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4.0 PROJECT DESCRIPTION

4.1 EXISTING FACILITIES

The Existing Floodway is a flood diversion channel that was constructed between 1962 and 1968 as a major element of a coordinated flood-defence response to massive damage incurred by the City of Winnipeg during the 1950 flood. After the 1950 flood, the Greater Winnipeg Dyking Board was established and began construction of a system of dykes to provide a measure of protection against flooding within the City of Winnipeg. This initial effort resulted in the construction of dykes within Greater Winnipeg along the Red, Assiniboine and Seine Rivers, as well as installation of pumping stations to lift runoff into the rivers and outside the dykes in low-lying areas. The dykes were constructed to an elevation 1.2 m4 (feet) below the peak 1950 water surface profile. A series of “borrow” sites were also established so material could be readily accessed to raise dykes in the event of future flooding emergencies. In 1956, the Province of Manitoba appointed the Royal Commission on Flood Cost Benefit to examine alternatives for providing structural flood protection to Winnipeg.

In 1958, the Royal Commission on Flood-Cost Benefit reported its recommendations based upon a major study to assess structural solutions to flooding problems on the Red and Assiniboine Rivers. As a result of this Commission’s recommendations, three Manitoba flood protection system projects were built within the period between 1962 and 1972:

1. Red River Floodway to divert 1700 cubic metres per second (m^3/s) (60,000 cubic feet per second [cfs])from the Red River south of Winnipeg to the east of the city, discharging this flow back into the Red River at Lockport (project completed in 1968);
2. a 700 m^3/s (25,000 cfs) diversion channel to convey floodwaters from the Assiniboine River immediately upstream of Portage la Prairie northward to Lake Manitoba (completed in 1970);
3. the Shellmouth Dam in the upper reaches of the Assiniboine River just north of Russell (completed in 1972) to store floodwaters and reduce flow peaks downstream by 200 m^3/s (7,000 cfs).

Together, these three flood control system components were designed to protect Winnipeg from floods with “natural” flows on the Red River downstream of the Assiniboine River up to 4,800 m^3/s 169,000 cfs. At the time of the design of these works, this level of protection corresponded to a 160-year flood flow. Today, once recent historical flow records are considered in the calculation of flood return period, the actual protection afforded by the existing design is estimated to be less than the original estimate at a return period of 90 years. The 1997 “Flood of the Century” had an estimated “natural” flood peak of 4,600 m^3/s (163,000 cfs), which represents a return period of approximately 100 years. Re-assessment of the floodway channel capacity (Section. 4.1.1.1) has indicated that 2,550 m^3/s (90,000 cfs) could be passed through the channel with some bridges submerged, although this is not considered reliable capacity. This allows flood protection for up to 5,900 m^3/s (208,000 cfs), which represents a 225 year return flood.

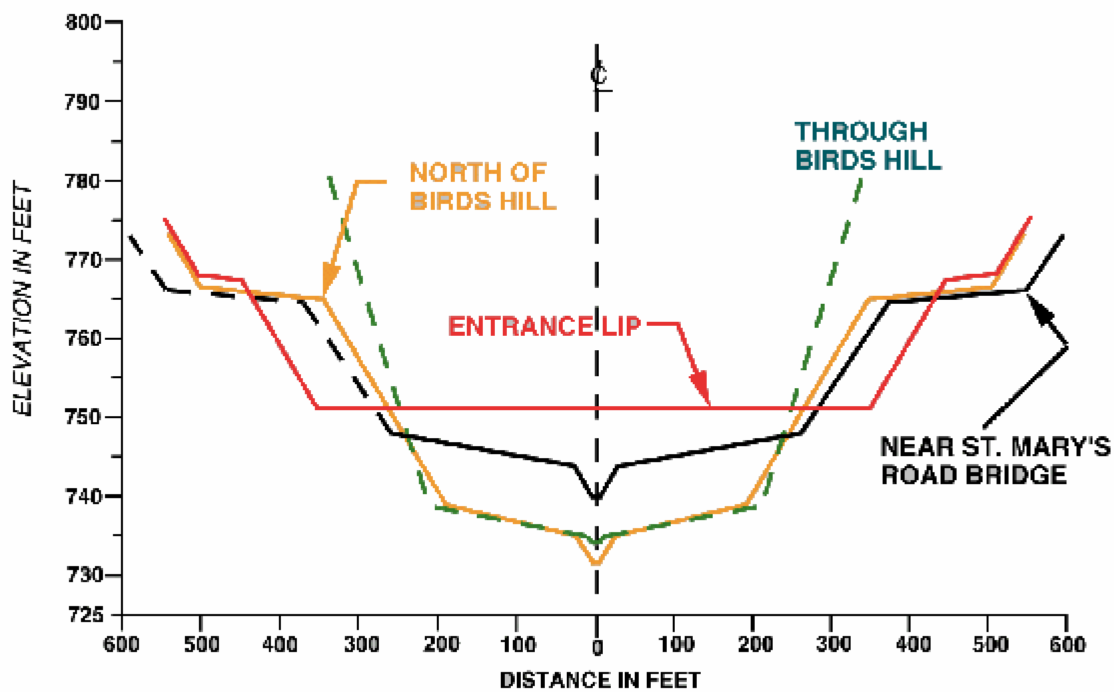
4.1.1 Existing Red River Floodway

The existing Red River Floodway comprises four components: the Floodway Channel, the inlet control system, the dykes, and the outlet structure. The Existing Floodway was built in the 1960's at a cost of $63 million dollars ($1968), completed in 1968, and first operated during a spring flood in 1969.

4.1.1.1 Existing Floodway Diversion Channel

The Existing Floodway Channel is about a 48 kilometre (29.5 mile) long, grass-lined diversion channel that conveys a portion of Red River flood flow around the City of Winnipeg and discharges this floodwater via the Floodway Outlet back into the Red River downstream of Lockport.

The channel is comprised of three distinct reaches. The average channel depth is 9.1 metres (30 feet) except through the Bird's Hill Ridge where the depth in this reach increases to 20.1 metres (66 feet). The three distinct reaches have different cross-sectional geometries, as shown in Figure 4.1-1.



Source: "Review of Red River Floodway Operating Rules", Red River Floodway Operation Review Committee, 1991.

Figure 4.1-1
Typical Cross Section for Red River Floodway

The upstream end of the Floodway Channel is equipped with an earthen lip with a crest 2.1 metres (7 feet) above the channel bottom. The earthen lip functions as an obstacle to keep river ice out of the

Floodway Channel, allowing river ice to break up and flow down the Red River through the City of Winnipeg before flows rise enough to begin entering the Floodway Channel. River ice is not desired in the Floodway Channel because it can jam against bridge crossings over the Floodway Channel, resulting in blockages that reduce the capacity of the Floodway Channel.

The existing channel has a longitudinal slope of 8.6 cm/**km** (0.5 feet per mile) upstream of Bird's Hill and a steeper slope of 16 cm/km (.8 feet/mile) downstream of Bird's Hill. To address **erosion** concerns within the channel, the maximum design water velocity is 1.5 m/s (5 feet per second).

The excavation of the existing channel involved moving 76,000,000 m^3 (100,000,000 cubic yards) of material. Most of the excavation was conducted in soil, however there are reaches of the channel alignment that required excavation in glacial till (hardpan).

A total of thirteen bridges cross the Existing Floodway Channel. Crossing locations are shown in Figure 4.1-2.

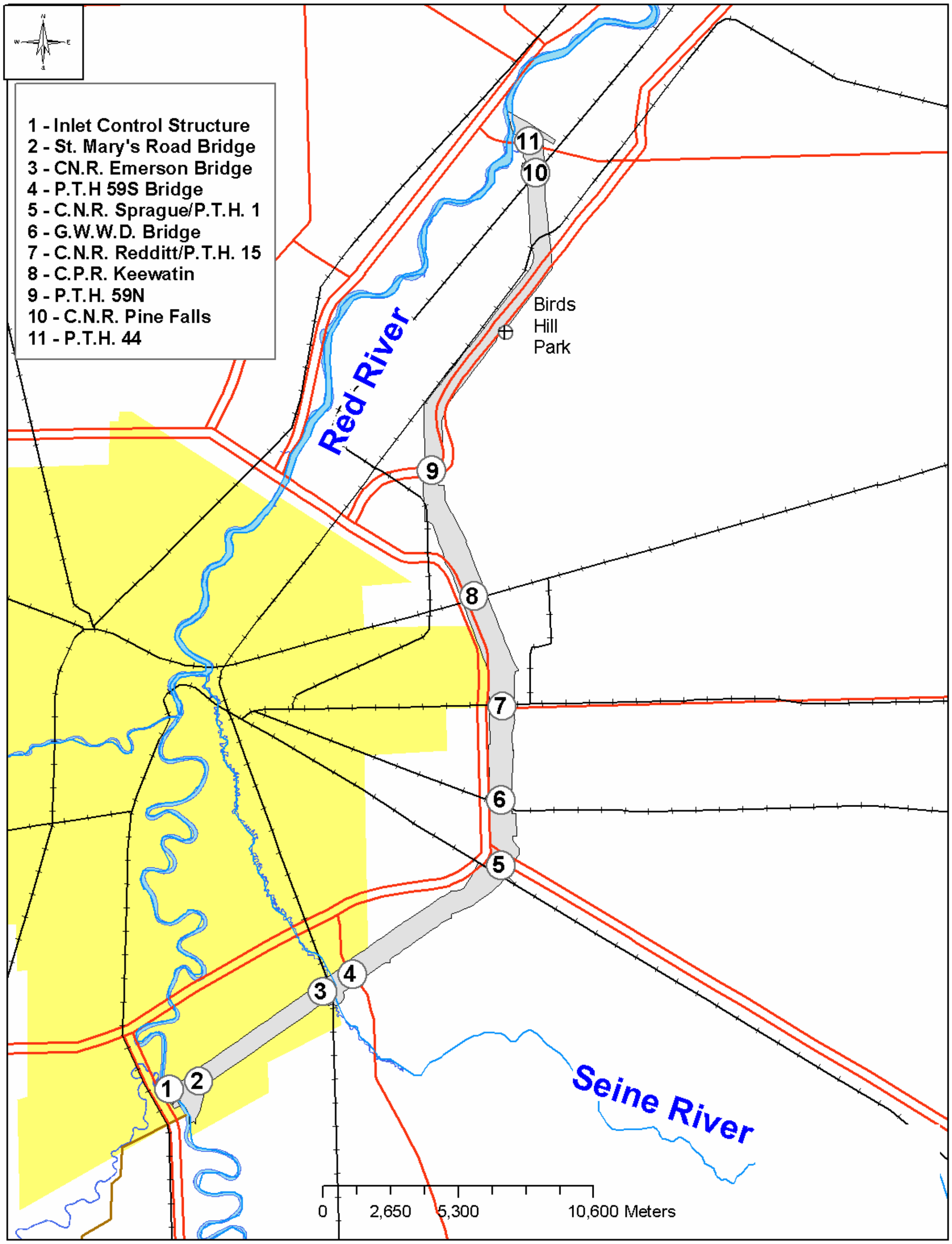
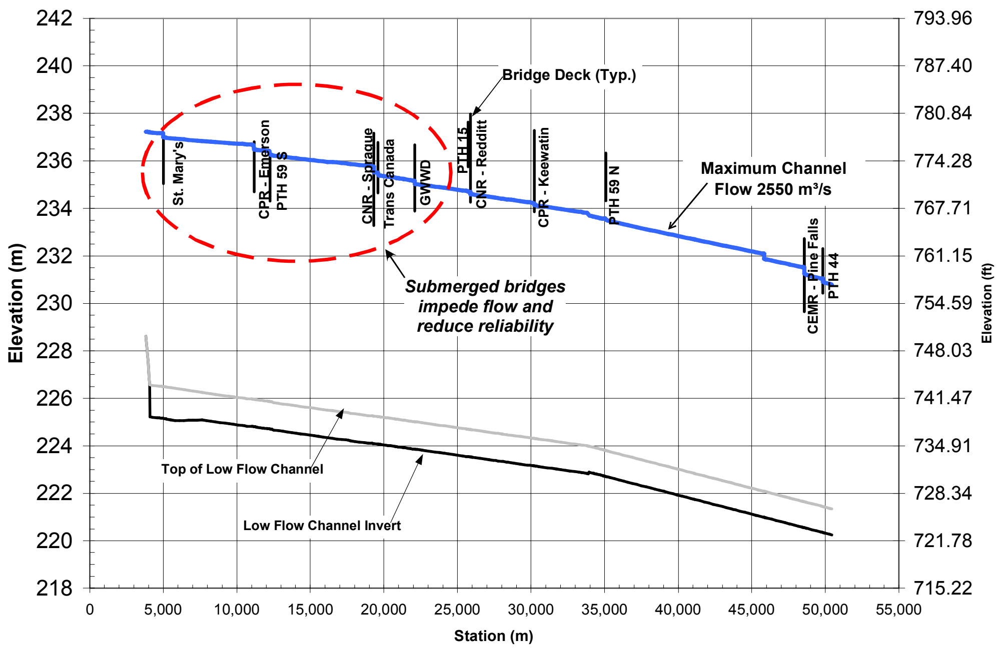


Figure 4.1-2
Bridge Crossings over the Existing Floodway Channel

The bridges were designed to have a maximum cumulative hydraulic impact on Floodway Channel water levels of 0.3 metres (1 foot) at a design flow of 60,000 cfs (1,700 m3/s).

