PROVINCE OF MANITOBA

DEPARTMENT OF MINES, RESOURCES AND ENVIRONMENTAL MANAGEMENT

WATER RESOURCES BRANCH

THE RED RIVER FLOODWAY

HYDRAULIC STUDIES

February, 1970.
Winnipeg, Manitoba.

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S Y N O P S I S

This report presents, in detail, the hydraulic investigations and design studies carried out in connection with the Red River Floodway together with a general description of the physical features of the project. In addition, a brief summary of previous investigations is included, together with descriptions of the basic features of related flood control projects on the Assiniboine River. Plates relating to the Floodway and hydraulic designs are presented and include hydraulic relationships for the computation of water surface elevations, through the City of Winnipeg, for all combinations of flow.

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INTRODUCTION

Glacial Lake Agassiz was created over what is now the Red River Valley during the retreat of the Pleistocene ice sheet until further recession opened up drainage through the Nelson River System. Flowing northward, to Lake Winnipeg, the Red River traverses the former Lake bed with a characteristically mild slope of approximately one-half foot per mile. The capacity of the River, which is approximately 40,000 c.f.s. at Emerson and 70,000 c.f.s. at Winnipeg, is frequently exceeded, during spring runoff, resulting in widespread flooding of the adjacent plains. At Winnipeg, the Red River, with a drainage area of 48,000 square miles, is joined by the Assiniboine River with a drainage area of 63,000 square miles. Disproportionately, however, the Assiniboine River contributes only 20% of the total flood flow.

Three major floods occurred during the early period of settlement of the Red River Valley.

Year	Estimated Discharge
1826	225,000 c.f.s.
1852	165,000 c.f.s.
1861	125,000 c.f.s.

During subsequent years, as development of the Red River Valley and Greater Winnipeg increased, only minor floods were experienced. It was not until the 1950 flood occurred, with a peak discharge of 103,600 c.f.s., that the damage potential was fully realized. Efforts were then initiated to find a solution to the flood problem culminating in the decision to proceed with construction of the Red River Floodway together with flood control works on the Assiniboine River.

The purpose of this report is to present, in detail, the various hydraulic studies carried out for the design of the Red River Floodway. These studies were primarily concerned with:

1. The computation of water surface profiles through Greater Winnipeg, for all floods, both under natural conditions and with the flood control works in operation;
2. The establishment of basic design criteria for the Floodway and Floodway structures; and finally,
3. The design of the Floodway and ancillary works with the exception of the inlet and outlet control structures which were designed by H.G. Acres and Company Limited and are discussed in separate reports.

In general, the Floodway location and capacity are in accordance with the recommendations made by the Royal Commission on Flood Cost-Benefit to the Government in 1958. The Commission estimated that the Floodway and Assiniboine River Projects would, in combination with the Greater Winnipeg dyking system, provide almost complete protection to the Greater Winnipeg Area against floods of up to 169,000 c.f.s. in magnitude, as measured below the confluence of the Red and Assiniboine Rivers, and which would be equalled or exceeded, on the average, once in 161 years. Individual projects, other than the Floodway, are discussed briefly in this report insofar as they affect the Floodway design.

Considerable data was collected and prepared during the previous studies, particularly by the Red River Basin Investigation. This material was reviewed and utilized to a large extent for the Floodway design, thus forming an integral part of the hydraulic studies.

A BRIEF OUTLINE OF PREVIOUS INVESTIGATIONS

The capacity and design of the Red River Floodway has been influenced to a large extent by previous studies on the Red and Assiniboine Rivers, notably by the Red River Basin Investigation and the Royal Commission on Flood Cost-Benefit. Besides the extensive hydrologic and hydrometric data collected during these investigations, the studies resulted in recommendations as to the projects to be incorporated along with the Floodway into a flood control system for the protection of Metropolitan Winnipeg. In addition, the Commission recommended capacities for the individual projects so that in combination they would economically provide the required degree of protection.

The Red River Basin Investigation

The Red River Basin Investigation, Water Resources Division, Engineering and Water Resources Branch, Department of Resources and Development of the Government of Canada, was formed following the 1950 flood, to investigate measures which would reduce the flood hazard in Winnipeg. The investigation included:

1. An hydrologic study of past floods and preparation of flood frequency curves;
2. Analysis of runoff, in the Red River Basin, to determine design floods; and
3. An investigation of a feasible flood forecasting procedure.
4. Preliminary designs and cost estimates, for design floods of various magnitudes, for the following basic types of flood control works:

(a) Channel improvements;

(b) Diversions;

(c) Dykes;

(d) Storage reservoirs.

The flood stage reduction in Winnipeg was determined for each of the works separately and for combinations of the works.

The results of the studies were presented in the Red River Basin Investigation report dated October 1, 1953. Appendix H of this report contains the main report prepared by the Prairie Farm Rehabilitation Administration (P.F.R.A.) Department of Agriculture, Government of Canada, which carried out a similar study on the Assiniboine River.

The Royal Commission on Flood Cost-Benefit

The Royal Commission on Flood Cost-Benefit was est blished by the Province of Manitoba in December, 1956, to compare costs with the benefits which would accrue from the flood control works investigated by the Red River Basin Investigation or from any other beneficial measures, and to make recommendations as to which measures should be implemented. The Commission submitted the results of the study to the Province in a report dated December, 1958, in which the following projects were recommended:

1. A 60,000 c.f.s. capacity floodway around Winnipeg;
2. A 25,000 c.f.s. capacity diversion of the Assiniboine River from Portage la Prairie to Lake Manitoba;
3. A storage reservoir on the Assiniboine River near Russell, Manitoba.

The Commission estimated that the capital cost of these projects would be $72,483,000, with a benefit-cost ratio of 2.73. These projects

would provide almost complete protection for the City of Winnipeg within the main dyking system against a flood of 169,000 c.f.s., as measured below the confluence of the Red and Assiniboine Rivers, with a frequency of occurrence of about once in 161 years.

Following the Commission's recommendations, additional studies were begun on each of the individual projects. These studies have led to some modification in location, design and operation of the preliminary designs; however, the capacities and function remain the same as those recommended by the Commission. The locations of the recommended projects are shown on Plate 1, Appendix A.

The capacity and design of the Red River Floodway, which will be the most effective of the recommended flood control works, are affected by the degree of protection required for Metropolitan Winnipeg and by the existing and proposed flood control works on the Red and Assiniboine Rivers. Since these works will form an integral part of the flood protection programme, they have been considered as completed projects with the operation of each project co-ordinated with the others by means of adequate flood forecasting and implementation of controls to provide maximum protection for Winnipeg. A brief description of each of the projects is included insofar as design of the Floodway is affected.

Greater Winnipeg Dyking: Following the 1950 flood, a dyking system was established to provide a measure of protection for the Greater Winnipeg area with all construction co-ordinated under the Greater Winnipeg Dyking Board. This system, enclosing the Red, Assiniboine and Seine Rivers, consists of broad boulevard dykes in low lying sections; lines of defence; pumping stations; and established borrow areas from which material can be obtained for emergency

dyking. The dyke profile was established approximately four feet below the maximum 1950 water surface profile. The dyke elevation at the James Avenue pumping station is 26.5 feet above the City of Winnipeg Datum, the zero of which corresponds at this location to elevation 727.57 Geodetic Survey of Canada Datum, 1929 adjustment. Provisions were made in the system to permit raising the dykes up to four feet higher by means of temporary, emergency dyking.

Modifications have been made to the city dykes since initial construction and a survey of the complete system was carried out in 1961 by the Water Control and Conservation Branch to determine the existing dyke profile from which the safe carrying capacity of the Red River could be established. For this prupose, the elevation at Redwood Bridge provides an adequate reference. The dyke elevation at Redwood Bridge is 752.5. Allowing one foot of freeboard, the safe capacity of the river, under natural conditions, is 80,000 c.f.s. Four feet of emergency dyking would increase the capacity to 102,000 c.f.s. or approximately equal to the 1950 peak discharge of 103,600 c.f.s. The safe capacity of the Red River would be reduced, however, when the Floodway is in operation, by the backwater effect extending upstream from the Floodway outlet at Lockport.

<u>The Portage Diversion</u>: Investigations on the Portage Diversion subsequent to the Commission's recommendations have led to selection of the Fort La Reine Route rather than the High Bluff Route recommended by the Commission. The Fort La Reine Route leaves the Assiniboine River just west of Portage la Prairie following a more or less straight route north to Lake Manitoba. The primary purpose of the diversion is to reduce flooding between Portage and Winnipeg and to effect a reduction in flood stages in Winnipeg.

Controls on both the river and diversion will permit part or complete diversion of up to 25,000 c.f.s. from the Assiniboine River. When the 25,000 c.f.s. design capacity of the diversion is reached, excess flows will be passed down the Assiniboine River.

The Shellmouth Reservoir: The Commission recommended a storage reservoir on the Upper Assiniboine River near Russell, Manitoba which would; reduce flooding downstream, particularly in the Brandon Area; reduce flood peaks at Portage la Prairie and Winnipeg; and provide storage for water conservation purposes. Investigations carried out by the P.F.R.A., following the Commission's studies, indicated a more favourable location farther upstream near Shellmouth, Manitoba, which would have essentially the same storage capacity and design requirements as the original site.

