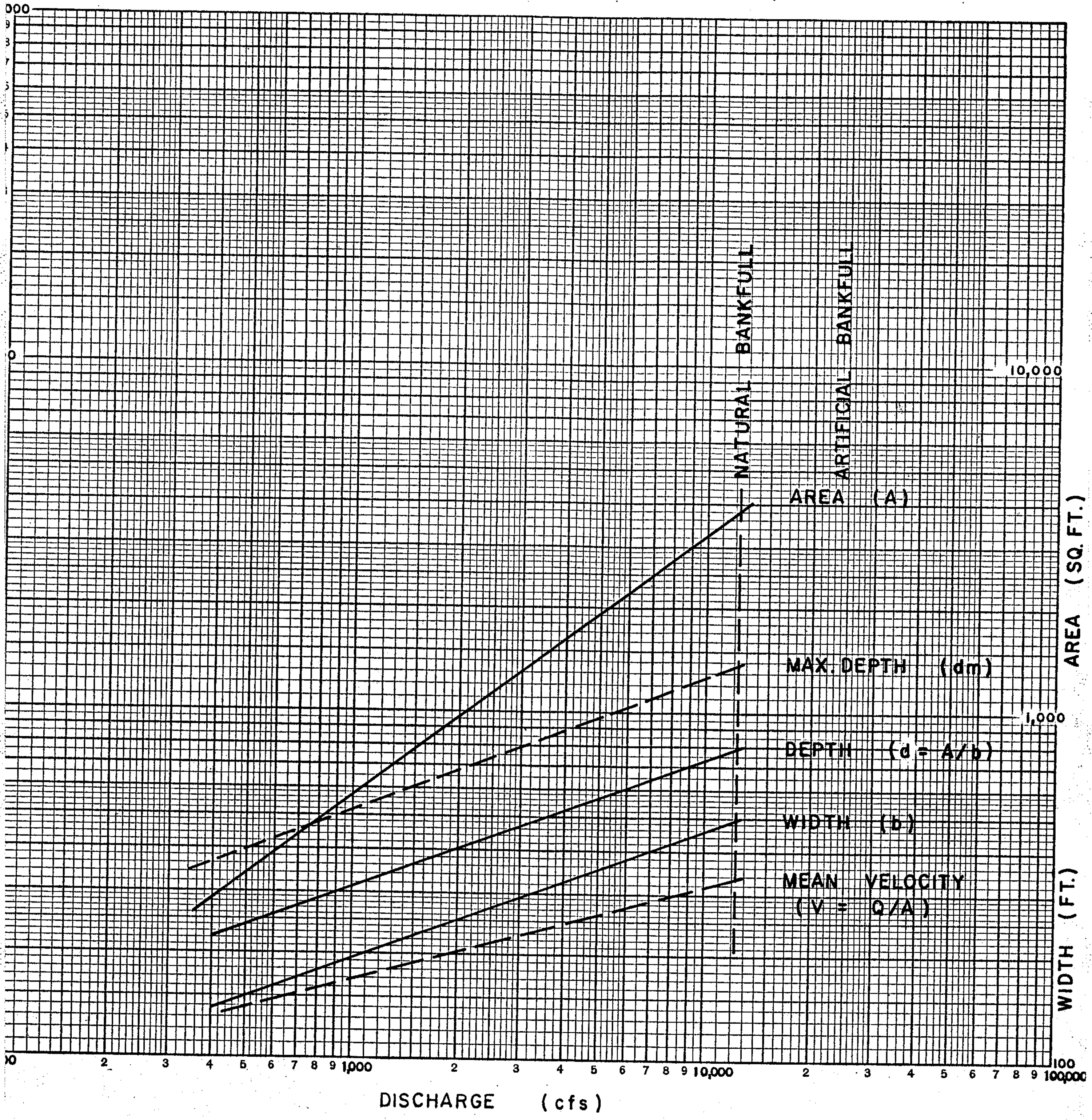


FIGURE 20

THE DOWNSTREAM REACH

HYDRAULIC ROUGHNESS PROFILE

(USING MANNING FORMULA)