The Existing Floodway Channel was designed to convey a flow of 60,000 cfs (1,700 m3/s) while maintaining a water surface elevation on the Red River at the Floodway Channel entrance of 234.77 metres (770.25 feet). This design was done using hydraulic backwater calculations using design cross-sections, longitudinal slope and a conservative channel roughness coefficient accounting for flow resistance in the Floodway Channel. The ultimate capacity of the Floodway Channel, assuming the existing bridges were not in place, was estimated to be 2,800 m3/s (100,000 cfs) with a corresponding elevation of the Red River water surface at the entrance to the Floodway Channel of 237.13 **m** (778.0 feet), and a minimum freeboard along the Floodway Channel embankments of 2 feet (0.6 metres). This represents the maximum water level south of the structure that could be allowed without potential overtopping of the West Dyke or the Floodway embankments.

Following construction of the Existing Floodway, discharge metering data collected between 1969 to 1999 allowed a re-estimation of the channel capacity to 1,700 m3/s (61,500 cfs) when the water surface elevation of the Red River at the Floodway Channel entrance is 234.77 m (770.25 feet). Additionally, the conservative value for the roughness coefficient of the channel was reduced to reflect the actual existing channel performance. With the bridges in place, the capacity is estimated to be 2,500 m3/s 90,000 cfs (90,000 cfs) for a level of 237.13m (778 feet) at the Floodway Inlet. If the bridge crossings were removed or raised, the ultimate capacity of the channel itself would be close to the original estimated channel design capacity of 2,800 m3/s (100,000 cfs). It was determined through analysis that it would not be necessary to remove or raise all the bridges to achieve this capacity, i.e., that the increased capacity could be gained if only the seven most upstream bridges were raised or removed (Appendix B: Preliminary Engineering Report, Floodway Channel Pre-design, 2004). Figure 4.1-3 shows the hydraulic profile indicating the seven bridge crossings which would be submerged at ultimate capacity. Note: The CPR Lac du Bonnet bridge was removed about 2 years ago



Source: KGS/Acres/UMA 2004b

Figure 4.1-3
Bridges that would be Submerged with Ultimate Capacity

4.1.1.2 Existing Inlet Control Structure

The entrance to the floodway is located in the eastern bank of the Red River near St. Norbert. An earth-fill weir at the entrance ensures that flows below flood level continue down the Red River.

The inlet control structure is located on the Red River just downstream from the floodway inlet. The purpose of the control structure is to regulate the flow between the natural channel of the Red River and the Floodway Channel, during the period of high water levels. The gates of the control structure are normally in a submerged position with about 1.8 metres (6 ft)of water over them in the summer months.

To prevent floodwaters from bypassing the inlet control structure, dykes have been constructed on either side. On the east side of the Red River, the dyke is incorporated into the floodway embankment. To the west of the Red River, the dyke extends for 44 kilometres from the inlet control structure to a point where the natural ground is above the design flood elevation.

The inlet control structure is comprised of two independent steel gates housed within a monolithic concrete structure. Each gate has its own flow channel, which is separated by a central concrete pier that supports the inlet control structure bridge deck and the Floodway Inlet Control Structure Control Room. Through controlled raising and lowering of the flow control gates, the inlet control structure regulates the Red River water level at the entrance of the Floodway Channel, controlling how much Red River flow is permitted to pass through the urban reaches of the Red River within Winnipeg.

During non-flood conditions, the gates rest in a fully down position with the top of the gates at elevation 728 feet (221.9 m) at the river bottom. In summer months, the Red River water surface elevation passing above the gates is normally at about 734.35 feet (223.4 m). In periods of flood, the gates are raised, and the inlet control structure functions as a flow restriction that causes Red River water levels to back up upstream of the Inlet Control Structure, allowing upstream water levels to rise enough to spill over the earthen lip and flow into the Floodway Diversion channel.

The Inlet Control Structure was designed to accommodate specific maximum design conditions, known as the Probable Maximum Flood (PMF) event without failing. The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a particular drainage area. At this flow, the water surface elevation on the Red River at the Floodway Channel's entrance would be regulated to 778 feet (237.13m). Under these conditions, floodwaters would not endanger the structure as the floor of the machine room containing the gate motors is at elevation 779.8 feet.

4.1.1.3 Existing Outlet Structure

The difference in water level over the entire reach of the Floodway Channel from inlet to outlet is about 5 metres (18 ft) under design conditions but the corresponding difference in elevation along the Red River Channel between those same points is about 10 metres (32 ft). The purpose of the outlet structure therefore is to dissipate the differential energy in the water from the Floodway Channel at its point of re-entry into the Red River near Lockport, thereby preventing damage and erosion to the channel and in the River. The outlet structure is founded on bedrock and is constructed of concrete with an uncontrolled rollway, having a crest length of 160 feet and a stilling basin 120 feet in length. The design capacity of the outlet Channel structure is 60,000 cfs.

4.1.1.4 Existing West Dyke

Dykes on either side of the Inlet Control Structure retain the floodwaters. East of the Red River, the East Dyke is incorporated into the embankment created by the Floodway Channel. The dyke extends parallel to the Floodway Channel and on its west base for a distance of 9.7 km (6 miles). West of the Red River, the West Dyke extends a distance of about 32 km (20 miles) in southern and a westerly direction from the Inlet Control Structure up to the point where the natural ground is above the design flood elevation. The West Dyke contains the floodwaters of the Red River from the south-west and prevents the flow from passing into the La Salle River watershed, where it could bypass the Floodway Inlet Control Structure and enter Winnipeg directly. During large floods, the river water level is well above the natural bank level and flooding extends laterally over many miles (some 25 miles in 1997, for example). During the 1997 flood, the dyke was raised and extended an additional 25 km to prevent floodwaters from bypassing the structure and entering the City.

4.1.1.5 Operating Rules

Operating Rules for the Existing Floodway were established between the various levels of governments and originally documented in 1970. The rules are intended to adhere to natural conditions upstream of the Inlet Structure until Winnipeg is threatened with major flooding after which some above-natural water levels are allowed, with the understanding that flood damage will be compensated. These rules were amended in 1999, and are discussed in Chapter 5, Section 5.3.

Some other guidelines include:

- The Floodway gates should not be operated until ice on the river is flowing freely unless flooding in Winnipeg is imminent.
- To minimize bank slumping along the river in Winnipeg and at the same time reduce the probability of sewer backup problems, final gate operations once the level at the entrance to the Floodway Channel recedes to elevation 229 metres (752 feet), shall be carried out in consultation with the City of Winnipeg.
- The horn at the Floodway Structure shall only be operated once, before the first gate operation of the year. The horn should be sounded a half-hour before the first gate operation to alert residents that the Floodway Structure is being put into operation.

4.1.2 Portage Diversion

The Portage Diversion is an 18 mile long channel designed to carry up to 25,000 cfs of flood flow from the Assiniboine River at a point just upstream of Portage la Prairie northward to Lake Manitoba. The removal of flood flows via the Portage Diversion provides flood protection not only to the City of Winnipeg but also to the City of Portage la Prairie and the area adjoining the Assiniboine River between those cities and North of Winnipeg along the Red River to Lake Winnipeg. Construction of the project commenced in 1965 and was completed in 1970 at a total cost of $20.5 million. It involved approximately 10,000,000 cubic yards of excavation as well as construction of several structures including three highway bridges and three railway bridges across the diversion channel. The major elements of the project are the dam in the Assiniboine River, the concrete spillway control structure, (River Control Structure), the Diversion Structure that controls water entering the Portage Diversion, the diversion channel itself, two gradient control structures and the Outlet Structure where the diverted flow passes into Lake Manitoba.

4.1.3 Shellmouth Dam

The Shellmouth Dam is located about 30 miles northwest of Russell in an area where the valley of the Assiniboine River is wide with high banks. The dam is about 70 feet high and 4,200 feet long. It has a reinforced concrete horseshoe-shaped conduit 15 feet in diameter by means of which reservoir releases are made. Flood flows in excess of the conduit capacity are either stored in the reservoir or are passed over an ungated concrete chute spillway. The reservoir created by the Shellmouth Dam is approximately 35 miles long and is capable of storing 390,000 acre-feet of water. The protection afforded by the reservoir extends over the entire reach of the Assiniboine River between the Shellmouth Dam and its **confluence** with the Red River at Winnipeg and North of Winnipeg along the Red River. The Cities of Brandon and Portage la Prairie, as well as Winnipeg, benefit by both flood reduction and low flow augmentation. Construction of this project was initiated in 1964 and was completed in 1972 at a cost of $10.8 million.

4.1.4 Need for Improved Flood Protection

The Floodway was designed 45 years ago to provide Winnipeg with protection up to an estimated flood magnitude that would be exceeded once in 160 years, based on statistical data on river floods that existed at that time. Recent recalculation of flood frequencies considering the past 40 years of flood data, a period that experienced a series of relatively large floods (in 1966,1974,1979,1996 and 1997), reveal that the design flood established in the 1960's actually would represent only a 1 in 90 year flood event. Analyses of the historical record reveal that large flood events can occur more frequently than were calculated using the old (pre-1960's) data.

Flood control works on the Red and Assiniboine Rivers have been extensively used since their completion. Floods occurred on the Red River in 1969, 1970, 1974, 1979, 1987 and 1997. The 1979 flood was similar in magnitude to the devastating flood of 1950, which was the catalyst for the construction of these works. In 1997, a flood surpassed only by the 1826 flood of record occurred on the Red River. This event served to demonstrate yet again the enormous value of the flood control works to the City of Winnipeg and, at the same time, to emphasize the likelihood that a flood of even greater magnitude could occur.

Costs incurred by damages to the City of Winnipeg under the "do-nothing" scenario in the event of a flood event larger than the 1997 flood would dwarf the cost of expanding the Existing Floodway system. The City of Winnipeg has benefited enormously from the establishment of the Red River Floodway and the overall flood protection system, where serious flooding has been avoided since control works went into operation. Potential damages prevented since 1969 are in excess of $8 billion in Manitoba. The damages prevented in 1997 are significant. In the absence of these works, a large area of the City would have been **inundated**, hundreds of thousands of people would have been evacuated, and the economic centre of Manitoba would have been completely paralyzed. The cost of the Red River Floodway, Portage Diversion and Shellmouth Reservoir totaled $94 million when constructed. It is clear that these flood control works have been extremely cost effective.

Following the 1997 Red River Flood, President Bill Clinton and Prime Minister Jean Chretien asked the International Joint Commission (IJC) to study the flood and its impacts. In June of 1997, the Canadian and American governments charged the IJC with examining and reporting on the causes and effects of damaging flood events in the Red River, and to make recommendations on means to reduce, mitigate and prevent harm from future flooding in the Red River Basin. In September of 1997, the IJC established the International Red River Basin Task Force to examine a range of alternatives to prevent future flood damage. This Task Force undertook a series of studies and in 1999 commissioned a study on flood risks in Winnipeg and possible means to reduce those risks. Reports from the study were submitted to the IJC Task Force in 1999 and in 2000.

The reports submitted to IJC contained several major findings:

- Winnipeg is at risk to major floods of the magnitude of the 1997 flood, or larger.
- Many vulnerabilities exist that should be improved.
- Potential damages to Winnipeg due to floods exceeding the 1997 Flood event would be as much as $17 Billion for a 1 in 1000 year flood.
- The two best options to provide a major increase in flood protection were the Ste Agathe Detention Structure or the Floodway Expansion.

As discussed in Chapter 1, Section 1.4.4, Floodway Expansion was selected as the preferred flood protection option. The following sections will discuss the conceptual design of the proposed Project.

4.2 OVERVIEW OF RED RIVER FLOODWAY EXPANSION PROJECT

4.2.1 General Design Criteria

The design objective for the expansion of the Red River Floodway is the requirement for the passage of a flood with a probability of being equaled or exceeded once in 700 years. This flood magnitude is also referred to as the "1 in 700 years flood", and represents the "design flood" for the project. The design criterion of the expanded Floodway to handle a flood of this magnitude is that this operating performance must be achieved at a maximum water level of 237.13m (778 feet above sea level) at the Floodway entrance. The overall design criteria are consistent with the current Floodway Operating rules as discussed in Chapter 5.0.

In the course of addressing the design objective, it became clear that the hydraulic capacity of the Expanded Floodway could be best achieved by a combination of widening the Floodway Channel and the improvement of flow capacity through the various channel bridges (by lifting or widening or both). The design optimization process focused on the balance between channel deepening/widening and bridge flow capacity, as discussed later in this Chapter.

These detailed documents are available for viewing at the MFEA Offices.

The preliminary design of the Floodway Project is a very complex engineering undertaking as there are many different components of the overall Project, all of which interact. There are numerous technical documents, which have been used as resource material to assemble this Project Description (see Table 4.2-1).

General design criteria for the overall Project will be described followed by a detailed description of the various distinct components of the Project.

The Project covers a large linear area and is difficult to describe in concise figures or maps. A series of drawings DWG 004c (19) have been included in Appendix 4, which provide physical details of the entire project.

4.2.2 Approach to Preliminary Engineering Design

Preliminary design of the Floodway Expansion was separated into two project definition stages. The first phase consisted of primarily site investigations and baseline data acquisition to determine existing conditions. This phase is known as Project Definition and Environmental Assessment Part 1, "PDEA1". A second phase (PDEA2) consisted of pre-design of the various components of the Floodway Expansion project, many of these components inter-related to each other in the pre-design process. PDEA1 was separated by MFEA into 11 work packages, which were contracted out to local engineering consultants according to their relevant expertise. The PDEA2 design work was separated into 7 Work Parcels, as shown in Table 4.2-2.