A conservation storage of 340,000 acre-feet would provide a dependable flow of 400 c.f.s. It is estimated, however, that irrigation, domestic and industrial requirements will not reach this demand until the year 2000. Initially, therefore, the conservation level of the reservoir has been reduced and the storage allocated as shown in Table I, for the purpose of flood routing studies.

TABLE I - SHELLMOUTH RESERVOIR - ASSUMED ALLOCATION OF STORAGE

Elevation	Storage Acre-feet	Allocation of Storage
Invert of conduits	0	
		Conservation
1388	150,000	
		Conservation and flood control
1404	340,000	
		Flood control
1410 Spilleay Crest Elev.	430,000	

HYDRAULIC INVESTIGATIONS PERTAINING TO THE RED AND ASSINIBOINE RIVERS

Hydraulic investigations were carried out to establish, for design of the Floodway, the conditions which would exist for all floods both under natural conditions and with the recommended flood control works in operation. In addition, the studies established, in part, the basic criteria for the inlet and outlet structures for the Floodway. To a large extent the investigations were a review and extension of the hydraulic studies made by the Red River Basin Investigation.

Flood Frequency Curves.

Included in the Red River Basin Investigation studies was the preparation of flood frequency curves for the Red and Assiniboine Rivers. The Red River Basin Investigation estimated flows for the historic floods of 1826, 1852 and 1861 by reconstructing the water surface profiles from the available records and combined them with the 69 years of records for the period 1875-1951 to produce a frequency curve for the Red River peak flows below the confluence with the Assiniboine River. Since an acceptable curve could not be constructed by the usual mathematical methods, a curve was drawn "by eye" through the points plotted on logarithmic paper. Because of the method of construction used, the frequency curve would not be improved by addition to the records of the relatively low flows experienced since 1951 and was accepted as the best interpretation of the available data. The flood frequency curve for the Red River below the confluence with the Assiniboine River is reproduced on Plate 5, Appendix B.

The Red River Basin Investigation prepared a flood frequency curve for the Assiniboine River at Headingley, Manitoba, based on 39 years of record for the period 1913-1951. Although this frequency curve was not used in connection with Floodway design studies, it is reproduced on

Plate I, Appendix D for comparison with flows obtained from the correlation, between the Red and Assiniboine peak flows, discussed in the following section.

Correlation Between Peak Flows on the Red and Assiniboine Rivers.

The Assiniboine River joins the Red River in Winnipeg just below Norwood Bridge and, on the average, contributes approximately 20 percent of the flood peak in the City. The amount of Assiniboine River contribution to Red River flows is complicated by the fact that at flows approaching 20,000 c.f.s., the Assiniboine River overflows, in the reach between Portage la Prairie and Headingley, forming a large storage basin south of the River. A portion of the overflow flows south via the Rivière Sale, joining the Red River at St. Norbert, approximately one mile below the Floodway inlet. The remaining portion of the overflow ultimately drains back to the Assiniboine River after the peak has passed. Although some time lag would be involved in reaching the Red River at St. Norbert, the overflow could contribute to the Red River peaks because of the comparatively flat flood hydrograph of the Red River. The Assiniboine River contribution to the Red River peak is, therefore, made up of the flow at Headingley plus the portion of overflow which reaches the Red River via the Rivière Sale.

The P.F.R.A. developed a relationship between Assiniboine River flows at Portage la Prairie and Headingley by using the available records and by routing flood flows through the reach; the difference between the flows at the two locations being the overflow which would go into storage. A relationship between the total overflow and the portion which would contribute to the Red River peak via the Rivière Sale was developed by the engineering consultants for the Royal Commission on Flood Cost-Benefit and

is reproduced on Plate 5, Appendix D. Based on limited recorded data under low flow conditions, the extrapolated curve is largely hypothetical but has been accepted since possible inaccuracies would not materially affect the water levels determined using the relationship. The two relationships discussed above were combined to obtain the relationship, between flows at Headingley and total Assiniboine River contribution to the Red River, shown on Plate 3, Appendix D.

The Assiniboine River will contribute a varying percentage of the peak flows, on the Red River at Winnipeg, depending on the degree of coincidence and magnitudes of the floods in the respective drainage areas. Under natural conditions and for a particular flood, the water levels, through the reach controlled by the Floodway, would be relatively unaffected by the distribution of flows since the short backwater profile, from the junction of the Assiniboine and Red Rivers to the Floodway inlet, would correspond approximately to the natural profile obtained if all of the flow originated on the Red River. The effect on the water levels downstream of the Floodway inlet would be significant, however, when the Portage Diversion and Shellmouth Reservoir are considered in operation since, in general, these works become more effective with an increase in the size of the flood. A correlation between the peaks on the two rivers is required, therefore, in order to select the probable Assiniboine River contribution to any flood peak on the Red River.

Although correlations between the Red and Assiniboine Rivers peaks were developed during previous investigations, a new correlation was prepared which would be suitable for design purposes; take cognizance of the overflow contribution via the Rivière Sale; and, include the additional available records. The relationship between the peak flow on

the Red River below the confluence with the Assiniboine River, referenced for convenience at Redwood Bridge, and the contribution from the Assiniboine River was determined by fitting a line, by the method of least squares, through the plotted positions of the 49 years of records available for the period 1913-1961. The relationship is shown on Plate 2, Appendix D and is explained in more detail in Appendix .

Local Inflow

Approximately 80 percent of the flood flows on the Red River originate in the United States, south of Emerson. The remainder of the flood is contributed by the drainage basin, between Emerson and Winnipeg, which contains several small tributaries. The magnitude of this local inflow and coincidence with the main peak depend on the climatic conditions prevailing at the time of break-up. Since the Red River flows in a northerly direction, break-up generally begins in the upper watershed and the flood crest moving northward coincides with the peaks on the tributaries which enter from the east and west. Studies by the Red River Basin Investigation indicated, however, that the more severe floods tend to occur when there is a late spring break-up. Under these conditions the break-up is more rapid and basin wide with the result that peaks on the tributaries are receding by the time the flood crest on the Red River has advanced northward.

With the exception of the Assiniboine River flows, local inflows to the Red River in the Winnipeg area have been disregarded in the hydraulic studies. Flows on the two remaining tributaries, the Riviere Sale and Seine River, which enter the Red River upstream of Redwood Bridge, are considered

insofar as they are included in the recorded flows at this point and in the flood frequency curve. No attempt was made, therefore, to correlate the flows since conditions will remain essentially the same when the Floodway is in operation. The Riviere Sale, entering the Red River just below the Floodway control structure, has a minor effect on water surface elevations, for any particular flood, and has been disregarded except for the amount of Assiniboine River overflow which would be carried. Seine River flows will enter the Red River, upstream of the Floodway inlet, under design flood conditions. A portion of the flow, if the Seine River is at flood stage, will be diverted by the Seine River diversion between Ste. Anne and St. Adolphe and the remainder of the flow will join the flood waters from the Red River which will extend across to the Seine River upstream of the Floodway under design flood conditions. At lower stages, when there is no connection between the two rivers, Seine River flows will be permitted to enter the Floodway at the point of intersection.

Correlation of the runoff from the ungauged local areas was not attempted because the small area draining into the Red River, within the reach affected by the Floodway, would have an insignificant effect on the total flow and computed water surface elevations. Local runoff had to be considered, however, in the design of a drainage system along the Floodway and is discussed in a subsequent section on drainage.

The Rating Curve at Redwood Bridge

Redwood Bridge, located across the Red River approximately three and one-half miles downstream of the confluence with the Assiniboine River,

was selected as a gauging station in August, 1912. Daily gauge readings have been taken since that time. By combining discharge measurements taken in 1922 and during the 1950 flood, a rating curve was constructed for this location applicable to open water conditions with the St. Andrews Dam at Lockport open. The rating curve was extended above the highest metered discharge of 103,600 c.f.s. by the Red River Basin Investigation which extrapolated the curve through the points representing the reconstructed floods of 1826, 1852, and 1861.

Discharge measurements made at the section, subsequent to 1950, indicated a progressive shift of the rating curve to the right reaching a maximum in 1956 and remaining relatively constant since then. The extent of the shift represents an increase in the carrying capacity of the Red River of approximately 5,000 c.f.s. between 1950 and 1956.

Before any hydraulic designs could be carried out, it was necessary to establish whether the increased capacity of the Red River was a permanent effect which should be considered in the hydraulic studies and designs. Possible causes for the shift were examined in detail as presented in Appendix . No satisfactory explanation was found, however, and the ultimate decision was to adopt the Red River Basin Investigation curve for design purposes.

Backwater Computations

Water surface profiles, for the reach of the Red River between the Floodway inlet and outlet, were required in order to establish criteria for the design of the Floodway and Floodway inlet and outlet structures. Relationships were required which would permit determination of water surface

elevations both under natural conditions and with all flood control works in operation. Because of the varying Assiniboine River constributions and Floodway flows, backwater computations had to be utilized to develop the relationships from which the water surface profiles could be calculated for all combinations of flows. The methods used in the backwater studies were dictated by the available data and variations in different reaches of the river. For the reach of the river, between the Floodway outlet and Norwood Bridge, at the confluence of the Red and Assiniboine Rivers, gauge records were available and a direct method of computation was used utilizing the available rating curves. pstream of Norwood Bridge, the standard step and reach methods were used in order to take into account the backwater effects from the Assiniboine River and the greater p tion of overbank flow. The results of the backwater studies are presented in Appendix C and the reader is referred to this section for a fuller description of the derivation and use of the various backwater relationships.