1972 data Arithmetic mean = 0.0266

6th root law " " " = 0.0316

(normally used for gravel bed rivers, included for comparison purposes only)

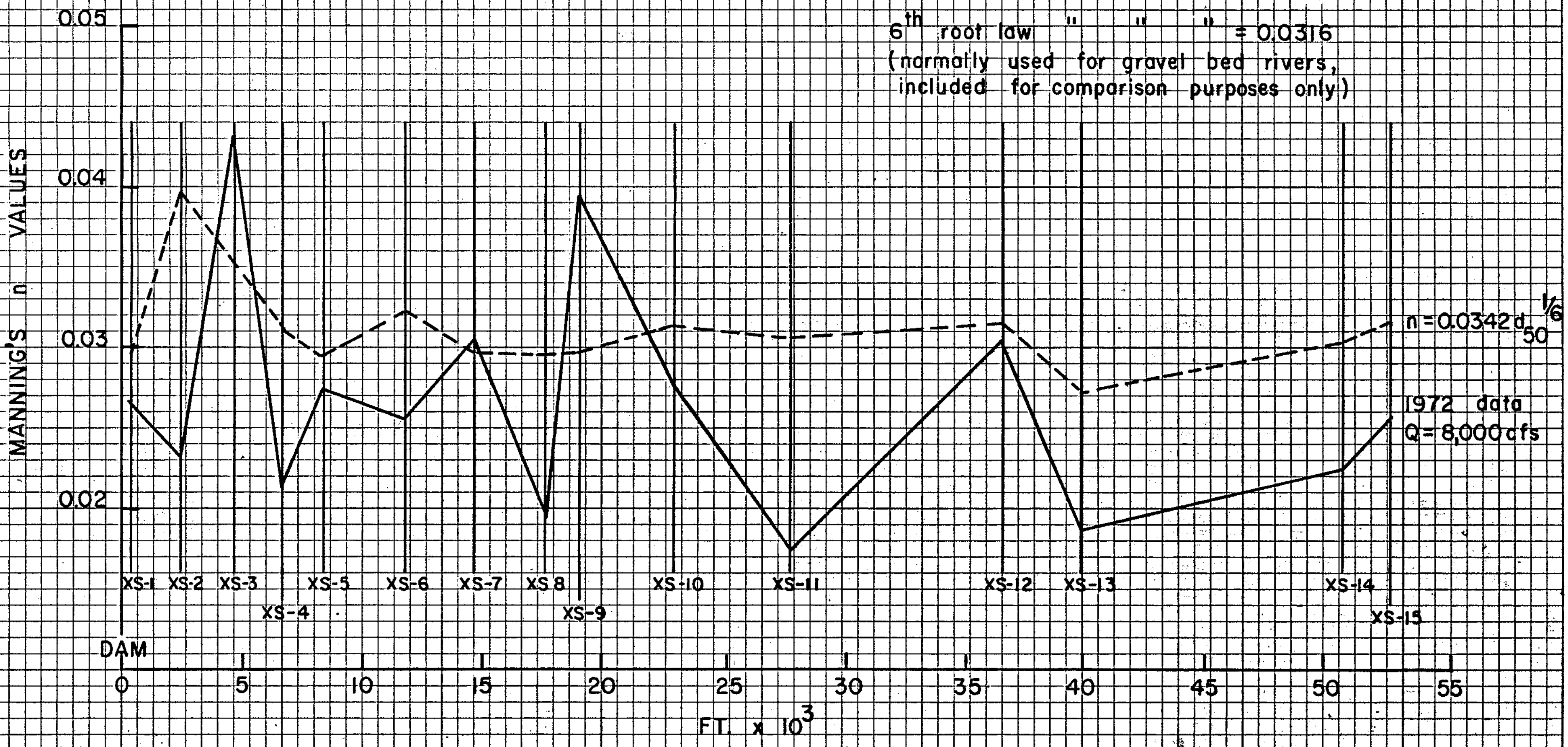
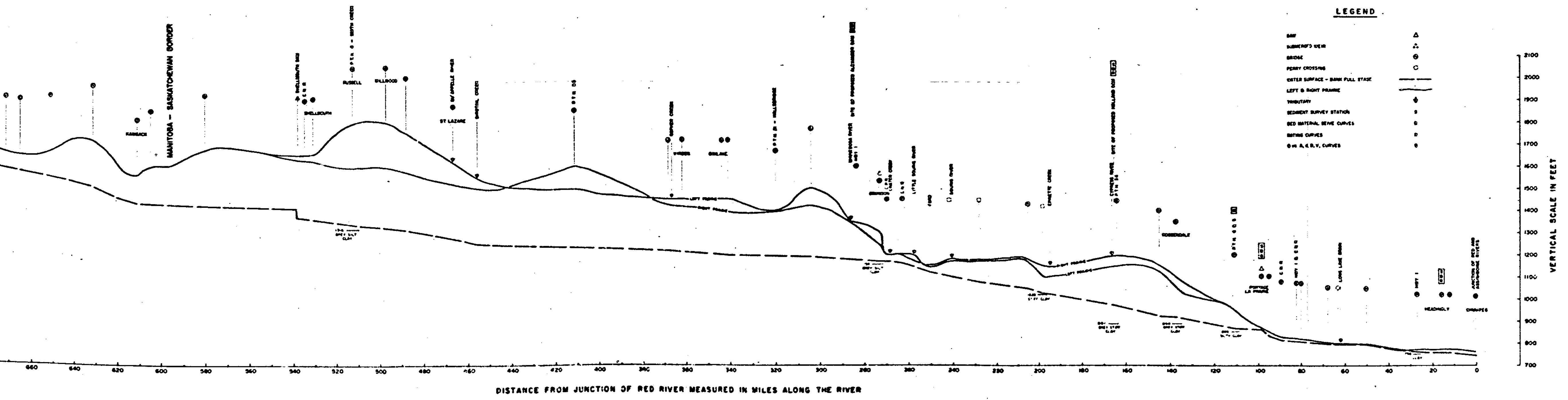