Table 4.2-1
Appendices of Preliminary Engineering Report

Reference	Appendix Name	Floodway Component Addressed	Information Contained	Author Company
A	Bridges and Transportation Pre-Design	Bridges, Roadworks	Pre-design of railway and highway bridges and associated road and rail works.	Dillon / Ndlea
B	Floodway Channel Pre-Design	Channel	Pre-design of Floodway Channel including Low Flow Channel, East and West Embankments, revegetation plan, and erosion control measures	KGS / Acres / UMA
C	Inlet Control Structure Pre-Design	Inlet Control Structure	Dam Safety Review, Inlet Erosion Protection Pre-Design, assessment of redundancy features for gate system.	SNC / Wardrop
D	Outlet, Local Drainage Structures and Syphons Pre-Design	Outlet Structure, Outlet Channel, Syphon, Aqueduct, Drainage Structures	Pre-design of Outlet Structure / Channel, Seine River Syphon, Local Drainage Structures, Aqueduct Crossings	KGS / Acres / UMA
E	Utilities Crossings Pre-Design	Hydro Utilities, Gas Utilities, MTS Utilities, Winnipeg Oil Pipeline, East St. Paul Waterline	Pre-design of modifications to Utilities crossings, Utilities database	Stantec / Teshmont
F	West Dyke Surveys, Field Investigations and Pre-Design	West Dyke	Pre-design of increases in freeboard and extension of the West Dyke, Surveys, Soil Logs	Acres / UMA
G	Site Investigations for Floodway Channel	Channel	Site Investigations of Channel, including soil logs	UMA
H	Investigations of Enlargement of East Embankment Gaps	Channel	Hydraulic analysis of East Embankment Gaps for use in Channel optimization	Acres
J	Site Investigations for Floodway Bridges	Bridges	Site Investigations of Bridges, including soil logs	SNC / Wardrop
K	Test Excavations	Channel	Test excavations in clay and till soils in the Channel, including soil logs	KGS
L	Environmental Baseline Studies – Water Regime Effects	Channel	Effects of Project on water levels and flows, information on **ice jamming**, and estimation of potential flood damages.	Acres
M	**Groundwater** Investigations	Channel	Drilling, testing, well installation, monitoring of the water levels and water quality in the bedrock **aquifer**, plus a regional well inventory	KGS

Reference	Appendix Name	Floodway Component Addressed	Information Contained	Author Company
N	Regional Groundwater Modeling	Channel	3-D modeling of regional bedrock aquifer and surface water intrusion in the channel to predict response to channel deepening scenarios	KGS
O	Compilation of Floodway Site Investigations	Channel	Compilation of all site investigations in Floodway, including soil logs	KGS
P	Surface Water Intrusion Modeling	Channel	Sensitivity modeling of surface water intrusion from Floodway into the groundwater system for channel widening and flood conditions	KGS
Q	Potential Groundwater Impacts for Channel Pre-Design	Channel	Summary of groundwater impacts of project for proposed Floodway expansion scheme	KGS
R	Groundwater Investigations of Floodway Area	Channel	A household well survey, well inventory, and monitoring well installation.	SNC / Wardrop
S	Groundwater Modeling of Floodway Area	Channel	3-D groundwater modeling of the Birds Hill sand and gravel aquifer and saline intrusion potential in the bedrock aquifer	SNC / Wardrop

Table 4.2-2
PDEA 2 Work Parcels and Descriptions

Contract Number	Description
Work Parcel 1	PDEA 2 Lead Consultant
Work Parcel 2	Bridge and Transportation Pre-Design
Work Parcel 3	Floodway Channel Pre-Design
Work Parcel 4	Inlet Control Structure Pre-Design
Work Parcel 5	Outlet, Local Drainage Structures and Syphons Pre-Design
Work Parcel 6	Utility Crossings Pre-Design
Work Parcel 7	Environmental Assessment and Licensing

PDEA2 was conducted through an iterative design process. The pre-design process was split into three “design iterations”, each documenting the status of design for each Work Parcel at appropriate time intervals for review by the other consultants. The iterative approach allowed all pertinent and inter-related information to be incorporated into the respective Floodway Expansion component pre-designs as prepared by the respective consultants. The iterative process provided an effective means to allow evolution of a complex design by transferring information between different Work Parcels that affect the design of other Floodway components.

4.2.3 Overview of Project Components

The components of the Floodway Expansion Project (KGS/Acres/UMA 2004a) are as follows: Figure 4.2-1 shows the main components of the project.

1. Channel Excavation
   - Widening of channel in varying amounts up to as much as 60 m (200 ft) (no deepening planned at this stage)
   - A volume of excavation of approximately 20,900,000 m^3 (27,300,000 yd^3)
   - Revegetation of all areas where bare soil will be exposed by the excavation

2. Restoration/Armouring of the Low Flow Channel
   - Infill of previously eroded zones of the Low Flow Channel
   - Placement of riprap protection in vulnerable zones to protect against future erosion

3. Expansion of the opening in the East Embankment on the east side of the Grande Pointe Drop Structure
   - This will involve removal of a length of approximately 400 m (1,300 ft) of the East Embankment
   - Excavation of existing fill down to El. 235 m (771 ft.)

4. Replacement of seven bridges
   - St. Mary's Road Bridge, including realignment of roadworks
   - CPR Emerson Rail Bridge
   - PTH 59 South (Southbound)
   - Trans Canada East
   - PTH 15
   - PTH 59 North
   - PTH 44

5. Rehabilitation of six bridges, including raising elevation of girders where needed
   - PTH 59 South (Northbound)
   - CNR Sprague rail bridge
   - GWWD rail bridge
   - CNR Keewatin rail bridge
   - CPR Redditt rail bridge
   - CEMR Pine Falls rail bridge

6. Enlargement and improvement of the Outlet Control Structure
   - Increase of width (laterally across channel) by approximately 50 m (164 ft)
   - Enlargement of the stilling basin and improvement in its capability to dissipate energy by using energy absorbing appurtenances

7. Replacement/Rehabilitation of drainage structures that discharge local runoff into the Floodway, including enhancement of the discharge capacity to comply with a 1 in 100 year design inflow where practical
   - Centreline Drain – replacement
   - North Bibeau Drain – replacement
   - Cook's Creek Diversion – repair (retains 1 in 50 year capacity)
   - Springfield Road Drain – replacement
   - Shkolny Drain – replacement
   - Ashfield Drain – replacement
   - Transcona Storm Sewer Outlet – replacement

8. Modification of the two water supply Aqueducts, and the Deacon Drain Line, owned by the City of Winnipeg

9. Modification of seven Electrical Transmission Line Crossings

10. Replacement of utility lines
    - 2 Manitoba Hydro fibre-optic communication lines
    - 10 crossings or parallel natural gas lines
    - 5 buried MTS cables
    - 5 MTS cables on modified bridges

11. Replacement of two oil pipelines

12. Increase in the height and length of the West Dyke to protect against wind effects during major floods
    - Extension in length by 15 km (9 miles)
    - Increase in height by up to 2.7 m (8.9 ft)
    - Fill quantities totaling 4,600,000 m^3 (6,000,000 yd^3)

13. Improvement in protection and reliability of the Floodway Inlet Control Structure
    - Erosion protection on the upstream and downstream surfaces of the embankments adjacent to Inlet Control Structure
    - Installation of a fire protection system in the control room and equipment room
    - Improvements in redundant features in the gate systems
    - Improvements to hoists

Although not included in the project at this time, the preliminary design of the Floodway Expansion assumes that there will be a variety of improvements made to the flood protection infrastructure in the City of Winnipeg, as recommended in the SAFE Study Report (KGS, 2001). The main Project components are discussed in greater detail in the following Section.

4.3 FLOODWAY CHANNEL EXPANSION

Details pertaining to Floodway Channel Expansion are derived from the most current design information as of June 2004. This information is contained in the document "Preliminary Engineering Report: Appendix B Floodway Channel Pre-design"(KGS/Acres/UMA 2004a).

4.3.1 Basis of Design

The design flow for the expanded Floodway Channel will be 3960 m^3/s (140,000 cubic feet per second [cfs]) with a maximum design level of Elevation 237.13 metres (778 feet). Design conditions for this flow and other design events consist of the following (Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004.):

<u>Channel</u>

- The Floodway Channel will be expanded by widening at locations that provide the most cost-effective increase in discharge capacity, as determined by optimizations techniques described in Section 4.3.3. No deepening of the Floodway Channel is required, based on the current knowledge of the project.
  - Shear stresses (i.e., erosion stresses) on the base of the channel that exceed those experienced in 1997 can be permitted for floods exceeding the 1 in 100 year event, in that these effects can be repaired.
- The channel will be designed so as not to deposit sediment from the flood waters.
- The lateral slope of the new channel bottom adjacent to the existing channel base will be the same as the existing channel base.
- The hydraulic roughness of the channel for design purposes will be characterized by a Manning's coefficient of 0.028, consistent with the experience with the Existing Floodway Channel. All other energy losses, including expansion, contraction and friction/form losses at bridges and changes in channel cross section are included in the calculation of channel discharge capacity.
- Changes in channel geometry will have adequate transition zones to minimize hydraulic losses.
- For channel side-slopes, including the utilities and bridges, a minimum estimated factor of safety against sliding of 1.5 is required under long-term normal operating conditions. For extreme operating conditions with a saturated bank and at the end of construction assuming critical high bedrock pressures, a safety factor of 1.3 is considered acceptable. At the City of Winnipeg Branch I and II Aqueducts, the minimum stability safety factor for end of construction/worst-case conditions has been set at 1.5.
- The stability analyses have shown that a setback distance of 10 metres on the bench from the top of the channel side-slope to the toe of the existing disposal embankment will be required as a minimum. Similarly, a setback distance of 25 metres will be required from the crest edge if the existing disposal embankment to the toe of the new excavation disposal material.

- Priority will be given to placing material within the existing project right-of-way to minimize the requirement for land acquisition. The extent of land outside the right-of-way that may be required for disposal of the main channel excavation material will be defined in Work Parcel 3. Land requirements for bridges, utilities, and the outlet structure will be defined under their respective Work Parcels.
- The side slopes of the widened Floodway Channel shall be set back at least 10 metres from the existing transmission towers in areas where the tower is to be left in place, without special measures. Encroachment on the 10 metres setback can be permitted where stabilization measures are constructed.
- Temporary closures of openings in the adjacent embankments, where required to prevent outflow beyond the limits of the Floodway right-of-way are deemed to be acceptable.
- Revegetation of the excavated surfaces of the Floodway Channel shall be designed to provide the maximum resistance to damage due to mid-summer flooding. This will require the use of specially selected grasses that minimize the risk of damage from inundation.

Low Flow Channel

- The location and orientation of the Low Flow Channel (also known as the Pilot Channel) shall continue as it exists.
- The low flow channel shall have a discharge capacity that is adequate to permit the passage of the local runoff that enters the Floodway via the connected drainage structures, without causing water levels to exceed the top of the Low Flow Channel. The design shall be based on the passage of the 95 percentile at bankfull stage in the Low Flow Channel, as derived from estimates of daily inflows to the channel during the summer season.
- The Low Flow Channel will be graded to a depth consistent with the existing channel. Areas that have eroded will be protected with a geotextile and a layer of riprap armouring of sufficient size and gradation to resist soil movement under design flood conditions (1 in 700 year). Those portions of the Low Flow Channel that are in till and have not shown any tendency for erosion will be left unprotected. The hydraulic roughness of the riprap armouring in the Low Flow Channel will have a Manning's coefficient of 0.034.

Excavation/Disposal

- The surface grade on the crest of new excavation material disposal embankments and the bench between the excavation edge and the disposal material shall be sloped at 2% minimum to promote surface water runoff.

Low Level Crossing

- The low level crossing at Dunning Road will be reinstated at the end of construction. Requirements for public access during construction are under consideration.

4.3.2 Geotechnical Pre-Design and Channel Geometry

The 2001 "SAFE Study" Report issued by KGS identified key issues of concern regarding geotechnical design of an expanded Floodway (Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004).

- End of construction and long–term stability of an expanded Floodway Channel. Issues included impact on channel sideslopes resulting from channel widening, stability concerns associated with placing new excavation disposal material on top of existing disposal embankments, and questions regarding the ability to safely steepen the slideslopes from the existing 1V: 6H configuration.
- End of construction and long term stability of the bridge abutments at all bridge crossings if the existing channel sideslopes (1V:9H) were steepened to 1V:6H and the abutment fills were raised more than 3 metres to allow for the bridge girders to be raised above critical maximum design flow levels in the Floodway Channel.
- End of construction and long-term stability of the base of the Floodway Channel against **basal** heave or blowout. Original construction of the Floodway Channel resulted in significant hydraulic fracturing and seepage problems in the 1960's. This issue has been eliminated by the recent decision not to deepen the existing channel.

In order to address these aforementioned geotechnical design concerns, the following steps were taken:

- Detailed geotechnical and groundwater investigations were undertaken as part of the PDEA1 activities. These investigations included an extensive advanced materials testing program to measure the structural characteristics of the Lake Agassiz clays.
- Detailed and extensive computer-aided slope stability modeling was completed using the results of the geotechnical investigations and supplemented by a laboratory-testing program. Slope stability modeling was conducted using a suite of software models developed by GeoSlope International Ltd. The GeoSlope suite of numerical models was selected for its ability to fully assess the stress deformation and groundwater flow aspects of the channel design. The GeoSlope package allows for estimation of the soil response due to excavation and loading, coupled with impacts to porewater pressures in the soils. These features of the model were considered important to model the behaviour of the Floodway Channel during the critical post-construction condition when porewater pressures are higher due to undrained excavation disposal material placement. Channel section stability was then analyzed for both short-term post-construction conditions and long-term conditions using the GeoSlope software.