Tables: Flows and water surface elevations have been computed and summarized in Tables 1, 2, and 3, Appendix A, for all floods and various Assiniboine River contributions, under both natural and controlled conditions. Tabulated results at the Floodway outlet and inlet are presented in graphical form on Plates 15 and 16, Appendix A so that for any particular flood condition elevations can be determined directly. The tabulated values are based on the following assumptions regarding operation of the flood control works:

1. That there would be an initial storage of 150,000 acre feet in the Shellmouth Reservoir.

2. It has been assumed that the Portage Diversion would be operated at all floods. However, when peaks are below flood stage in Winnipeg the Portage Diversion would not necessarily be in operation unless it were desirable to divert flood waters into Lake Manitoba for storage or to reduce the possibility of ice jams occurring along the Assiniboine River downstream of Portage la Prairie.
3. It has been assumed that the water surface elevation upstream of the structure would be determined on the basis of natural conditions and average Assiniboine River contribution. Thus, the resulting Floodway discharges are based on average conditions regardless of the relative magnitude of the Assiniboine River contribution. In actual operation it is expected, however, that the inlet elevations would be computed on the basis of the actual Assiniboine River flows.

Accuracy: The tabulated flows and water levels are presented to the closest one hundred c.f.s. and one hundredth of a foot respectively in order that a certain uniformity would result from the use of the various relationships. Actual conditions cannot be predicted with this accuracy, however. Within the range of recorded values, the elevations obtained by backwater studies check within a few tenths of a foot with the 1950 recorded values. For any particular flow condition, above the 1950 Flood level, the derived water levels would probably be within plus or minus one-half foot of the actual level.

THE RED RIVER FLOODWAY

Following selection and approval of the route for the Red River Floodway, design work was undertaken on the Floodway by the Planning Division of the Water Control and Conservation Branch of the Province of Manitoba and continued under the Floodway Division, after its formation early in 1961. On May 28, 1962, agreement was reached between the Governments of Manitoba and Canada for sharing of the construction costs of the Floodway.

Location and Brief Description

The final location of the Red River Floodway as shown on Plate 2, Appendix A was selected by the Water Control and Conservation Branch. Details of the location studies are contained in a separate report. In general, modifications to the route, on which the Commission's recommendation was made, were based on the hydraulic requirements of the Floodway and economic consideration of excavation, right-of-way, structures and crossings of the Floodway by public utilities. In accordance with the Commission's recommendation that St. Norbert should be included within the protected area, if feasible, the Floodway inlet was relocated from the site selected by the Red River Basin Investigation, upstream to River Lots 182 to 185, Parish of St. Norbert, in the City of St. Vital. The configuration of the Red River at this location provided a suitable offtake from the River and also a suitable location, approximately one-half mile downstream, for the control structure and adjacent dykes.

The Red River Floodway is an excavated earth channel 29 miles in length, which will carry flood waters from the Red River, around and to the

east of Greater Winnipeg, and back into the Red River at Lockport. As shown on Plate 2, Appendix A, the Floodway: leaves the Red River approximately two miles upstream of St. Norbert; passes to the east of the Metropolitan Winnipeg Bypass and the City of Transcona; cuts through the gravel ridge near Birds Hill; and parallels P.T.H. No. 59 North, re-entering the Red River one-half mile below the St. Andrews Dam of Lockport.

Diversion of flows into the Floodway will be regulated by a control structure located across the Red River approximately one-half mile downstream of the Floodway inlet. The structure, utilizing two 112.5 foot long submersible gates, will be capable of maintaining the upstream water surface elevations at those which would obtain under natural conditions. In order to contain the flood waters upstream of the Greater Winnipeg Area, dykes are required adjacent to the control structure. On the east side of the Red River, the dyke is incorporated into the disposal area and extends, from the control structure, parallel to the west bank of the Floodway for a distance of six miles. On the west side of the Red River, the dyke ties into high ground approximately 20 miles south and west of the control structure as shown on Plate 1, Appendix A.

Along the 29 mile length of the Floodway, the water surface elevation will drop a total of 18.03 feet, from elevation 770.25 to elevation 752.22, under design flood conditions. The corresponding water surface elevation of the Red River, at the outlet, will be 737.96. There will, therefore, be a drop in water surface elevation of 14.26 feet and a drop structure is utilized to reduce the velocities and to provide an outlet alighment which will not induce scour in the river downstream.

The Floodway is crossed by many of the public utilities servicing Winnipeg. In addition to seven railway and six highway bridges, provisions have been made for transmission lines, communication lines, two branches of the Greater Winnipeg Water District aqueduct and an oil and a gas pipeline.

Numerous drainage ditches, which follow the general slope of land, from the east towards the Red River, are intercepted by the Floodway. These drains and local runoff are gathered in a collection drain on the east boundary of the Floodway which discharges into the Floodway at suitable locations through drop structures. A drop structure is also required to carry Seine River flows into the Floodway.

Design Flood

Based primarily on engineering studies by the Red River Basin Investigation, the Royal Commission on Flood Cost-Benefit made an economic study, analyzing the merits of the various flood control projects singly and in combination. The Commission selected and recommended a 60,000 c.f.s. capacity floodway, the 25,000 c.f.s. capacity Portage Diversion and the Russell Reservoir and noted that "these projects will ensure almost complete protection to all parts of Greater Winnipeg behind the main dyking system from all floods of 169,000 c.f.s. or less" leaving approximately one foot of freeboard on the existing city dykes. The flow of 169,000 c.f.s., measured below the confluence of the Red and Assiniboine Rivers, was therefore adopted as the project design flood.

Design Capacity

A design capacity of 60,000 c.f.s., was selected for the Red River Floodway, in accordance with the Commission's recommendation. Under design flood conditions, and assuming: average Assiniboine River contribution; an initial storage of 150,000 acre feet in the Shellmouth Reservoir; and, 25,000 c.f.s. discharge in the Portage Diversion, it was verified that one foot of freeboard would result on the Greater Winnipeg dyking system.

Design Criteria

Prior to design of the Floodway channel, criteria, governing various factors which would affect the design, were established from previous investigations, available literature, and additional studies.

Water Surface Elevation at the Inlet: One of the basic requirements of the design was that the water surface elevations upstream of the control structure should not exceed those which would obtain under natural conditions. For the project design flood of 169,000 c.f.s., the calculated water surface elevation of the Red River at the Floodway inlet would be 770.25 at which stage the Floodway would require a capacity of 60,000 c.f.s.

Roughness Coefficient: Manning's formula for uniform flow in open channels of

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

was adopted for use in the Floodway design and a roughness coefficient of $n = 0.028$ was selected. In part, the selection was governed by the studies made by the Red River Basin Investigation primarily on data published on reaches of the Panama Canal which, having values of hydraulic radius of approximately 20, correspond in size to the Floodway channel.

An "n" value of 0.028 may appear to be high for a large, uniform, relatively straight earth channel with grassed slopes. The choice, however, was deliberate. Selection of a high "n" value ensures that the Floodway capacity, which is the critical aspect of the design, will at least meet the design requirements at the expense of increasing velocities slightly. For example, if the actual "n" value were 0.025 the Floodway would carry 67,000 c.f.s. rather than 60,000 c.f.s. resulting in a reduction in stage at Redwood Bridge of 1.4 feet but the average velocity of 5.0 f.p.s. would be increased to 5.6 f.p.s. The choice of too low an "n" value, on the other hand, would result in a channel which would not meet design requirements.

Velocity: A limiting velocity of 5.0 f.p.s. was selected for design flow conditions in order to ensure that excessive erosion would not occur. Based on published data 5.0 f.p.s. is the maximum permissible velocity for aged canals excavated in colloidal silts or clays and carrying water which contains colloidal silts. Although the Floodway will not become aged like a channel in continuous use, the large size allows an increase in permissible velocity. Subsequent checks on the tractive force of various Floodway designs indicated that even higher velocities, although not practical, could be allowed. These studies indicated that fines may be washed from the non-cemented and exposed cemented hardpan materials encountered in portions of the channel. The erosion would be limited, however, by the large percentage of coarse material present and the relatively short duration of flood flows.

For most discharges velocities will exceed nonsilting velocities but under low flow conditions, velocities as low as 1.0 f.p.s. will occur. Since the Floodway inlet is approximately 30 feet above the bottom of the Red River the sediment load in the Floodway will be finely suspended sediment which settles very slowly. Although maintenance of the channel will be required, sedimentation should be no problem.

Side Slopes: The Floodway channel is excavated through silts and highly plastic clays with the exception of the reach through the Birds Hill gravel ridge which is composed of gravels and outwash materials and areas of cemented glacial till material between Birds Hill and Lockport as shown on Plate 3, Appendix A.