FIGURE 21



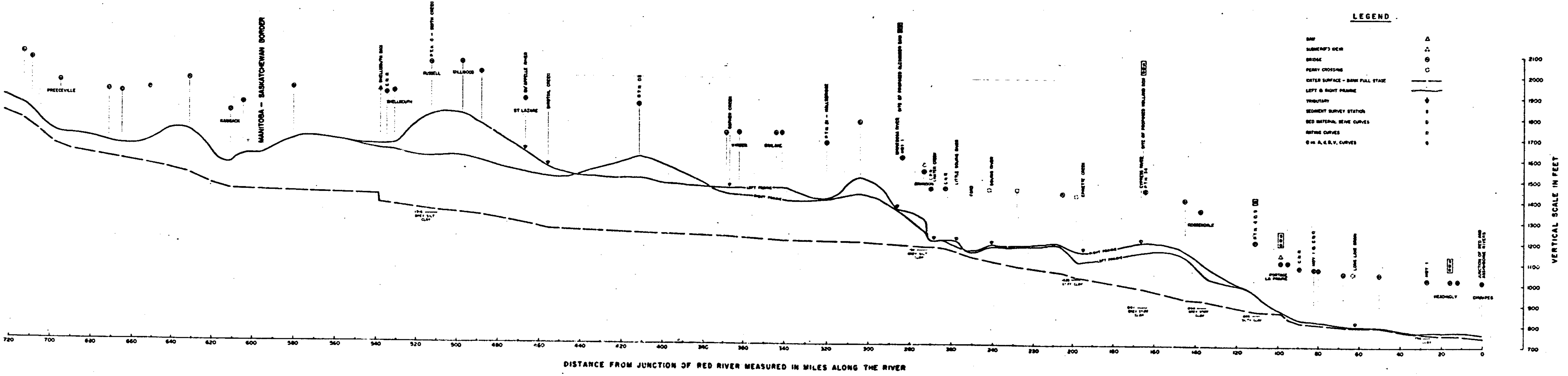
LONGITUDINAL PROFILE

OF THE

ASSINIBOINE RIVER

FIGURE 22

PLATE



LONGITUDINAL P

OF THE

ASSINIBOINE R