The models also examined the impact of lateral excavation of the channel slopes on local and global stability. The models examined the stability of three equal excavation "steps" in widening the existing side-slopes and lateral excavation. The modeling concluded that the existing channel could be widened, but a minimum offset of 10 metres must be maintained between the top edge of the expanded channel sideslope and the toe of the disposal berm to prevent a decrease in existing stability conditions. This offset has been established as a design requirement for the channel expansion.

Based upon the aforementioned analyses and modeling efforts, a recommended expanded channel geometry was defined as follows:

- Maintain the existing 1V:6H side slopes for the main channel areas
- Widen the Floodway Channel only to the point where at least a minimum of 10 metres (33 ft) offset is maintained on the bench between the top edge of the channel side slopes and the toe of the existing disposal embankment (Figure 4.3-1).
- New disposal material may be placed on top of the existing disposal embankment providing the toe of the new disposal embankment is set back at least 10 metres (33 ft) from the top edge of the existing disposal embankment. A preference was identified that a set back distance of 25 metres be maintained, where possible, and that preferred setback distance has been maintained throughout this pre-design phase.
- The total height of the new disposal embankments should not exceed 5 metres (16.5 ft), and the total height of the existing plus new disposal embankments should note exceed 10 metres (33 ft).

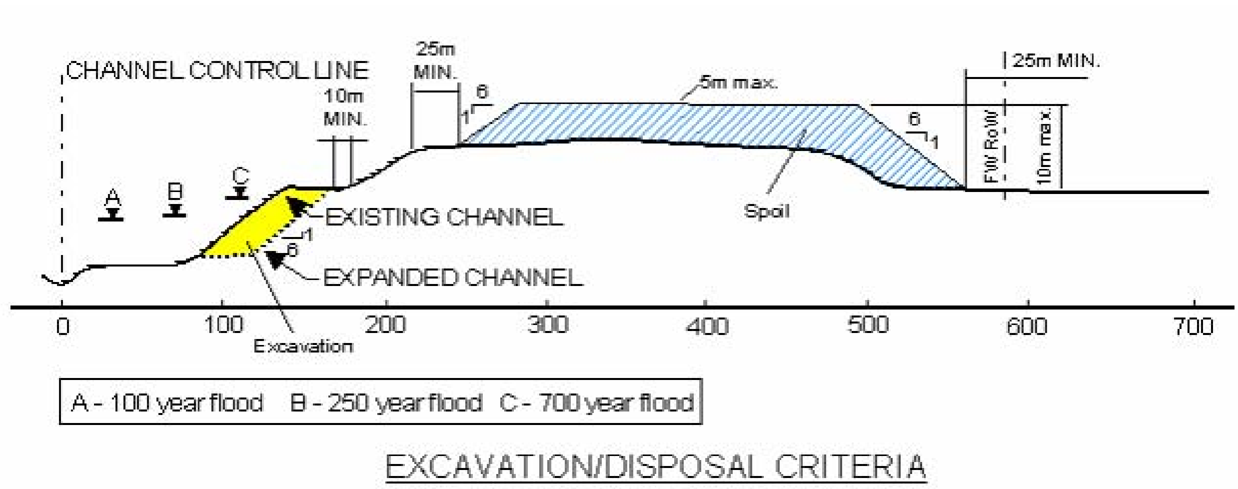


Figure 4.3-1
Excavation / Disposal Criteria

Based upon the geotechnical investigations and modeling, a recommended channel expansion geometry was developed which causes no reduction to the existing channel's current design safety factor. Minor additional stability modeling of the Floodway Channel is anticipated during the final design phase provided there are no changes to the proposed expanded channel geometry. A need for detailed abutment stability modeling at each bridge crossing during final design was identified, however this modeling requires data that is only available once the abutment configurations are finalized.

4.3.3 Floodway Channel Expansion Optimization Considerations

The engineering design consultants recognized that significant cost savings could be realized through a carefully developed Floodway Channel configuration that meets the stated design objectives, while balancing other factors, such as bridge crossing, that will influence the cost of construction. The pre-design included an optimization effort of the expanded channel configuration in preparation for final design. While significant optimization has been incorporated into the pre-design phase of the expanded channel configuration, ongoing optimization is anticipated for specific components through the final design process. Multiple factors were addressed in the pre-design optimization of the channel design, including (but not limited to):

- Unit pricing for excavation depending upon the nature of the material.
- Varying hydraulic effectiveness of excavation depending on location along the channel (the most cost-effective is near the upstream end of the channel).
- Channel orientation so that transmission lines, various utilities and other adjacent facilities are impacted to the least extent practical.
- Consideration of groundwater effects especially as related to Floodway Channel depth.
- Consideration of limiting channel velocities and associated erosion.
- Consideration of additional improvement at the Floodway entrance through the enlargement of the existing gaps in the east embankment, and how this could reduce overall channel excavation requirements.
- Incremental cost of bridges
- Reconstruction of Branch Aqueduct crossings
- Springhill area complexities

The channel configuration that achieves all the simultaneous objectives required a methodical numerical iterative technique of balancing each of the competing factors. The overall process of optimization is described in KGS Report Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004. Some of the key optimization conclusions are discussed below.

4.3.3.1 Bridge Crossing Optimization

The optimization process led to several key design decisions. Costs were minimized if the bridge girders at each Floodway crossing were set above the maximum water surface profile, i.e., no submergence. Consequently, the expanded Floodway Channel design adopted an approach where the lengthening of the bridge decks/girders would occur only as required to span the channel at the designated new increased height.

4.3.3.2 Channel Deepening vs. Widening Optimization

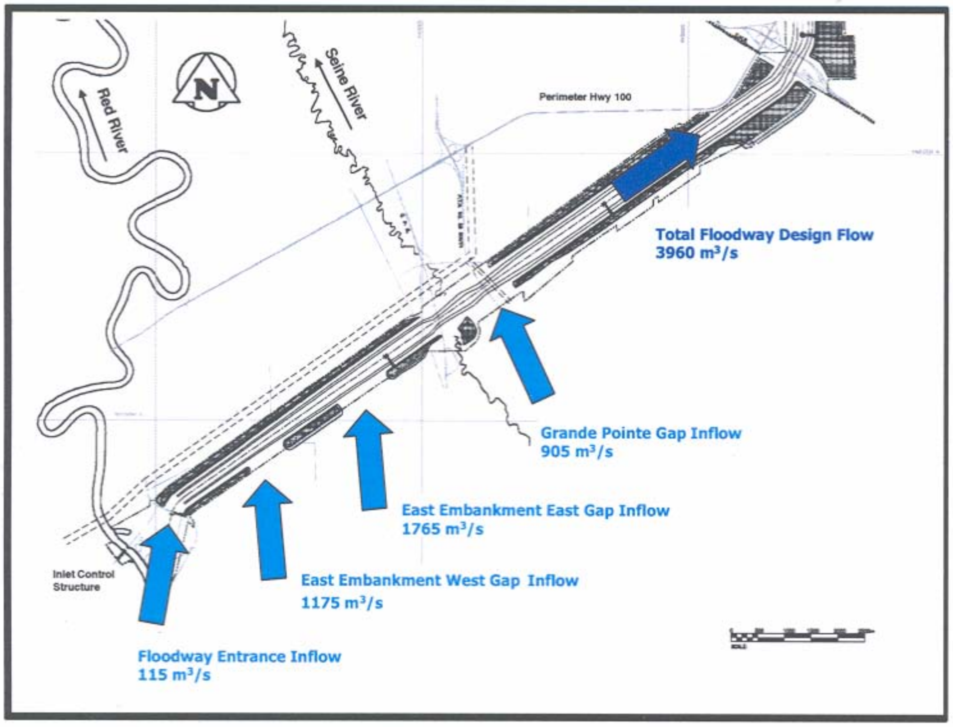
The optimization of channel design also assessed the options of Channel Widening vs. Channel Deepening. The optimization results showed that deepening of the Floodway Channel is a more cost-effective option than widening alone, if considering only the channel and bridge costs. The cost advantage to doing so represented a $9 Million savings (approximate). When the potential for necessary mitigation works to deal with potential groundwater changes resulting from channel deepening are considered, it was determined that the costs associated with groundwater mitigation, public concerns, and potential project delays could approach the potential $9 Million savings associated with channel deepening. Consequently, the revised design objective considered expansion of the Floodway Channel by widening only. While channel deepening cannot be abandoned completely until completion of the final design, there are no issues at present that are known that could reverse the decision to abandon Floodway Channel expansion by deepening. Some minor exceptions to the “widening only” design objective are anticipated in highly localized areas of the channel. It is anticipated that modest (0.3 metres) deepening of the Low Flow Channel near the upstream end of the channel may be required to avoid the potential of pooling of water that may make this area attractive to fish and result in fish stranding.

4.3.3.3 Channel Sideslopes Optimization

Optimization investigated possible economic advantages with steepening the channel sideslopes in the reaches between the bridges. Due to substantial increase in risk of shallow slope failures and the potential for large future maintenance costs associated with steepened sideslopes, the optimized design calls for steepening of sideslopes only at bridge crossings and not in reaches of the channel between each crossing.

4.3.3.4 East Embankment Gaps Optimization

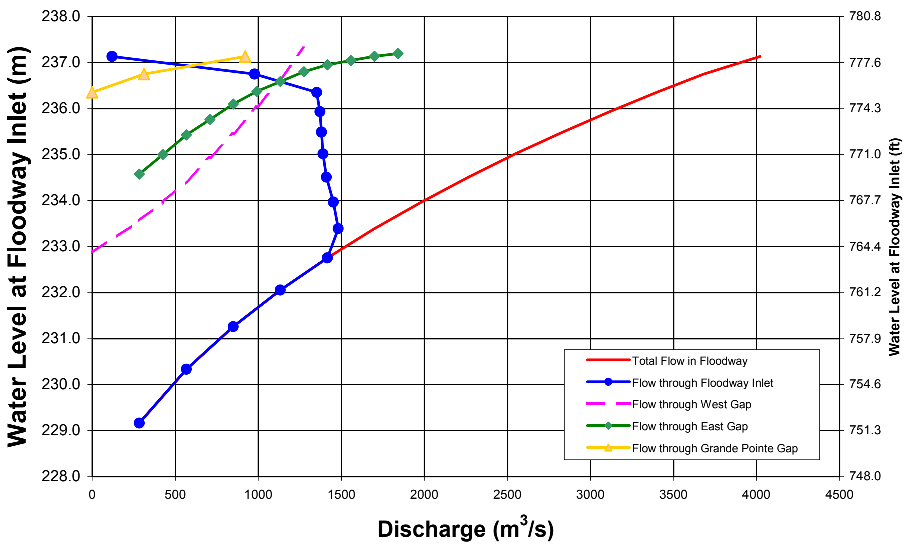
The 1997 Flood revealed head loss and reduction of discharge capacity of the Floodway Channel due to constrictions near the entrance of the channel. Studies in 1999 and 2000 presented options to improve the hydraulic conditions at the entrance of the Floodway Channel. The selected improvement option was to remove two segments of the East Embankment on the southeast side of the Floodway Channel, approximately 3 km and 5 km respectively, from the original channel entrance. These gaps are now known as the East and West Gaps in the East Embankment. In addition to the East and West Gaps, a new gap now exists at the Grande Pointe Drop Structure as shown in Figure 4.3-2. Expansion of the existing East and West Gaps was assessed in the optimization of the Floodway Channel expansion and was found to be only marginally effective. Optimization did reveal that expanding the opening of the Grande Pointe Drop Structure by 400 metres would reduce the needed excavation in the Floodway Channel by at least a 2:1 ratio (Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004). The 400-metre limit to lengthening the Grande Pointe Gap was set to avoid increasing the risk of damage to the 59 Highway as the flood flow overtops to reach the Grande Pointe Structure area. Figure 4.3-3 shows the distribution of flow into the Floodway Channel via the four inlet points to the channel.