The side slopes to be used were dictated by the requirements that the slopes should be generally stable in order to reduce maintenance costs. Dr. A. Casagrande, Soils Consultant for the Floodway, recommended 6:1 side slopes throughout the length of the Floodway with the exception of Birds Hill where 2.5:1 slopes could be used. The latter slope was reduced to 3:1 for practical considerations. In addition, side slopes of 9:1 were recommended at bridge sites to ensure bank stability at these locations.

Bends: The Floodway channel has eight changes in direction. Application of rule of thumb criteria in use by various organizations would result in the selection of a degree of curvative of 1.5° to 15°. A 3° curve with a radius of 1910 feet was adopted as a practical degree of curvature for all curves considering the relatively low velocity of the water in the Floodway. Hydraulic losses in the bends will be minor and were considered as being included in the roughness coefficient selected for the channel design.

Transitions: As shown on the profile of the Floodway on Plate 3, Appendix A, there are three changes in the Floodway base width. In addition, the base width was decreased at each of the bridge sites to compensate for the enlarged cross section resulting from the change in side slopes from 6:1 to 9:1. Each of these changes necessitated some form of transition. Although a relatively abrupt change could have been used considering the low velocities

in the Floodway, the transitions were formed of circular curves so that a smooth transition between base widths, surface widths and water surface profile would result. The transitions in each case were made long enough so that the change in angle between tangents did not exceed 12.5°. Hydraulic losses in the transitions were computed as 0.2 times the change in velocity heads when the cross-section area expanded and 0.1 times the change in velocity heads when the area contracted. Typical transition sections are shown on Plate 5, Appendix A.

Channel Design

Design of the Red River Floodway channel involved the selection of the most economical channel which would meet the hydraulic requirements and the established design criteria. In this regard, consideration was given to the costs of right-of-way and structures as well as to excavation costs. Since the channel excavation would involve approximately 100 million cubic yards, amounting to almost half of the project cost, the unit prices used in the estimates had to be selected carefully. The Red River Basin Investigation and Royal Commission of Flood Cost-Benefit selected a price of 30 cents per cubic yard for common excavation and one dollar per cubic yard for the hardpan or cemented glacial till material. These prices were adopted for the Floodway estimates. Costs for structures were based on estimates by the agencies involved in the design.

Preliminary Designs: The ground elevation along the Floodway, shown on Plate 3, Appendix A, rises gradually from the inlet to the vicinity of the Trans Canada Highway, remains more or less constant to the Birds Hill ridge beyond which it drops and remains relatively constant to the outlet. North of Birds Hill, the cemented glacial till material rises nearer

to the surface resulting in more costly excavation in this reach. These features indicated that the most economical channel would have the following features:

1. A wide base width and 6:1 side slopes between the inlet and Birds Hill in order to reduce the hydraulic losses and more costly excavation north of Birds Hill;
2. A section through Birds Hill utilizing the recommended 3:1 side slopes; and
3. A uniform channel north of Birds Hill with 6:1 side slopes.

Approximately 50 designs were prepared, with base widths varying from 250 to 500 feet and with varying depth of flows and velocities not exceeding five feet per second. The designs were based on approximate backwater calculations and for each design approximate earthwork costs were calculated. These approximate calculations indicated the economical depth of flow for each design and also which designs were obviously uneconomical.

Computer Studies: The I.B.M. 1630 data processing system was used to compute more accurately the standard step backwater and earthwork calculations for ten channel designs selected from the studies outlined above. Backwater computations were based on sections 1000 feet apart and earthwork calculations on sections 500 feet apart. The variation between estimated costs of the ten designs was approximately only two percent of the total cost. Two channel designs, selected on the basis of the cost estimates, were presented to the Floodway Advisory Board, which recommended selection of the wider, shallower channel in view of the probable reduced costs and difficulties which would result during construction and maintenance.

The selected trapezoidal channel was modified as shown on the

cross-sections on Plate 4, Appendix A so that positive drainage of the Floodway base would be ensured. A channel with a capacity of 100 c.f.s. was designed along the Floodway centre-line and a two percent transverse slope introduced between this low flow channel and the toe of the main slope. The purpose of these features is to prevent low flows of the Seine River and outside drains from meandering across the main channel base. In addition, drainage and silting will be localized permitting easier maintenance of the Floodway channel. The capacity of the low flow channel was selected primarily on the basis of the summer flow duration of the Seine River which has flows of less than 100 c.f.s. 70 percent of the time, for the months of June to October inclusive. Higher flows or coincidence of peaks on outside drains will be contained on the two percent transverse slope, draining back into the low flow channel when the peaks subside.

Standard step backwater computations done on the computer at 100 foot sections for design flow conditions were utilized in selecting the final channel grades shown on Plate 3, Appendix A. Backwater computations were then made at various discharges to obtain the rating curve at the inlet shown on Plate 8, Appendix A, and the water surface profiles shown on Plate 3, Appendix A. These data are approximate insofar as designs and locations of bridge crossings had not been finalized. Backwater effects from the bridges were included in the backwater computations, however, and variations between actual and assumed conditions will not affect results to any significant extent. Between the inlet and the Seine River, flows will be above ground level for the higher Floodway discharges. In the backwater computations, a roughness coefficient of 0.030 was used for the flows on the 150 foot berm section between the Floodway channel and the disposal areas.

<u>Bridges</u>: The side slopes of the channel were reduced from 6:1 to 9:1 at bridge locations as recommended to ensure bank stability at the

structures. In order to reduce the bridge lengths and costs, the base width of the channel was reduced at each bridge location. For the four bridges north of Birds Hill, the base width could be decreased economically until the average velocity at the section reached the maximum permissible velocity of 5 f.p.s. at design flood stage. Although there would be a slight difference in backwater effect between highway bridges and railway bridges due to variation in span length and pier thickness, a standard base width of 330 feet was selected for the four bridges. Backwater effects were calculated for various channel base widths at the bridge locations south of Birds Hill. Costs of excavation and bridges were plotted for each base width so that the most economical base width could be selected graphically. A standard base width of 390 feet was selected for each of the nine bridges.

In addition to selecting the channel cross-section at the bridges, the following criteria were established for the bridge designs:

1. The bridges should have a clearance between the underside of girders and the design flood level of not less than one foot.
2. The upstream and downstream ends of the piers should be streamlined.
3. The piers should be designed to withstand floating ice which could occur at stages up to five feet below design flood level.

Inlet: A flat sill at elevation 750.0 was established for the Floodway inlet which would allow normal summer flows and spring flows of up to 30,000 c.f.s. to pass down the Red River without entering the Floodway. On the average, Red River flows will enter the Floodway once in three years. Located on a tangent to the Red River, as shown on Plate 6, Appendix A, the inlet will permit diversion of the Red River flows with a minimum of

hydraulic loss. The shape of the inlet was designed with a 700 foot base width, gradually narrowing and deepening to the Floodway channel shape to maintain even velocities and smooth flow patterns. The arrangement of the inlet, in conjunction with the inlet control structure, disposal areas and dykes, was model tested on a movable bed model located at the University of Manitoba. Details of the model tests are contained in a separate report.

Inlet Control Structure.

If no control works were installed, flood flows on the Red River would split between the natural channel and the Floodway, resulting in a drawdown effect extending upstream from the point of diversion. At design flood, the inlet water level for a 60,000 c.f.s. floodway would be eight feet below the natural water surface elevation. A control structure across the Red River capable of maintaining natural conditions upstream was therefore planned to reduce the amount of excavation for the Floodway channel and to regulate the amount of water entering the diversion.

The Floodway Division established the following requirements to be met in the design of the control structure:

1. The control structure should be capable of maintaining the upstream water surface elevations at those which would obtain under natural conditions, for all flows up to the 1000 year flood of 270,000 c.f.s., considering the Shellmouth Reservoir, Portage Diversion and Red River Floodway in operation and for the combinations of flood flows on the Red and Assiniboine Rivers which may be expected to occur 90 percent of the time.
2. For floods greater than the design flood of 169,000 c.f.s., the control structure should be capable of restricting flows in the Red River downstream, so that the water levels at the Redwood

Bridge do not exceed elevation 755.5, under the condition of maximum Assiniboine River contribution, until the maximum permissible upstream water surface elevation of 778.03 is reached. The water surface elevation and discharges resulting from this type of operation are tabulated in Table 4, Plate 14, Appendix A.

3. The control structure should permit the passage of small boats during the navigation season, having a draught of up to six feet and an overhead clearance of less than 15 feet.

Design of the inlet control structure works was carried out by H. G. Acres and Company Limited. Details of the design are contained in separate reports. The structure is located across the Red River as shown on Plate 6, Appendix A, and consists of two submersible gates 112.5 feet long and 34.9 feet high supported by two abutments and a centre pier. The crest is at elevation 728.0, approximately six feet below the summer level controlled by St. Andrews dam at Lockport, to allow for the passage of summer flows and pleasure craft.

Operation of the control structure will be primarily to maintain natural conditions upstream of the Floodway inlet. Flows will begin entering the Floodway when the Red River discharge is approximately 30,000 c.f.s. For flows above this discharge, the gates will be raised to maintain the upstream water level at the elevation which would obtain under natural conditions. For the project design flood of 169,000 c.f.s. and with average Assiniboine River contribution, the upstream water level will be maintained at an elevation of 770.25. At this time 60,000 c.f.s. will be entering the Floodway and 70,700 c.f.s. will be passing through the control structure under a head of approximately eight feet, the remainder of the flood waters having been reduced by the Assiniboine River flood control projects.