Source: KGS/Acres/UMA 2004b

Figure 4.3-2

Flow Through Floodway Embankment Gaps for Design Flow



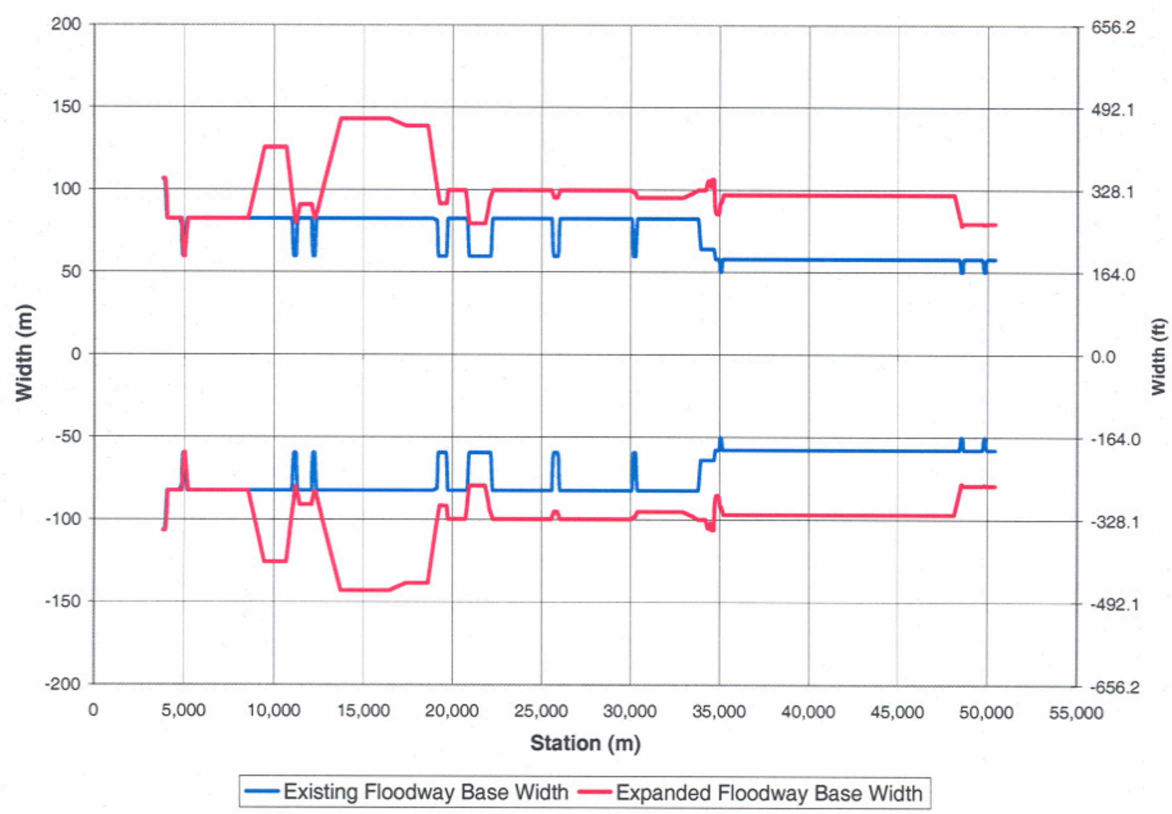
Source: KGS/Acres/UMA 2004b

Figure 4.3-3

Flow Distribution Through Floodway Embankment Gaps

4.3.4 Optimized Floodway Channel Configuration

Figure 4.3-4 shows the changes in Floodway Channel base width (depicted as changes in width from channel centerline) that were selected from the results of the optimization studies, as compared to the existing channel.



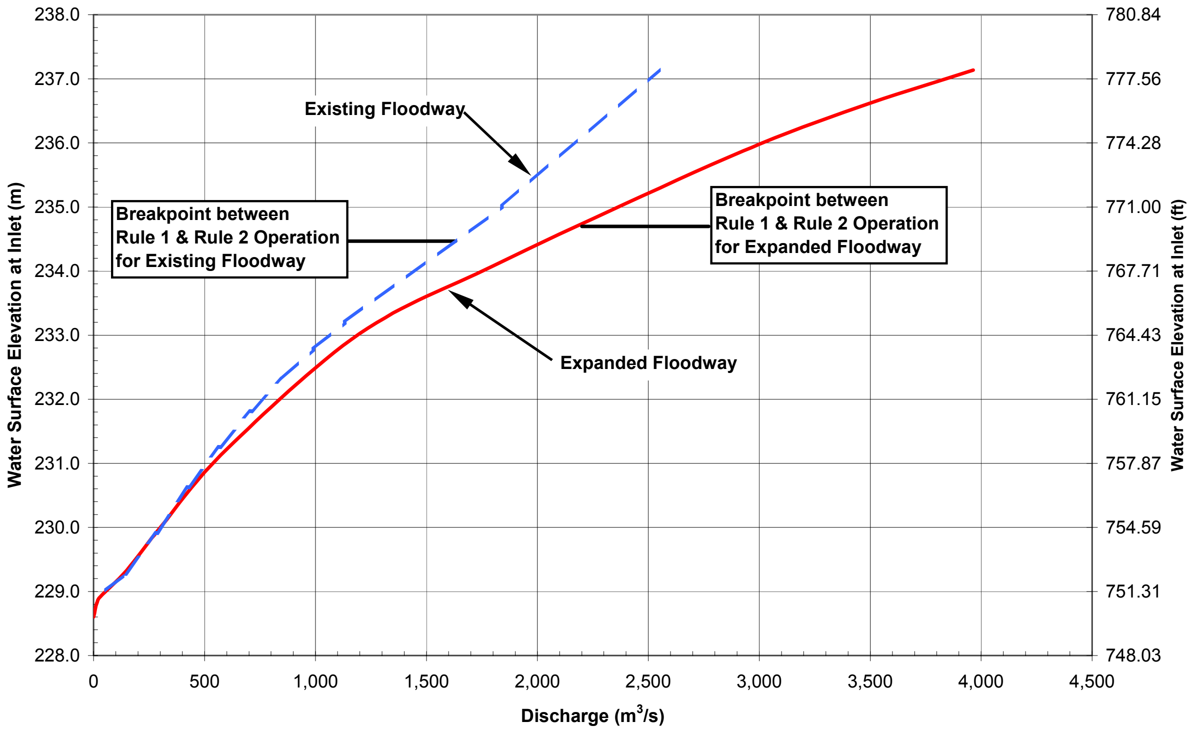
Source: KGS/Acres/UMA 2004b

Figure 4.3-4
Floodway Channel Base Widths

The proposed channel geometry is shown in plan view in Appendix 4 Dwg. Number 004.c Sheets 1 to 19, and in section on Dwg. Number 005.c Sheets 1 to 6 (KGS/Acres/UMA 2004b). The excavation limits shown on the plan drawings have been based on the requirement to pass 3960 m^3/s under the selected design criteria. The disposal embankments show the approximate limits available for material placement based on the stability design criteria, while being cognizant of existing/proposed features that will restrict material disposal (e.g., bridges, roads, critical utilities, etc.).

4.3.4.1 Stage-Discharge Relationship: Expanded Floodway

The estimated stage-discharge relationship at the Inlet for the expanded Floodway compared to the estimated relationship for the Existing Floodway is shown in Figure 4.3-5. It demonstrates that the desired discharge capacity will reach 3960 m^3/s at the maximum elevation of 237.13 metres (778 ft).

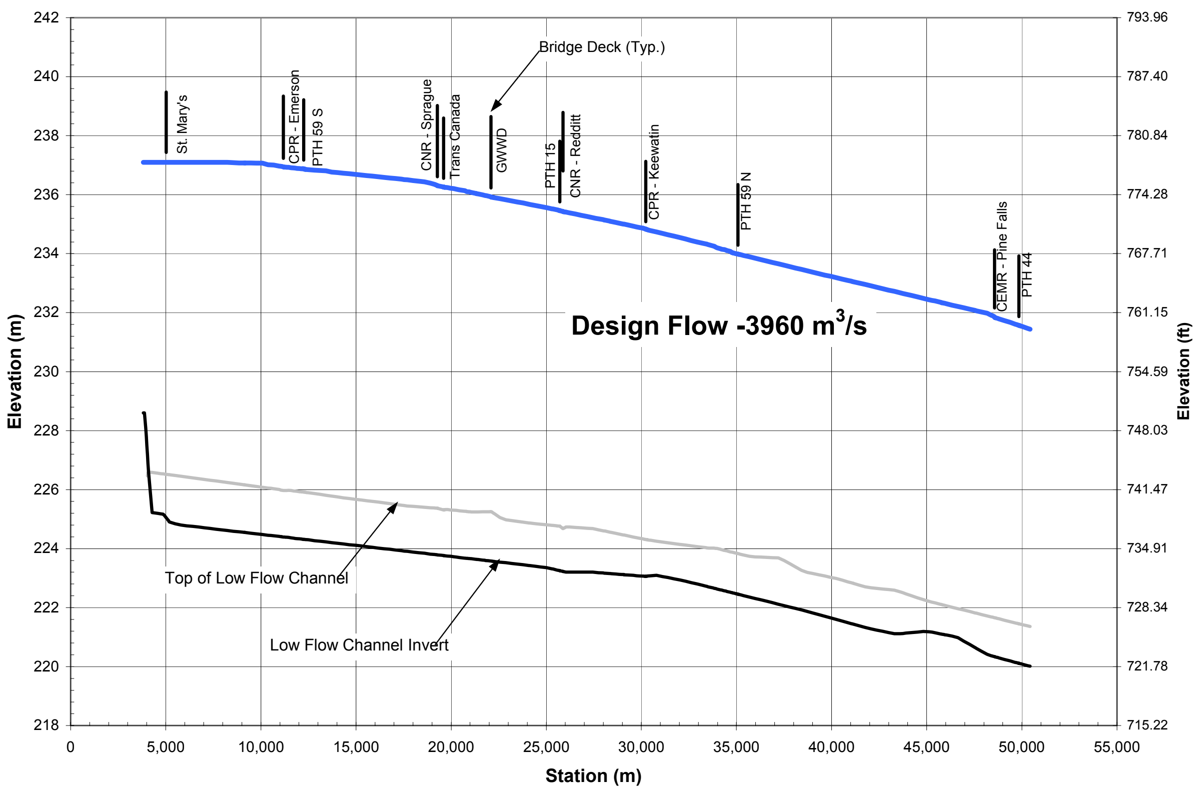


Source: KGS/Acres/UMA 2004b

Figure 4.3-5
Stage-Discharge Relationships for Expanded Floodway Channel at Inlet

4.3.4.2 Water Surface Profiles: Expanded Floodway

Estimated water surface profiles and the existing and proposed slope of the channel base for the proposed channel configuration are shown in Figure 4.3-6. The volume of excavation for this channel is estimated to be 20,900,000 cubic metres (27,300,000 yd^3), which includes excavation through the bridge areas.



Source: KGS/Acres/UMA 2004b

Figure 4.3-6
Water Surface Profiles for Expanded Floodway Channel

4.3.4.3 Channel Routing: Expanded Floodway

The excavation of the expanded channel base has generally been designed as symmetrical about the existing channel centerline. There are four main areas where this has not been the case, and the channel has been shifted off-centre to accommodate or minimize the impacts on existing site features.

The four zones along the Existing Floodway Channel requiring changes in alignment in order to accommodate existing services under Floodway Expansion include the Branch I Aqueduct/GWWD Bridge, Kildare and Cook's Creek Drainage Structures, Spring Hill Ski Area, and the Floodway Outlet Structure.

<u>Branch I Aqueduct/GWWD Bridge</u>

The channel expansion from Sta. 21+850 to 22+250 metres has been designed such that the excavation required to achieve the increase in capacity has been shifted into the east slope at the existing 1V:9H slope. The west channel slope will be maintained at its current location. This channel alignment is part of the preferred scheme to minimize the overall cost through this reach, and potential adverse impacts on the Aqueduct.

Previous stability analyses at the Aqueducts have demonstrated that the preferred solution to satisfy the selected design criteria through the area was to maintain the existing 1V:9H channel side slopes. The Branch I Aqueduct is in close proximity to the GWWD Bridge, particularly on the west side and warranted further evaluation to determine the most effective and cost efficient alternative. A number of options were considered that evaluated the interrelationship between side slope, mechanical stabilization (e.g., rockfill columns), Aqueduct alignment, channel alignment, and bridge geometry. The preferred solution is summarized below (Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004):

- Realign the Branch I Aqueduct pipe on the original pre-Floodway alignment;
- Maintain 1V:9H side slopes through the GWWD bridge;
- Expand the channel (widening only) all towards the east and maintain the existing west slope at the bridge.

This alternative provides the lowest overall cost for construction as related to pipe relocation, channel excavation, bridge works, road and rail works, and slope stability improvements. It also will avoid potential scheduling conflicts related to the bridge and Aqueduct sequencing, which could have added unnecessary complexity to the construction, and temporary stabilization work (and cost).

<u>Kildare and Cooks Creek Drainage Structures</u>

The expanded channel has been shifted towards the west between approximate Sta. 27+000 and 28+100 to maintain the existing east channel slope at the Cooks Creek Drain Drop Structure. Partial salvage of this drop structure results in significant cost savings to the project. The realignment causes further encroachment towards the Kildare Structure, which requires replacement regardless of the alignment, and has no negative impact on the overall costs.

<u>Springhill Area</u>

The channel expansion through the Springhill area is one of the most complex reaches along the entire Floodway. There are numerous components that are interrelated, and influence the channel geometry optimization, including:

- Overhead transmission lines with towers that currently encroach very close to the top edge of the existing channel. One of these lines (500 kV) provides power sales for Manitoba Hydro, and there are very high estimated costs associated with any modifications to the line or replacement of the towers ($12,000,000).
- Requirement for riprap erosion protection on the slopes adjacent to the towers.
- The municipal well system located on the east side of the channel provides water supply to the community of Birds Hill. A low permeability cut-off barrier was installed during the original Floodway construction on the east channel side slope to help maintain the water

supply. Accurate limits of this cut-off and the material type within it are not known, with only partial information available.
- The infrastructure for the Springhill Ski Hill.
- The PTH 59 North bridge.

Detailed geotechnical investigations and slope stability modeling of the area have not been completed. The original side slopes were constructed at 1V:3H, and reflect existing sand and gravel or till soil material within the side slopes. There has been no evidence of movement on these slopes related to the 1V:3H slopes in the 40± years since the original construction. The geotechnical design through this area has assumed that 1V:3H side slopes can be reestablished for the expansion geometry within the same limits as the current. Also, any channel widening towards the east causing closer encroachment to the municipal well supply may require the installation of a low permeability cut-off to minimize the potential impacts on the well system. Both of these assumptions have been made for this pre-design stage, and further investigation and evaluation will be required under final design.

The expansion geometry through this reach has been aligned to minimize the impacts to the numerous facilities in the area, including overhead utilities, municipal wells, and the ski hill. From Sta. 33+800 to 34+300 m, the expanded geometry assumes excavation on both sides of the channel base, although the channel has been shifted slightly towards the east. This was necessary to minimize the impact on the existing 500 kV transmission line, where the estimated cost for any work on the line is approximately $12,000,000. However, given the uncertainty in the potential for erosion at this location, this area is considered to require erosion protection in the form of riprap. The soil characteristics at this location and the possibility of retaining an effective coverage of vegetation will be considered in final design. It is conceivable that the riprap could be eliminated as a design feature in this final design assessment.