Outlet Control Structure.

At design flood, water levels at the outlet of the Floodway will be approximately 14 feet above the Red River level and an outlet control structure is required to dissipate this potential energy so velocities and scour, below the confluence of the Floodway and Red River, will not exceed those which would occur under natural conditions.

The following design requirements of the outlet control structure were established by the Floodway Division:

1. The outlet control structure should effectively reduce velocities and erosion, downstream of the outlet, for all flows up to the Floodway design flow of 60,000 c.f.s., considering the Shellmouth Reservoir, Portage Diversion and Red River Floodway in operation and for the combination of flood flows in the Red and Assiniboine Rivers which may be expected to occur 90 percent of the time.
2. The water surface elevation in the Floodway, at the upstream end of the outlet transition, should be 752.2 at the design discharge of 60,000 c.f.s.
3. Conduits should be provided through the structure with a capacity of 100 c.f.s. when the water surface elevation, at the upstream end of the outlet transition, is 726.2.

Design of the outlet control structure and channel, from the Red River to the upstream end of the approach transition to the structure, was carried out by H. G. Acres and Company Limited. Details of the design are contained in separate reports. The structure has a 162 foot wide reinforced concrete ogee section with a crest elevation of 730.0, but a conventional stilling basin has not been used, advantage being made of

the natural bedrock formation at the site. Smooth entrance and exit conditions at the structure are ensured by a 485 foot long inlet transition and an exit channel flared to provide suitable re-entrance of Floodway flows to the Red River. A plan and cross-section of the outlet control works are shown on Plate 7, Appendix A.

Model Studies.

H. G. Acres and Company Limited used hydraulic models in the design of the Floodway inlet and outlet control works. The models were located at the University of Manitoba and at the H. G. Acres laboratory in Niagara Falls, Ontario.

Inlet Control Works Models: A movable bed model was constructed at the University of Manitoba to a horizontal scale of 1:120 and a vertical scale of 1:60. The model covered a reach of the Red River extending approximately 3000 feet upstream and 10,000 feet downstream of the Floodway inlet, plus approximately 3,000 feet of the Floodway channel. Tests were carried out under all flow conditions to evaluate, with respect to the design requirements: the overall performance of the inlet works; riprap requirements upstream and downstream of the structure; and to determine the most economical alignment of the channel required to divert river flows around the control structure site during construction. A more detailed discussion of the model and tests is contained in separate reports. The inlet to the Floodway, discussed in a previous section, was also tested on this model by the Floodway Division.

Further testing of the control structure and submersible gates was carried out on models located at Niagara Falls. A section of the gate and stilling basin, representing a length of 60 feet of gate

in the prototype, was tested on an undistorted model built to a scale of 1:60. In addition, a portion of the inlet control works, of one and one-quarter gate sections plus the Red River for a distance of 400 feet upstream and 500 feet downstream of the control structure, was tested on an undistorted model constructed at a scale 1:60. The models were used to evaluate the size and shape of gates, skin pressures on the gates, design data for the gates, the shape of the stilling bucket, the alignment of the wing walls, and riprap requirements at the structure. Details of the models and testing program are contained in a separate report.

Outlet Control Works Model: An undistorted model of the outlet control works was built, at the University of Manitoba, to a scale of 1:100, covering a reach of the Red River extending approximately 2000 feet upstream and 1200 feet downstream of the Floodway outlet plus 2000 feet of Floodway channel. Sections of the model were constructed of movable bed material where scour conditions were to be evaluated. In general, the model was used to determine the most economical design of the outlet works, having regard for the overall hydraulic requirements. More specifically, the following features of the design were evaluated: the size and shape of the rollway section; the stilling basin requirements; the alignment and shape of the wing walls and training walls; the alignment and shape of the approach and exit channels; the riprap requirements; and the scour action in the Red River downstream of the structure. Details of the model and testing program are contained in a separate report.

Dykes.

Flood waters of the Red River inundate a large area of land to the south of Winnipeg in the former bed of glacial Lake Agassiz.

Flooding to the south of the Floodway will occur to the same extent as under natural conditions when the proposed flood control works are in operation. Dykes are required to tie the control structure into high land on each side of the Red River, to contain the flood waters upstream of the control works.

Hydraulic studies involved selection of the degree of protection to be provided by the dykes and the elevation to which the dykes should be built to provide the required protection. Normally a certain amount of freeboard is allowed above a fixed or design water level but water surface elevations upstream of the Floodway dykes will vary for each flood. Therefore, the frequency with which a certain dyke elevation would be exceeded was determined and the dyke elevation selected which would provide the required degree of protection. Freeboard, consisting of wind set-up above still water level plus wave uprush, is a function of the water depth and area; wind velocity, direction, duration and fetch; and the side slopes and materials of the dyke. For any water surface elevation and wind velocity the freeboard can be calculated. The computations were based on studies of waves on inland reservoirs by the United States Army Corps of Engineers. Frequency curves of water surface elevations at several points along the dyke location were combined with a frequency curve of wind velocity assuming that there would be no correlation between wind velocity and flood magnitude. The frequency curve for wind velocity was prepared for the maximum winds of one hour duration recorded during April and May for a 45 year period. The highest velocity within a period of a month was used because the duration of major floods would last between three and four weeks. The wind velocity of one hour duration was adjusted for the length

of duration required to develop set-up and wave action and increased to allow for the decreased resistance to winds travelling over water surfaces. The freeboard requirements were then calculated for the probable combinations of wind velocity and water level and the resulting dyke elevation plotted against the corresponding frequency. The highest water surface elevation for which the freeboard was calculated was for the 1000 year flood of 270,000 c.f.s., for which the average wind velocity of 35 miles per hour for a one hour duration was assumed. The resulting dyke elevation was plotted against a frequency of 0.1 percent.

Waves overtopping the dyke in the vicinity of the control structure could result in failure of the dyke and the release of sufficient water to suddenly overtop the city dykes downstream in addition to endangering the control structure. The degree of safety required for the dykes is, therefore, the same as for the control structure which was designed to withstand the 1000 year flood. The corresponding top of dyke elevation of 781.0 was selected for the portions of the dyke adjacent to the control structure, providing a freeboard of 10.75 feet above the design flood level.

West of the Red River: The most extensive dyking is on the west side of the Red River, where approximately 20 miles of dyke are required between the control structure and high land, as shown on Plate 1, Appendix A. The most economical location was selected with consideration given to severance of property and disruption of existing drainage works. The freeboard and degree of safety is gradually decreased from the control structure towards the west end of the dyke as the water depths decrease and the consequences of a dyke failure become less severe. Since the dyke travels in a

southerly direction to avoid the Rivière Sale the water surface elevations along the dyke increase. The top of dyke elevation is reduced, therefore, to a minimum of 779.00 at the west end with a probability of exceedance of 0.3 percent.

East of the Red River: On the east side of the Red River the dyke extends approximately six miles from the control structure along the north west side of the Floodway. Adjacent to the structure, the dyke has an elevation of 781.0, but along the Floodway, where protection is provided by the disposal areas, the elevation corresponds to the water surface elevation of the design flood of 60,000 c.f.s. The dyke section is incorporated into the disposal area used for the Floodway excavation as shown on Plate 4, Appendix A. Flood waters will extend along the upstream side of the Floodway disposal area, which is constructed to an elevation of 781.0.

Drainage.

The Red River Floodway crosses both man-made and natural drainage systems, notably the Seine River, which, in general, flow westerly from the higher land in the east towards the Red River. With the exception of the Seine River, flows in the existing drains are collected in a drain paralleling the Floodway and located outside of the east waste embankment, reducing the number of drainage outlets into the Floodway. Each of the outlets requires a drop structure to dissipate the energy resulting from the difference in elevation between the drain and Floodway bottom. Six outlet locations were selected, besides the Seine River outlet, providing the most economical arrangement of drains and outlets, based on consideration of costs of excavation, crossings and outlet structures. In addition to

the main drain along the east embankments, drains affecting local areas outlet to the Red River, Seine River, and to existing drains on the west side of the Floodway.

Design Criteria: Particular attention was given to the design requirements of the drainage systems in view of possible future development of property adjacent to the Floodway. Design criteria were selected which would provide adequate drainage of Floodway property and which would allow for future development of existing drains beyond the requirements presently accepted for drainage of land used solely for agricultural purposes.

1. Design Discharges. The simplest and most commonly used drainage equations are empirical formulae relating design discharge with the allowable frequency of occurrance and drainage area. Discrepencies between the available formulae exist and erroneous discharges result when they are used for small drainage areas where local effects have a large influence on the discharge and where individual treatment is required. A drainage equation was, therefore, developed for use in the Floodway designs. Drainage areas were corelated to discharge using frequency curves prepared by the P.F.R.A. for rivers with small drainage areas located on the east side of the Red River. The equation is given by:

$$Q = 7.40C\ A^{.71}\ T^{.444}$$

Where Q = the peak flood in c.f.s. that may be expected to be equalled or exceeded, on the average, in a period of T years.

A = the effective drainage area in square miles.

T = the frequency period in years.

C = a run-off coefficient which is dependent on the watershed characteristics taken as equal to 1.0 for the area east of the Red River.