The excavation on the east slope of the channel is proposed to encroach towards the existing wells in that area, and a low permeability cut-off barrier contingency has been adopted through the slope to minimize the impact on the water supply wells. This is an area that will require further examination and refinement in final design.

North of Sta. 34+300, the channel has been shifted towards the west. The existing channel slope at the Springhill Ski Hill has not been modified, and no excavation has been assumed. This will minimize the impact on both the ski hill infrastructure as well as the apparent existing low permeability cut-off barrier below the hill. The expanded geometry becomes symmetrical about the channel near the PTH 59 North Bridge (approximate Sta. 35+000 m).

<u>Outlet Structure</u>

Downstream from the PTH 44 Bridge, the channel expansion has been shifted towards the east. This is consistent with the design requirements for the Outlet Structure, and maintains the existing west channel slope. This avoids the need for any excavation along the west side of the Floodway near the Outlet where such excavation could affect the archaeological interests that are known to exist there.

4.3.5 Erosion Control in the Expanded Floodway Channel

The Red River Floodway has performed well since its original construction that was completed in 1968. Flows from the Red River have been passed through the channel during 24 flood events up to June 2004. The Low Flow Channel has eroded in some areas, almost exclusively where the base of the Low Flow Channel is in lacustrine clay, plus some granular material zones. The other parts of the channel are primarily in till have essentially been erosion-free (although some reaches have suffered some bank sloughing) and have also not experienced sediment deposition to any substantial measurable extent (Preliminary Engineering Report: Appendix B Floodway Channel Pre-Design, 2004).

Erosion within the Floodway Channel is dependent upon the velocities and inherent shear stresses associated with flow of water through the channel. The expanded Floodway Channel will be subject to a revegetation plan.

Different Floodway Channel bed materials and vegetative **species** will offer different abilities to withstand shear stresses along the lining of the expanded Floodway Channel. Floodway Channel Pre-Design Appendix B presents the values of maximum permissible shear stresses for pre-design of the expanded channel.

The channel pre-design goal to achieve erosion control is based on limits of maximum permissible shear stresses with a further condition that the channel flow should not significantly exceed, during a 1 in 100 year flood, the level of shear stress that has been known to have occurred during the maximum flood on record for the Existing Floodway (i.e., that associated with the peak of the 1997 flood event). These criteria rely on the continued maintenance of vegetation coverage of the channel base and side slopes that will provide protection that exceeds the capability of the bare soil alone. A comprehensive vegetation plan intended to minimize erosion within the expanded Floodway Channel is described in Section 9 of Appendix B, Floodway Channel Pre-Design.

4.3.5.1 Revegetation Plan

The Manitoba Floodway Expansion Project is scheduled to begin in July, 2005. This will involve approximately 21 million square meters of exposed clay and glacial till subsoils that must be effectively vegetated. During this period, the Floodway must be available to function during spring events each year of construction as well as to provide drainage during spring and summer rainfall events. This will likely cause some damage to new vegetation still establishing root systems, requiring repair and re-seeding. In addition to the requirement for spring operation, it is possible that the Floodway use will be expanded after construction is complete to include summer operation.

A preliminary vegetation plan has been prepared without the benefit of a thorough vegetation and soils assessment of the current Floodway. The preliminary plan is based on assumptions dealing with construction methodology, initial observation of the existing plant cover, estimates of exposed soil, known capabilities of grass species, compressed time frames for planting, probabilities of surface damage from flood events during construction, and other unknowns regarding weather. It provides a comparison of vegetation types, it identifies critical issues with, and major differences between, vegetating the original Floodway and the re-constructed Floodway, and provides a preliminary assessment of the costs for the vegetation effort.

Planning vegetative treatment for the Floodway poses a number of challenges. With the exception of the Floodway base, most of the time that the vegetation will be exposed to ambient weather, including freeze-thaw cycles and soil moisture conditions. During most growing seasons, there will be a deficit between the available soil moisture and evapotranspiration. To be effective, vegetation has to not only survive the dry periods, it must prosper and produce sufficient ground cover to provide soil protection for the flood events that are likely to follow drought. The vegetation then needs to survive the flood event and inundation in sufficient condition to self-repair as necessary.

Based on the assessment of brome and alfalfa vegetation versus native plants, native grasses are the recommended choice for the Floodway side slopes and channel base revegetation. They have been selected for their superior ability to withstand summer submergence versus the alternatives, improved drought tolerance, superior erosion resistant qualities and the ability to generate significant biomass on degraded soils. For the Floodway and disposal embankments, native prairie grasses have capabilities that make them a strong alternative to introduced grasses. Mixes can be tailored to suit soil changes and moisture regimes on the Floodway. Native grasses will produce long-lived cover and generate forage more efficiently than can introduced grasses on the highly disturbed soils. Native grasses also offer the opportunity to include native prairie wildflowers to enhance the attractiveness to native pollinators, wildlife, and people. This possible enhancement to the revegetation plan has not been considered in the cost estimate, but could be accomplished during final design.

The proposed vegetation plan, at this stage of design, has been divided into three zones, as described below and as illustrated on Figure 4.3-7.

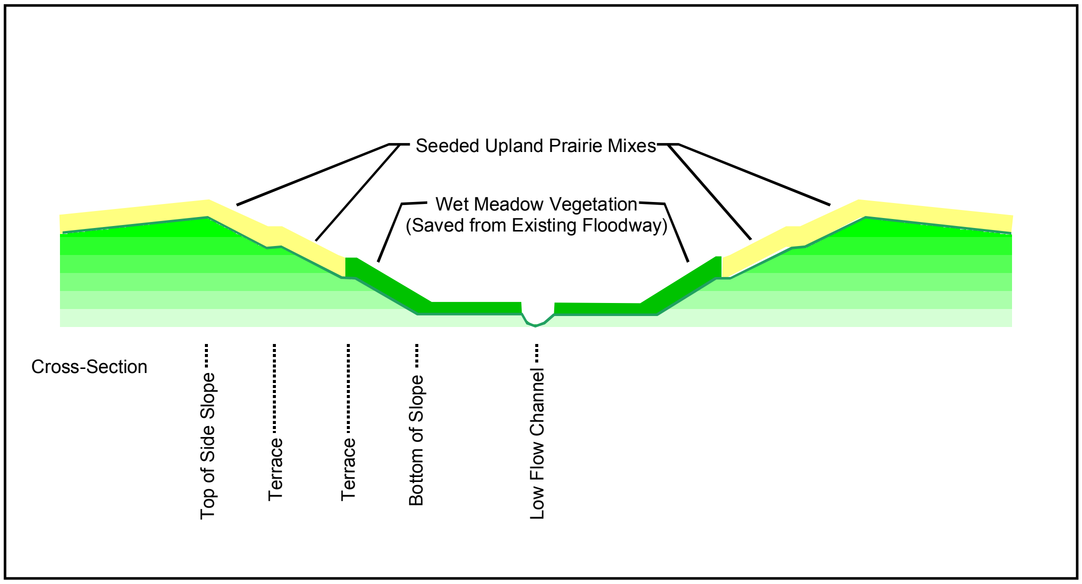


Figure 4.3-7
General Planting Concept for Revegetation Plan.

Zone 1: Disposal Embankment Upland Areas.

The vegetation plan considered two primary alternatives, one that is familiar to the general farm community, and one that is generally unfamiliar. Although the native grass alternative is particularly attractive for Floodway erosion control, the lack of experience with these grasses on farms in the local area has caused concern. It has been indicated by the MFEA and Manitoba Agriculture representatives, that local leaseholders may abandon their interest in harvesting hay if native grasses are selected for these upland areas. Brome and alfalfa, therefore, need to be considered as a required component of the vegetation plan in this zone. With appropriate maintenance, the different species can co-exist in adjacent areas. Alternatively, native grasses can be harvested later than introduced cool season grasses, so a beneficial harvest schedule can be devised whereby optimal harvests of both kinds of hay can be planned.

Zone 2: Channel Side Slopes.

On the side slopes and on the disposal embankments (to the extent acceptable by the local stakeholders) there will be an opportunity to create and maintain an upland prairie plant community. To do this successfully the remaining bromegrass and red fescue on the surface of the side slopes and embankments should be buried under the surface layer rather than saved as surface cover material. The agricultural soil between the existing disposal embankments and the right-of-Way limits should ideally be harvested and utilized to enhance the seedbed characteristics of the clay and till subsoils. The benefits of doing this are substantial in terms of early vegetation success and long-term tolerance to environmental stresses. Although it is recognized that the application of the topsoil will make the side slopes more vulnerable to erosion during construction prior to root establishment, its long-term benefits are significant. These soils have been farmed for many years with varying levels of management and it is to be expected that in addition to bromegrass rhizomes, there are thistle, red fescue rhizomes and other plant parts that are not desirable if moved in a live state with the harvested soil. Therefore it is recommended that a spray program be initiated to control the existing vegetation on these soils during late summer before they are to be excavated and moved. The most benign herbicide to use will be glyphosate (Roundup is one trade name) plus a broadleaf-specific herbicide where needed to control invasive species. The use of this “topsoil” will be subject to a more rigorous assessment of the logistics associated with capturing and re-using this material, as well as the cost and benefits associated with doing so. Preliminary costs for salvaging the topsoil have been addressed in Section 12.0. This will be assessed in greater detail at the final design stage.

Zone 3: Floodway Base.

The vegetation on the existing Floodway base is the result of the conditions that have occurred over the last 30 years. It is comprised of a mix of wet site species on wet sections, and drier site species where gravels form the bottom. As well, there are bare areas near Birds Hill perhaps due to a combination of excessive drainage and recreational vehicle traffic. The locations on the base that are well drained will receive a species mix similar to the side slopes to enhance the prospects of retaining a grass cover in these areas. Where the floor is wet, the best strategy will be to retain as much of the existing cover as possible rather than remove it and revegetate. This assumes that the willows within this area will be treated effectively. The vegetative survey will reveal in greater detail the species to be found there, which will affect the ultimate planting prescription. Exposed soil areas as the base will be increased as the Floodway is excavated to a greater width. There will be approximately 4.6 million square meters (1150 acres) of additional Floodway base created. The size of the area to vegetate will in itself dictate what methods are practical. The planting approach is likely to be a combination of seeding the entire area, and augmenting the seeding with a few vegetatively planted species that do not lend themselves to planting by seed. However, an effort will be required to obtain the wet meadow species seed by planned harvests of native stands from the southern Manitoba region. These species generally do not appear in the marketplace in the quantity needed to plant the acreage involved. Site preparation will also be needed on donor areas where transplant materials can be harvested, whether this is within Floodway, outside the disposal embankment areas, or off-site. These preparatory efforts must be initiated in 2004, and will require that the vegetation contract be let in time to allow the necessary preparations to begin.

The willow population on the Floodway base must be reduced to reclaim the efficiency of the Floodway discharge capacity. If this cannot be done, the willows will spread to occupy more of the area, including the new space in the expanded channel. The potential use of the Floodway for summer operation will result in the channel base being wet on a more consistent basis, further enhancing willow growth. There are four methods that can be employed to reduce the willow vigor: change the hydrologic regime on the base to a standing water wetland, herbicide treatment, mowing, and prescribed burning.

A full description of the revegetation Plan, including performance characteristics of selected plant species, is available in Preliminary Engineering Report Appendix B: Floodway Channel Pre-Design.

4.3.5.2 Low Flow Channel Design

The Low Flow Channel of the Existing Floodway has eroded, primarily in the portions of the channel that are cut through the lacustrine clays. The proposed treatment of the Low Flow Channel is to infill areas that have eroded, and to protect them from further erosion by placement of a riprap armour layer. The riprap and infilling of the Low Flow Channel would minimize any pooling effects that have occurred over the years. Riprapping and infilling of the Low Flow Channel to the original bed profile would require infilling most of the length of the channel and would be relatively costly. A modestly lower profile that is above its current condition is proposed to minimize costs. This profile does not lower the invert of the channel below that of the existing invert with the exception of a reach approximately 3 km in length at the upstream end of the Floodway. Even that short length would only be lowered from the existing level (which is also equal to the original design level) by 0.3 metres (1 ft). Figure 4.3-8 shows the profile of the proposed invert of the Low Flow Channel compared to the existing and the original design invert of the Low Flow Channel.