The equation is shown graphically on Plate 17, Appendix A.

The following discharge frequencies were selected for design of the drainage works:

Drains - 50 year flood.

Culvert installations - 75 year flood.

Drop structures - 100 year flood.

2. Drain Cross-Section. Design of the drainage channel was based on: Mannings equation; a roughness coefficient of 0.030; a velocity between 1.0 and 3.0 feet per second; and a minimum channel slope of 0.02 percent.

For ease of construction and maintenance, a trapezoidal section was selected with a minimum base width of eight feet and side slopes which vary depending on the depth of cut as follows:

Average Depth of Cut	Side Slopes
Below 4 feet	3:1
4 to 6 feet	4:1
Deeper than 6 feet	5:1

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DEPARTMENT OF MINES, RESOURCES AND ENVIRONMENTAL MANAGEMENT

WATER RESOURCES BRANCH

APPENDICES

to Accompany

THE RED RIVER FLOODWAY

HYDRAULIC STUDIES

February, 1970.
Winnipeg, Manitoba.

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APPENDIX A

to

THE RED RIVER FLOODWAY

HYDRAULIC STUDIES

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THE RATING CURVE AT REDWOOD BRIDGE

Redwood Bridge, located across the Red River approximately three and one-half miles downstream of the confluence with the Assiniboine River, was selected as a gauging station in August, 1912. Daily gauge readings have been taken since that time. By combining discharge measurements taken in 1922 and during the 1950 flood, a rating curve was constructed for this location applicable to open water conditions with the St. Andrews Dam at Lockport open. The rating curve was extended above the highest metered discharge of 103,600 c.f.s. by the Red River Basin Investigation which extrapolated the curve through the points representing the reconstructed floods of 1826, 1852 and 1861.

Discharge measurements made at the section, subsequent to 1950, indicated a progressive shift of the rating curve to the right reaching a maximum in 1956 and remaining relatively constant since then. The extent of the shift represents an increase in the carrying capacity of the Red River of approximately 5000 c.f.s. between 1950 and 1956.

Before any hydraulic designs could be carried out, it was necessary to establish whether the increased capacity of the Red River was a permanent effect which should be considered in the hydraulic studies and designs. The more obvious causes for the shift such as a change in datum or method of measurement were investigated and found not to be significant. The only remaining causes to be investigated were an increase in channel cross-section or a decrease in channel roughness.

Gauge Relationships:

Numerous gauges were established through Winnipeg and along the Red River to Lake Winnipeg during the 1950 flood. Daily readings were also taken at several of these locations durign the 1956 spring runoff making it

possible to construct gauge relationships for the two periods using Redwood Bridge as base. The locations of the gauges are shown on Plate 2, Appendix A and the gauge relationships on Plate 16, Appendix B. The gauge relationships were used to determine the extent of the lowering in stage at various locations as shown in Table 1.

TABLE 1 - REDUCTION IN STAGES BETWEEN 1950 AND 1956

Gauge Location	Red River Discharge		
	50,000 c.f.s.	60,000 c.f.s.	70,000 c.f.s.
Gauge 8	1.15	1.06	0.95
Gauge 7	+0.22	.13	.67
Gauge 3			0.13
North City Limits		0.64	0.66
Redwood Bridge	0.97	1.08	1.10
Louise Bridge		1.09	1.10
C.P.R. Bridge		0.80	0.82
James Avenue	0.78	0.92	0.96
Provencher Bridge		0.56	0.63
Norwood Bridge		0.60	0.68
Elm Park Bridge	0.65		
Fort Garry	0.90		
R.R.B.I. Inlet		1.60	1.60
Ste. Agathe	0.0	6.0	0.0

During the 1956 flood, few water levels were obtained north of the city limits except at gauges 7 and 8 at Lockport where daily readings were obtained by the Federal Department of Public Works. It should be noted that gauge 8 is affected by the level of Lake Winnipeg. However, during the period used in preparing the gauge relationships the lake level was essentially the same in 1956 as in 1950 except for daily variations so there is a lowering in stage at gauge 8 of approximately one foot. There are discrepancies in the effect at gauge 7 due to the shape of the 1950 gauge relationship, although St. Andrews Dam was open in both 1956 and 1950. Two gauge readings were obtained by the P.F.R.A. on May 18, 1956, near the head of Lister's Rapids. One gauge was located slightly upstream of gauge 3 and the other between gauge 3 and gauge 4. The water levels obtained were adjusted to correspond to gauge 3 and 4 locations where a lowering of 0.48 and zero feet was indicated between 1950 and 1956 at the respective gauges. At the corresponding discharge, Redwood Bridge readings are lowered by 0.75 feet. One gauge reading was obtained at gauge 3 at the peak of the 1956 flood on April 27. This gauge was not zeroed, however, until August 3, 1961, when it was found in good condition and according to local residents had not moved except for a possible settling during the past two years. The elevation indicated a lowering between 1950 and 1956 of 0.13 feet. In order to establish gauge relationships at locations upstream of the Norwood Bridge, the backwater caused by the Assiniboine River had to be eliminated. The results shown in Table 1 for these locations were therefore derived by computation rather than measurement but indicate a lowering in stage for the reach of river between the mouth of the Assiniboine River and the Floodway inlet.

Red River Cross-Sections:

Following the 1950 flood, an extensive hydrographic survey of the Red River was made by the Red River Basin Investigation and the Greater Winnipeg Dyking Board. Within the city, cross-sections were taken at 200 foot intervals and at 800 foot intervals in the other reaches. These cross-sections were compared to surveys made in 1912 and 1886. The Red River Basin Investigation noted only minor changes in section and also that there was no evidence of deposition at the St. Andrews Dam at Lockport.

Numerous bank failures have occurred since 1950 and when the shift in the rating curve at Redwood Bridge was noted, were considered to be a contributing factor. The P.F.R.A. took a series of soundings at each of the following locations by echo sounder: Elm Park Bridge; Redwood Bridge; Polson Avenue; Jefferson Avenue; Newton Avenue, and Bergen Cut-off. A comparison of these soundings to corresponding Red River Basin Investigation cross-sections indicated little net change in section below the stage corresponding to a discharge of approximately 30,000 cfs. Cross-sections taken during topographic surveys at the Floodway inlet and outlet sites were also compared to the 1950 cross-sections and no appreciable change in area was noted. These investigations confirmed the conclusion reached by the Red River Basin Investigation that the small local changes have a tendency to counteract each other and further that the cross-sections taken in 1951 would still be accurate enough for use in hydraulic investigations. Although the local bank failures would have minor local effects on stage, there was no indication of sufficient channel enlargement to cause the various changes in stage noted in the previous section.

Roughness Coefficient:

Having determined that erosion and changes in channel cross-

section were relatively insignificant and were not responsible for the shift in the rating curve between 1950 and 1956, the only remaining explanation for the shift, assuming that other factors such as sediment load and turbulence would have had a minor effect on the carrying capacity of the river, was that a change in roughness coefficient had occurred. The Red River Basin Investigation calculated average "n" values, applicable to Manning's equation, for the Red River between Selkirk and the University of Manitoba for the 1950 peak discharge. These values were checked by computations through the City of Winnipeg by the Provincial Water Resources Branch during studies on channel improvements. In order to provide a comparison between 1950 and 1956 conditions, "n" values were computed for various reaches for the 1956 peak discharge and for the corresponding stage under 1950 conditions. The results of these studies are presented in Table 2.

TABLE 2 - ROUGHNESS COEFFICIENTS FOR THE RED RIVER

Reach of Red River	1950 Conditions			1956 Conditions
	Peak Discharge		At Stage Corresponding to Max.1956 Stage	At Peak Discharge
	RRBI	WC&R		
At Floodway Outlet	0.0320	0.0339	0.0356	
Lockport to Bergen Cut-off	0.0330	0.0340	0.0326	
Bergen Cut-off to Redwood Bridge		0.0301	0.0241	0.0206
Redwood Bridge to Univ. of Man.		0.0318	0.0292	0.0275
Bergen Cut-off to Univ. of Man.	0.034	0.0316	0.0284	0.0265

The "n" values were computed using: 1951 cross-sections; average slopes between recorded water levels, excluding losses at bridges; and, the hydraulic radius equal to the hydraulic mean depth. The values listed above are weighted means of the values computed for shorter reaches. Discrepancies between the Red River Basin Investigation and Water Control and Conservation Branch values could result from the methods of computation used; however, the calculations by comparable methods and for corresponding reaches of the river indicated that the "n" values decreased both with a decrease in river stage and between 1950 and 1956. These average "n" values were verified by use in backwater computations which were compared to the recorded river stages.

As indicated above, the shift in the rating curves can be explained and verified by assuming that the roughness changed between 1950 and 1956. There is, however, no clear explanation for the change. It has been generally assumed that under flood conditions with corresponding high velocities that there would be a general smoothing of the channel with removal of brush and small obstructions. If this were true some change would have been indicated during the rising and falling stage of the 1950 flood. However, discharge measurements, during the falling stage in 1950, plot on the relationship and there was no change in slope between rising and falling stage. In addition, discharge measurements taken in 1951 indicate only a minor change; in 1952 and 1955 a greater change; and in 1956 the maximum shift which was verified by a measurement in 1960. The shift in the rating curves has, therefore, been a gradual one.

During the studies roughness coefficients as low as-0.016 were computed for individual reaches. Although some discrepencies could result from local disturbances at the gauges, several values, particularly through the city, were as low as 0.020. These values are extremely low for a natural

alluvial river and no further decrease in roughness coefficient could be expected. The reverse, however, could occur since even a value of 0.030 is considered low for a natural channel. Bank slides, growth of brush during low flow periods and even man-made works could cause an increase in "n" value. Consideration of these factors resulted in selection of the 1950 rating curve as the basis for hydraulic designs and studies.

APPENDIX C

to

THE RED RIVER FLOODWAY

HYDRAULIC STUDIES

APPENDIX C

Plates

BACKWATER COMPUTATIONS

Water surface profiles, for the reach of the Red River between the Floodway inlet and outlet, were required in order to establish criteria for the design of the Floodway and Floodway inlet and outlet structures. Relationships were required which would permit determination of water surface elevations both under natural conditions and with all flood control works in operation. Because of the varying Assiniboine River contributions and Floodway flows, backwater computations had to be utilized to develop the relationships from which the water surface profiles could be calculated for all combinations of flows. The methods used in the backwater studies were dictated by the available data and varying channel characteristics for different reaches of the River. For the reach of the River, between the Floodway outlet and Norwood Bridge, at the confluence of the Red and Assiniboine Rivers, gauge records were available and a direct method of computation was used utilizing the rating curves. Upstream of Norwood Bridge the standard step and Leach method were utilized to develop rating curves which were in turn used for the direct or method of computation.

Norwood Bridge to the Floodway Outlet

Backwater studies, for both natural and controlled conditions, for the reach of River downstream from Norwood Bridge, were based on the 1950 rating curves and gauge records obtained under open water conditions with St. Andrews Dam open.

Natural Conditions: Backwater studies through the city indicated that the existing city dykes had a negligible effect on the water levels as far upstream as Elm Park Bridge. The 1950 and subsequent recorded data are therefore valid for natural conditions and also when considering the city dykes. No

backwater computations were required since there are no local inflows which would affect the recorded water levels.

Controlled Conditions: When the flood control works are in operation, the Floodway discharge at Lockport will create a backwater which will extend upstream through the City of Winnipeg reducing the discharge which can be carried safely between the city dykes. The purpose of the backwater computations was to provide relationships from which water surface profiles could be determined for various combinations of Red River and Floodway discharges. In order to reduce the time and effort required in the preparation of such relationships, a direct method of backwater computation was adopted which utilized the available 1950 gauge and discharge records.

Gauge relationships, based on the 1950 readings, were prepared between the gauge at Redwood Bridge and each of the individual gauges located downstream of the mouth of the Assiniboine River. Where gauges were located at bridges, the readings taken at the upstream side were used. The gauge relationships had to be extended beyond the recorded range and this was done by: separate gauge relationships; utilizing the flood profiles of 1826 and 1852 reconstructed by the Red River Basin Investigation; and, standard step backwater computations for some locations. The gauge locations are shown on Plate 2, Appendix A and gauge relationships on Plate 16, Appendix B. Slope-discharge relationships used in the direct method of backwater computations were prepared between the following gauges: Floodway outlet, 8, 7, 6, 5, 4, 3, Bergen Cut-off, North City Limits, Redwood Bridge and Norwood Bridge. The water surface elevations, under bakcwater conditions, can be determined directly using the above relationships in successive steps. For convenience, the slope-discharge relationships between the Floodway outlet and Redwood Bridge were summarized as shown on Plate 1, Appendix C verifying and extending

a similar relationship prepared by the Red River Basin Investigation. Although the elevation at Redwood Bridge indicates the adequacy of the dyking system, Plates 1 to 4, inclusive are included in Appendix C so that the water surface profile between Bergen Cut-off and Norwood Bridge can be calculated when the discharges in the Red River and Floodway are known.

Norwood Bridge to the Floodway Inlet

The primary purpose of the backwater studies for the reach of the Red River upstream of the mouth of the Assiniboine River, was to determine the water levels at the inlet for all flows under natural conditions and to determine the tailwater elevations which would obtain at the control structure site when the flood control works were in operation. Backwater computations were required due to the effect of the Assiniboine River and Riviere Sale flows under natural conditions and due to the additional effects from the Floodway flows and city dyking system when the flood control works are in operation. Three methods: standard step; direct; and, Leach methods of computing backwater profiles were utilized.

Natural Conditions: The Red River Basin Investigation prepared rating curves at the original inlet site, just downstream of St. Norbert, by backwater computations assuming zero and 20,000 c.f.s. flows in the Assiniboine River. These curves were verified by backwater computations and the rating curve for 40,000 c.f.s. flow in the Assiniboine River added as shown on Plate 10, Appendix B. An "n" value of 0.034, as calculated by the Red River Basin Investigation for use in Mannings' equation, was used for the river channel and an "n" value of 0.1 was considered representative of the overbank areas which consist of relatively heavy bush or built up areas. Although the "n" of 0.034 is slightly higher than the values shown in Table 3, it is more representative for conditions at design flood stage and no attempt was made to

vary the "n" at lower stages. In addition, the "n" values and method of calculation were checked by verifying the recorded 1950 water surface profile by backwater computations between Redwood Bridge and Ste. Agathe.

The backwater computations for flows below bank full stage were made by the standard step method and above bank full stage by Leach's method. In the latter case, the river was divided into reaches, average cross-sections computed, non-effective areas excluded and the slope for the overbank areas adjusted to channel slope. For all computations the hydraulic radius was taken as the mean hydraulic depth.

Since the computations of two separate investigations checked at the Red River Basin Investigation inlet and the backwater computations also checked reasonably well with the limited gauge records for the 1950 flood, the computations were extended upstream from the Red River Basin Investigation site to the Floodway inlet. Rating curves as shown on Plates 11, 12 and 13, Appendix B were then prepared at the following locations: the mouth of the Riviere Sale, at the Red River Basin Investigation cross-section S-47; the Floodway inlet, at cross-section S-54; and upstream of the inlet at cross-section S-64. Slope-discharge relationships as shown on Plates 9, 11 and 13, Appendix C were prepared between these locations by the direct backwater method so that water surface profiles could be computed through the reach of river at the inlet for all combinations of Red and Assiniboine River flows.

Slope-discharge relationships were also computed from the Floodway inlet to Ste. Agathe and Ste. Agathe to Morris so that backwater conditions could be determined upstream of the control structure site. Although neither of the rating curves at Ste. Agathe or Morris are well defined, the relationships shown on Plates 14 and 15, Appendix C, will give the relative backwater

effect at these locations.

Controlled Conditions: Since the water surface profiles would be affected by backwater from: the Floodway outlet; the Assiniboine River; and, the city dyking system, backwater studies were carried out to determine water surface profiles through the city and tailwater elevations at the inlet with the flood control works in operation. Standard step backwater computations were used to prepare rating curves at the following points which would provide a complete water surface profile for this reach of the river: Elm Park Bridge; Fort Garry at Red River Basin Investigation cross-section 32; the University of Manitoba at cross-section S-8; at the Red River Basin Investigation inlet at cross-section S-26; the mouth of the Riviere Sale at cross-section S-47; and the Floodway inlet at cross-section S-54. These rating curves are shown on Plates 7 to 12 inclusive, Appendix B.

Slope-discharge relationships, Plates 4, 5, 6, 7, 8, 10 and 12, Appendix C, were prepared from the above rating curves so that water surface profiles or elevations could be determined for all conditions up to the stage at which the city dykes would be overtopped. When the water surface elevation at Redwood Bridge is above 755.5, that is, the present elevation of the dykes of 752.5 plus three feet of possible, effective emergency dyking, the dykes would be overtopped and the relationships computed for natural conditions would apply.

APPLICATION OF THE BACKWATER RELATIONSHIPS TO DETERMINE WATER SURFACE ELEVATIONS

The following summary is presented in order to facilitate the use of the various figures for the computation of water surface elevations under the various, possible conditions.

Norwood Bridge to Floodway Outlet

Natural Condition: For all Red River flows, the water surface elevations can be obtained from the individual rating curves, Plates 1, 2, 3, 4 and 6, Appendix B and gauge relationships, Plate 16, Appendix B. The effect of the existing city dyking system on these rating curves is negligible and they can be used whether dyking is a consideration or not.

Controlled Conditions: The water surface elevations at Redwood Bridge can be determined for any combination of Red River and Floodway flows by using Plate 1, Appendix C. Elevations upstream and downstream of Redwood Bridge can be obtained using Plates 2, 3 and 4, Appendix C, providing a continuous water surface profile between Bergen Cut-off, at the north end of the city, and Norwood Bridge.

Norwood Bridge to Floodway Inlet

Natural Conditions: The water surface elevations at the Red River Investigation Inlet site can be determined for any combination of Red and Assiniboine River flows using Plate 10, Appendix B. Elevations upstream can be obtained using Plates 9, 11 and 13, Appendix C and adjusting for Riviere Sale flows, providing a continuous water surface profile from St. Norbert to upstream of the Floodway inlet.