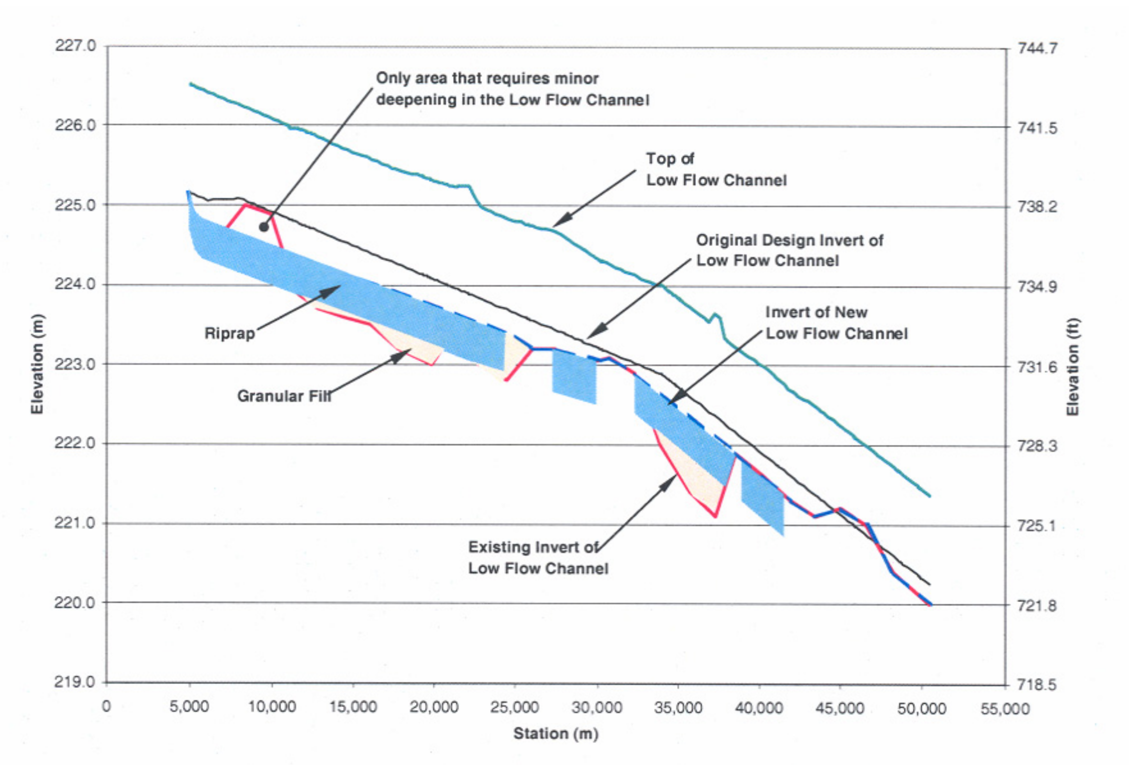


Figure 4.3-8
Proposed Invert Profile of the Low Flow Channel

Figure 4.3-8 also shows the locations where the proposed riprap armouring of the Low Flow Channel and where granular fill would be required. This action does not affect the selection of the overall design strategy for the channel. There have been questions regarding the increase in channel bed roughness that may be attributed to the proposed armouring in the Low Flow Channel, and the loss of hydraulic capacity in the Floodway that may result. This is not expected to affect the overall performance of the channel. Although the apparent roughness of the Low Flow Channel may be greater than that of the

Floodway Channel base, it is not believed to be greater than what has occurred in the Low Flow Channel due to the development over the years of significant growth of willows and other relatively large wetland vegetation along the sides. In fact, the overall roughness of the Low Flow Channel in its proposed condition after expansion is believed to be a reduction from the existing condition. Furthermore, the width of the Low Flow Channel is a small component of the Floodway Channel and any changes in that small part would have only minimal influence on the overall hydraulic performance. However, in future, it will be prudent to maintain the channel and prevent the development of vegetation that could adversely affect the hydraulic performance of the Floodway. The proposed Low Flow Channel is designed to convey 95% of the flows in the summer period from June 1st to October 15th without causing water to spill onto the base of the Floodway Channel, or in other words, without overtopping the top of the Low Flow Channel.

4.3.6 Construction of Expanded Floodway Channel

Although not fully defined, the construction schedule is assumed to roughly follow the timeline given below:

- Year 1 (2005) – July 1 to December 15 = 20% of required excavation.
- Year 2 (2006) – May 1 to December 15 = 30% of required excavation.
- Year 3 (2007) – May 1 to December 15 = 30% of required excavation.
- Year 4 (2008) – May 1 to December 15 = 20% of required excavation.

The fourth year is assigned a smaller portion of the overall excavation to provide a contingency for possible delays in excavations. The sediment and erosion control plan needs to be flexible for shorter/more intense construction (i.e., channel excavation reduced to 2 or 3 years total). The above schedule will be finalized during the next phase of design, and does not have a significant impact on the effectiveness of the sediment and erosion control measures performance.

Given that there is approximately 42 kilometres (26 miles) of required excavation within 28 months, on average it is estimated that approximately 1500 metres (1 mile) of channel will need to be completely excavated on a monthly basis. It is likely that the overall yearly length (between 9 and 12 kilometres [5.5 and 7.5 miles]) will be divided among a number of contractors, allowing each contractor to open approximately 500 metres (16,400 ft) of the existing channel for excavation on a monthly basis. This will result in at least 3 sections of the channel under construction at one time (on average), although it is more likely to be 6 crews with a 500-metre section per side of the channel. By **staging** the overall sequencing in this fashion, manageable units of the channel can be excavated at one time and then seeded without prolonging exposure of excavated slopes. This construction staging and the related erosion control logistics will be subject to further evaluation at the next stage of design to ensure that the excavation Contractor is not unduly restricted in his preferred sequencing methods.

The proposed construction sequence (contractor staging) for a typical year of excavation is illustrated on Figure 4.3-9. Limiting the amount of open excavation to discrete intervals allows for maximum and prompt re-vegetation of the channel. Actual contractor staging will need to be assessed and finalized before the tendering process, and could be accommodated in many ways depending on logistics and preferences by the general contractors; seeding contractors; final design considerations and owner preferences.

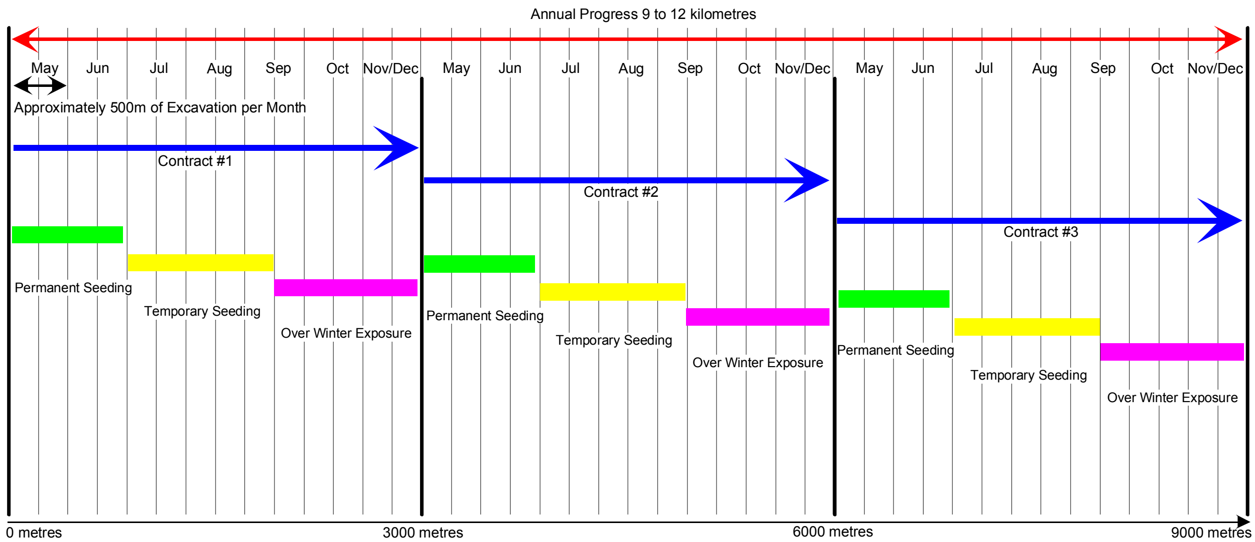
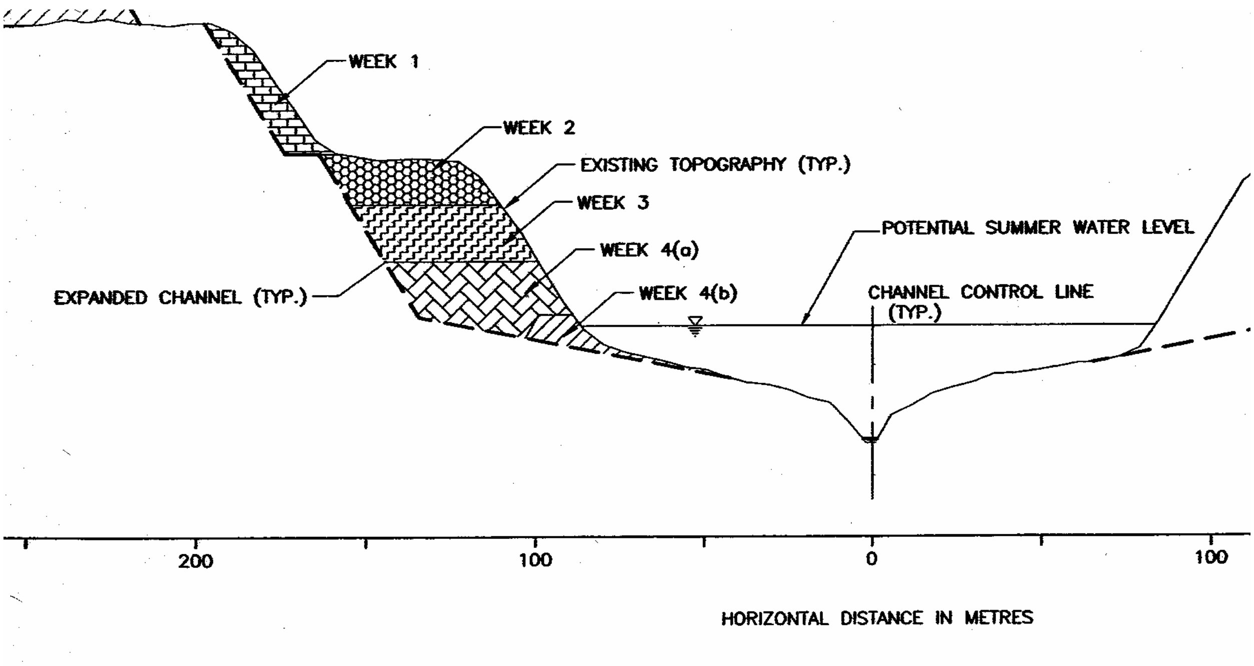


Figure 4.3-9
Conceptual Construction Sequencing

<u>Excavation Best Management Practices</u>

The management of the excavation within the given 500-metre section depends on which month the - excavation is completed and in what year. There will be slight differences between the management of the excavation in May-June versus October-November, and there will be differences between Year 1 and Year 4. The following preliminary guidelines are recommended for managing the excavation:

- Excavation should be completed from the top down not from the inside out. This will maximize the vegetation buffer below the excavation (see attached Figure 4.3-10).



Source: KGS/Acres/UMA (2004b)

Figure 4.3-10
Proposed Excavation Sequencing

- When excavating below 1:20 year summer Floodway level (years 2, 3 and 4) excavate from the outside in, and leave an earth plug until the end of the construction period (see Figure 4.3-10). This will maintain excavation in the dry, and allow for containment of internal sediment during storm runoff.
- Implement slope roughening techniques on exposed side slopes to limit erosion.

The above management strategies are anticipated to reduce erosion and sediment transport by more than 10 percent. Slope roughening alone can achieve 10% reduction in erosion, while maintaining the maximum amount of vegetation buffers will further reduce the delivery of sediment to the main Floodway conveyance channel. Maintaining vegetation strips between 3 and 20 metres can further reduce sediment delivery by approximately 20%. On this basis, implementing these BMPs will reduce the potential TSS concentrations by 20 to 30%. The greatest erosion will take place in the clay zones, which are the dominant soil type throughout the channel excavation. Based on the PDEA1 site investigations and material testing, it is assumed that the following soil characteristics are common for the clay soil over the channel length:

- Clay content is 50-60%.
- Silt content is 40-50%.
- Particles of size greater than silts are less than 1%.
- Organic material is essentially 0% except for the very top (0.5 to 1 metre) portion of the excavation. Organic soil may be stripped and stock piled for later use, but is not of significance in terms of the sediment and erosion controls.

The construction sediment and erosion plan for each 500-metre section is based on accepted design practice and is intended to be flexible by providing options for further reducing the delivery of sediment beyond the excavation limits. Estimates of effectiveness are given where they can be quantified.

Perimeter Controls

The best alternative for establishing and maintaining a perimeter around exposed areas is with the installation of silt fencing. A row of silt fences should be placed at the bottom of the existing channel within the existing permanent vegetation. As well, a row should also be placed on the outside perimeter of the excavation to minimize the transport of sediment beyond the construction site via the external ditches.

Intermediate Controls

A wide variety of sediment control techniques could be implemented to reduce sediment transport. A few are described below. Silt fencing could be installed parallel to the benched areas allowing at least 3 to 4 metres (10 to 13 ft) of buffer between the toe of the up slope and the line of the silt fence. Silt fences should not be installed directly on the slope as they are not robust enough to withstand the weight of trapped sediment behind them. Based on the silt content of the soil structure, it is estimated that "properly maintained" silt fences could remove up to 40 percent of sediment from runoff. Proper maintenance includes routine inspection and removal of sediment build up behind the fence after rainfall events. The construction of flow interceptor swales at regular intervals of 25 to 30 m (80 to 100 ft) cross-slope (parallel to contours) to capture or reduce the energy of the runoff is considered another low cost technique worth consideration. The interceptor swales could be used in conjunction with sediment barriers/filters at centralized down slope swale locations. The interceptor swales will run east and west for approximately 100 to 125 metres (320 to 410 ft) toward a central down slope swale. This scheme will effectively cover up to a 250-metre (820 ft) length of channel. The interceptor swales can be easily constructed with a Bulldozer with an adjustable blade to create a deep furrow into the side slope. Interceptor swales will not effectively trap or filter sediment, but they will minimize further erosion preventing deep rills or gullies. If interceptor swales are used in conjunction with centralized down slope swales, the outlet controls on the down slope could consist of a series of permeable sediment barriers, which may include the following techniques:

- Straw bale sediment barriers.
- Geosynthetic Dykes.
- Rockfill Check/Filter Dam.
- Permeable instream sediment barriers/flow control systems.