Between Norwood Bridge and the Red River Basin Investigation Inlet site, the relationships shown on Plates 5, 6, 7 and 8, Appendix C, include existing and emergency dyking which has a negligible effect as far as

Elm Park Bridge but beyond which would result in higher water surface elevations than under natural conditions.

Controlled Conditions: When the water surface elevation, including the backwater effect from the Floodway, at Redwood Bridge, is below elevation 755.5, the elevations can be calculated by going upstream in successive steps using the slope-discharge relationships presented in Plates 4, 5, 6, 7, 8, 10, 12 and 13, Appendix C. These relationships include the effect of the existing and emergency dyking system.

When the water surface elevation at Redwood Bridge is 755.5 or greater, natural conditions apply. Tailwater elevations at the inlet can then be obtained by converting the backwater effect at Norwood Bridge to equivalent Assiniboine River flow and using the relationships discussed above for natural conditions.

APPENDIX D

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HYDRAULIC INVESTIGATIONS PERTAINING TO THE ASSINIBOINE RIVER

The effect that the control works, proposed for the Assiniboine River, would have on flood conditions at Winnipeg had to be taken into consideration in the design of the Red River Floodway. For any flood condition, the amount of Assiniboine River has to be known under natural conditions and then the effects of the Shellmouth Reservoir and the Portage Diversion computed. The effect of the Portage Diversion is readily available once the natural flow are known. Computation of the Shellmouth Reservoir effects are considerably more complicated.

The Shellmouth Reservoir

In addition to its flood control benefits, the Shellmouth Reservoir can also be used for conservation purposes. In order to derive the flood control benefits the reservoir storage was assumed to allocated as shown in Table 1.

TABLE I - SHELLMOUTH RESERVOIR - ASSUMED ALLOCATION OF STORAGE

Elevation	Storage Acre-feet	Allocation of Storage
Invert of conduits	0	Conservation
1388	150,000	Conservation and Flood Control
1404	340,000	Flood Control
1410 Spillway Crest Elev.	430,000	

A storage of 150,000 acre-feet would provide a dependable flow of 300 c.f.s. annually, which would be sufficient for present demands. Increased accuracy in flood forecasting methods and experience in operation of the reservoir could permit the conservation storage level to be increased in the

future. The optimum storage of 340,000 acre-feet could be maintained insofar as possible but it was assumed that the reservoir would be drawn down to a storage of 150,000 acre feet in the fall. The adequacy of the remaining storage in reducing spring floods was determined by routing studies carried out using an I.B.M. 1620 data processing system, and unit hydrographs and valley storage curves prepared by the Prairie Farm Rehabilitation Administration. Hypothetical floods were routed through the reservoir and down to Portage la Prairie where the reduction in the peaks was determined for various initial storage levels in the Shellmouth Reservoir. The relationship showing the peak reduction at Portage la Prairie, for various floods, with an initial storage of 150,000 acre-feet in the Shellmouth Reservoir is shown on Plate 6, Appendix D. This relationship was used for all Floodway studies because the Shellmouth Reservoir with an initial storage of 150,000 acre-feet, in combination with the Portage Diversion and 60,000 c.f.s. capacity Floodway, would reduce the design flood peak at Winnipeg, to one foot below the existing City of Winnipeg dyke levels.

The relationship on Plate 6, Appendix D also indicates the overall effectiveness of the Shellmouth Reservoir location. Although situated on the upper Assiniboine River, it controls the watershed which contributes the largest percentage of the flood waters. In addition, for the larger floods, the flood peaks originating upstream of Shellmouth become more coincident with the flood peaks on the other tributaries.

Correlation Between Peak Flows on the Red and Assiniboine Rivers

The Assiniboine River joins the Red River in Winnipeg just below Norwood Bridge and, on the average, contributes approximately 20 percent of the flood peak in the city. The amount of Assiniboine River contribution to Red River flows varies and is additionally complicated by the fact that at

flows approaching 20,000 c.f.s., the Assiniboine River overflows, in the reach between Portage la Prairie and Headingley, forming a large storage basin north and south of the River. A portion of the storage flows south via the Riviere Sale, joining the Red River at St. Norbert, approximately one mile below the Floodway inlet. The remaining portion of the overflow ultimately drains back to the Assiniboine River after the peak has passed. Although some time lag would be involved in reaching the Red River at St. Norbert, the overflow could contribute to the Red River peaks because of the comparatively flat flood hydrograph of the Red River. The Assiniboine River contribution to the Red River peak is therefore made up of the flow at Headingley plus the portion of overflow which reaches the Red River via the Riviere Sale.

The P.F.R.A. developed a relationship between Assiniboine River flows at Portage la Prairie and Headingley by using the available records and by routing flood flows through the reach; the difference between the flows at the two locations being the overflow which would go into storage. A relationship between the total overflow and the portion which would contribute to the Red River peak via the Riviere Sale was developed by the engineering consultants for the Royal Commission on Flood Cost-Benefit and in reproduced on Plate 5, Appendix D. Based on limited recorded data under low flow conditions, the extrapolated curve is largely hypothetical but has been accepted since possible inaccuracies would not affect the water levels determined using the relationship. The two relationships discussed above were combined to obtain the relationship between flows at Headingley and total Assiniboine River contribution to the Red River, shown on Plate 3, Appendix D.

The Assiniboine River will contribute a varying percentage of the peak flows on the Red River at Winnipeg depending on the degree of coincidence

and magnitudes of the floods in the respective drainage areas. Under natural conditions and for a particular flood, the water levels through the reach controlled by the Floodway would be relatively unaffected by the distribution of flows since the short backwater profile, from the junction of the Assiniboine and Red Rivers to the Floodway inlet, would correspond approximately to the natural profile obtained if all of the flow originated on the Red River. The effect on the water levels downstream of the Floodway inlet would be significant, however, when the Portage Diversion and Shellmouth Reservoir are considered in operation since, in general, these works become more effective with an increase in the size of the flood. A correlation between the peaks on the two rivers is required, therefore, in order to select the probable Assiniboine River contribution to any flood peak on the Red River.

Although correlations between the Red and Assiniboine Rivers peaks were developed during previous investigations, a new correlation was prepared which would be suitable for design purposes; take cognizance of the overflow contribution via the Riviere Sale; and would include the additional available records. The relationship between the peak flow on the Red River below the confluence with the Assiniboine River, referenced for convenience at Redwood Bridge, and the contribution from the Assiniboine River was determined by fitting a line, by the method of least squares, through the plotted positions of the 49 years of records available for the period 1913 - 1961. The relationship which is shown on Plate 2, Appendix D is based on the following assumptions:

1. The Red and Assiniboine River peaks are coincident.
2. The recorded peaks of 1950 are not representative of the sample.
3. The theoretical thousand year flood may be included in the sample.
4. A linear regression line would give a suitable fit.

During the 49 years of record, peaks on the Red and Assiniboine Rivers have been coincident eight times and the Assiniboine River peak at Headingley has occurred within one week of the Red River peak 50 percent of the time and within two weeks 75 percent of the time. Because of the flat flood hydrograph on the Red River the peaks can be considered as coincident.

The proportion of Assiniboine River contribution during the 1950 flood was considerably lower than for the other recorded floods. The 1950 peaks, although recorded events, were therefore discarded as being unrepresentative of the sample. The Red River Basin Investigation estimated the 1000 year flood at Winnipeg to be 270,000 c.f.s., of which 57,000 c.f.s. would be contributed by the Assiniboine River. These flows were introduced to the sample in order to provide a more accurate extrapolation of the relationship beyond the recorded events.

The correlation between the peaks is given by the equation of the linear regression line

$$Y = 2{,}300 + 0.213\ X$$

where: Y = The total Assiniboine River contribution to the Red River peak in c.f.s. i.e. the Assiniboine River flow at Headingley plus the portion of the overflow between Portage la Prairie and Headingley which would contribute to the peak flow of the Red River.

X = The peak discharge, in c.f.s., of the Red River below the confluence with the Assiniboine River.

A coefficient of correlation of one indicates a perfect correlation. The coefficient for the above regression line, adjusted for size of sample, of 0.91 indicates, therefore, that a good correlation exists between the peak flows or, more correctly, between conditions causing the peak flows. Although the correlation is good, the adjusted standard error

of estimate of 3,620 c.f.s. is relatively high.

The regression line represents an average condition only and the Assiniboine River contribution could vary considerably as indicated by the scatter of the points and the standard error of estimate. Although the variation would have a negligible effect on the natural water surface elevations for a particular flood, it would influence the water levels, through the reach of the river controlled by the Floodway, when the flood control works are in operation. A range, within which the Assiniboine River contribution would fall 90 percent of the time, was selected as the practical range to be considered for design purposes and the limits of this range have been defined as the maximum and minimum Assiniboine River contribution. By using a statistical approach it was possible to determine what the variation would be. There is a standard error of estimate in the ordinate and slope of the regression line from which it is possible to determine the range within which the true line would probably be 90 percent of the time. The range of Assiniboine River contribution is given by the value of the regression line plus or minus 1.69 times the adjusted standard error of estimate. Thus, for the design flood of 169,000 c.f.s. at Redwood Bridge, the Assiniboine River contribution would probably be between 29,100 c.f.s. and 47,500 c.f.s. 90 percent of the time.