The system selected will depend on the cost, and the effectiveness of the technique or combination of techniques. An allowance for the outlet controls to minimize sediment delivery to the pilot channel has been considered in the cost estimate based on a spacing of one outlet swale every 250 metres.

The combination of interceptor swales with the outlet controls could effectively reduce the delivery of sediment to the extent required to meet surface water quality objectives (see Section 5.5).

The final design of the erosion control techniques will not differ significantly for the various sections of the Floodway from year to year. Where periodic inundation by summer flows in the Floodway is a threat, especially in the downstream sections of Year 2, 3, and 4, the methods employed will have to ensure that the runoff leaving the final outlet sediment control measure carries only a portion of the sediment eroded from the up slopes. The vegetation buffer between the toe of the slope and the pilot channel cannot be relied on to remove the final sediment load, because this sediment may be picked up and transported downstream during a more significant summer storm that produces flows and levels above the capacity of the pilot channel. Where the storms are more localized, the sediment buffering qualities of the channel base vegetation will be substantial.

As a minimum, the following measures are recommended to mitigate erosion and the transport of sediment beyond work areas:

- Excavation of the channel in discrete length and from the top down will allow flexibility for Contractor to reduce impacts to the overall construction).
- Maintenance of vegetation buffer where possible (assumed negligible cost to overall construction).
- Slope Roughening techniques
- Perimeter and intermediate (along the disposal embankment and Floodway benches) silt fences
- Flow interceptor swales at regular intervals (25 to 30 m [80 to 100 ft]) cross-slope (parallel to contours)
- Temporary seeding

The above measures are estimated to reduce erosion and sediment discharges by as much as 50% with proper maintenance. Allowances for maintenance have also been estimated as given below. Additionally, the centralized swales can be implemented with more rigorous outlet controls should this be warranted or necessary depending on the final definition of regulatory requirements for mitigating downstream effects. The following list of factors would contribute to the need to implement an sediment and erosion control plan with more stringent controls:

- Condensing the construction schedule and/or allowing greater areas of open excavation.
- Greater regulatory requirements and/or reduced capacity of the Red River to dilute sediment laden discharges from the Floodway to meet suggested guidelines.

These measures will be described in more detail in the EPP for channel construction.

<u>Recommended Construction Sequence</u>

Four alternative construction sequences were developed and investigated from the perspective of the hydraulic performance and risk of erosion in the Floodway each spring during the construction period. These four alternative sequences include:

- Scheme A – Excavate four consecutive segments of the channel starting at the downstream (north) end of the Floodway and working upstream (south).
- Scheme B – Excavate four consecutive segments of the channel starting at the upstream (south) end of the Floodway and working downstream (north).
- Scheme C – Excavate four segments of the channel in a random order.
- Scheme D – Excavate in four vertical stages along the entire Floodway length.

A risk assessment was carried out for each option. This assessment was based on the surface area of the Floodway that would be at risk of erosion during the construction period. Since construction will be ongoing throughout the summer and late into the fall of each year, there will be limited time available for the establishment of a permanent vegetative cover in the excavated areas. Therefore, after each year of excavation, a portion of the Floodway Channel will be left over the winter with either an immature, temporary or no vegetative growth at all. The lack of a stable mature vegetative cover in the Floodway allows for the possibility of erosion in the channel during a potential spring flood.

The risk assessment provides methods to (1) compare the risk of channel erosion for each sequence at each stage of construction, and (2) compare the overall of the risk of channel erosion for each scheme.

An erosion risk factor (ERF) was used to provide a single quantitative number to compare the risk of erosion for each of the sequencing schemes. The ERF was calculated by taking the summation of the average annual areas at risk at each stage of construction. Table 4.3-1 summarizes the average annual area at risk of erosion. The ERF for each sequencing scheme is also shown in Table 4.3-1.

Table 4.3-1
Comparison of Erosion Risk Assessment Factors for Considered Construction Schemes

Stage of Construction	Average Annual Area at Risk of Erosion (m^2)			
	Scheme A	Scheme B	Scheme C	Scheme D
April 2006	5 985	0	0	0
April 2007	20 932	26	7 137	0
April 2008	12 378	12 646	753	15 421
April 2009	10 109	36 522	52 324	98 940
Erosion Risk Factor	49 404	49 195	60 214	114 361

Based on this risk assessment, the preferred construction sequencing scheme is considered to involve excavation of the Floodway Channel in four segments starting at the upstream (south) end of the Floodway and working downstream (north).

An assessment was carried out to quantify the magnitude of sediment that would be eroded from the Floodway Channel and released into the Red River at Lockport during the construction period. This assessment was carried out for both sequence Schemes A and B. The USACE's HEC-6 sediment transport analysis software was used to quantify the magnitude of sediment that could potentially be eroded from the Floodway Channel during construction if an spring flood were to occur that requires use of the Floodway. The results of this assessment suggest that the increase in sediment concentration that could occur during construction result in a minimal impact on sediment concentration in the Red River during the flood events (see Section 5.5).

From an overall perspective, the preferred sequence of construction has been deemed to be Scheme B – progression sequentially from upstream to downstream. The construction in the Low Flow Channel should proceed in advance of the excavation in the main channel, as much as possible. Winter work in the Low Flow Channel is preferred, as the channel will have the least flow during that period.

The original schedule for the Floodway Expansion developed in the SAFE Study adopted a duration of 4 construction seasons. That was based on an estimated volume of excavation of approximately 35,000,000 m^3 (46,000 yd^3) and the premise that the work should be done at a rate that could be easily managed by the excavation industry in Manitoba. Since that time, the concept for the Floodway Expansion has evolved and the excavation volume has reduced to 21,000,000 m^3 (27,300,000 yd^3). It is possible that this work could be executed in much less than four years, and possibly in as little as two seasons.

Discussion of these issues is required prior to the development of the overall project construction schedule. It should be noted that although quantitive analyses for a 2 or 3 year excavation schedule, it has been concluded that the preferred sequence of excavation would also be in segments from upstream to downstream.

4.4 INLET CONTROL STRUCTURE UPGRADES

Details pertaining to Floodway Inlet Control Structure Upgrades are derived from the most current design information as of June 2004. This information is contained in the document Preliminary Engineering Report: Appendix C-Inlet Control Structure Pre-Design (SNC/Wardrop 2004a).

The Floodway Inlet Control Structure is situated in the Red River just downstream from the inlet to the Floodway Channel. The structure consists of reinforced concrete abutments, end piers and a central pier with two large submersible sector gates, each 34.29 metres (112.5 ft) wide. The gates normally are in the submerged position, with about 2.44 metres (8 ft) of water over them in the summer months. Under these conditions the crest of the Channel Inlet at el. 228.6 metres (750 ft) permits flows to enter the Floodway when the Red River discharge exceeds 850 m^3/s (30,000 cfs). As the natural river stage increases above 850 m^3/s (30,000 cfs) there is a division in flow between the Floodway and the Red River. The purpose of the Floodway Inlet Control Structure is to regulate the division in flow between the Floodway and the Red River. The gates in the Floodway Inlet Control Structure are generally operated so as to maintain an upstream water surface elevation at the level that would have occurred under natural conditions (see Section 5.3 Operating Rules). The structure is founded on limestone bedrock.

Figure 4.4-1 provides an overall perspective of the inlet control structure and its relationship to the floodway. The general arrangement of the structure is shown in Figures 4.4-2 and 4.4-3.

A number of measures were identified as work required at the Inlet Control Structure to be incorporated into the Floodway Expansion Project at this design stage (KGS/Acres/UMA 2004a). These include:

- Measures to improve redundancy or reliability of the Inlet Control Gates
- Improvements to the erosion control measures on the embankments immediately adjacent to the Inlet Control Structure
- Improvements to existing components of the gate operating system

Figure 4.4-1
Aerial view of Inlet Control Structure looking upstream along the Red River. The Floodway is shown in operation during April 2004 spring flooding. Floodway channel is in background.

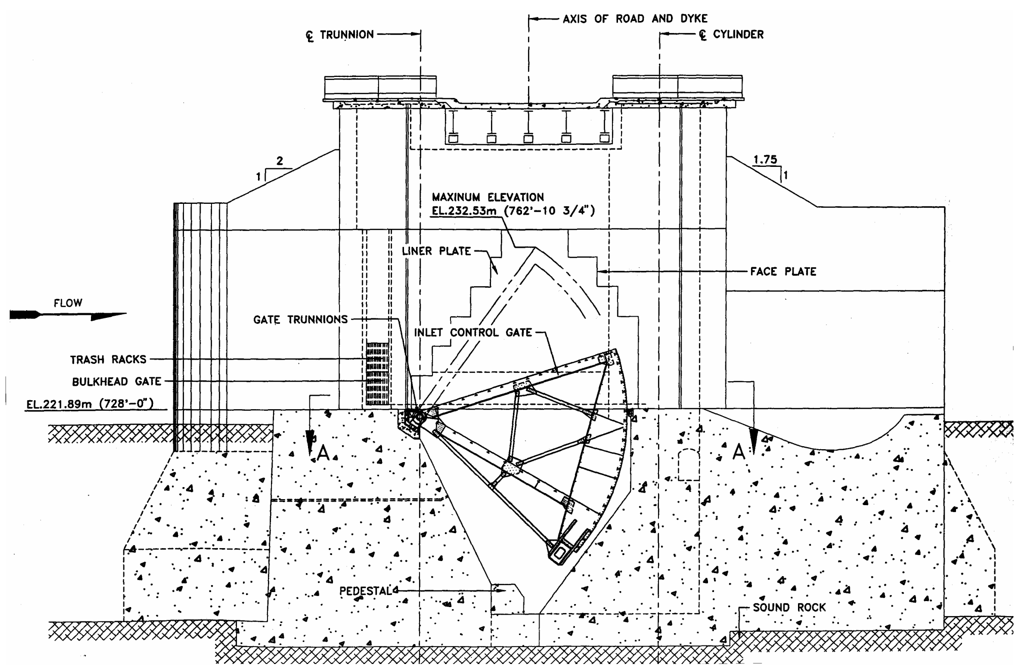


Figure 4.4-2
Floodway Inlet Control Structure Section at end view

Figure 4.4-3
Floodway Inlet Control Structure and earthfill embankments as viewed from the north face upstream on the Red River

4.4.1 Description of Measures to Improve Reliability and Redundancy

The level of redundant features incorporated in the Inlet gate design and the reliability of operation have been reviewed, based on previous experience, combined with a failure modes and risk analysis. Based on the findings of the review, a number of measures to add redundancy and improve reliability have been identified and are recommended to be incorporated into the final design. Although a wide range of options, including backup gates, were considered in the evaluation, the cost of backup gates has not be included in the cost estimate at this time. The need for incorporation of redundant or “backup” gates is currently being assessed in greater detail before a final decision is made.

Measures currently incorporated into the preliminary design, and planned for final design, are summarized below.

Hoists - Power Supply

Based on potential weaknesses in the existing backup power supplies from Manitoba Hydro's system, more permanent provisions are recommended for “hooking up” a standby generator. These include:

- Identification of suitable alternative, portable standby generators and plans for mobilization during the early stages of a major flood.
- Installation of permanent facilities for the electrical distribution to allow the straightforward connection of the alternate power supply via an emergency generator.

- Installation of new starters or supply of individual spare starters to avoid the potential for extended outages of obsolete equipment, in the event of electrical failures.

It is understood that Manitoba Hydro will also be making modifications to their power supplies to provide an independent supply from the east side of the Inlet Control Structure. This modification should be implemented as soon as possible.

Hoists - Mechanical

With the exception of the items identified below, the existing hydraulic pressure supply systems and piping are in good condition. A number of relatively low cost improvements are recommended to enhance the reliability of the hydraulic system and provide further system redundancy.

Protection Against Oil Contamination

The installation of improved hydraulic fluid filtering on the discharge of each pumping system is recommended to provide increased protection against system contamination. Contamination of the system with metal particulate could potentially cause failure of both systems.

Provision of Backup Pressure Supply System

The provision of isolation valves and additional feed connections for each cylinder will provide protection against system piping or pressure supply equipment failure. Two trailer mounted units are recommended to match the arrangement of the existing system and to maintain a practical equipment system for portability. Suitable indoor storage should be provided for the equipment and it should be tested regularly.

Provision of Redundant Hydraulic Power Supply

The provision of connections near the power units on the existing hydraulic power supplies are recommended to permit the proposed portable systems to provide backup to the existing pressure supply systems. The existing redundant tractor driven hydraulic power system installed on the East Gate system would also remain.

Provision of Spare Parts

The original system hydraulic equipment including control valves, relief valves, pumps and associated components have remained generally unchanged since the time of installation. The procurement of a stock of spares (one of each of the control valves, relief valves, pumps, etc) is recommended to permit immediate access to the equipment in case of failure.

Fire Protection

The mechanical room contains the motor control centre, air compressors and the hydraulic pressure supply systems, oil storage, and hydraulic controls used for the operation of the gates. While the room is generally constructed of non-combustible materials, there are combustible and flammable materials, including the hydraulic oil, in the space. Loss of the equipment in the room due to fire could render the gates inoperative. It is planned that this risk will be mitigated with improved fire protection.

Inlet Control Gate Buoyancy

Reliability of the Inlet Control Gates is affected by hydraulic dynamic forces that can cause an opening or closing tendency. The results of the physical model indicate that there is an opening tendency (i.e. the gates pressurized internally and are "buoyant" by design, or tend to open/raise without assistance from the hoists) over a range of operation. The opening tendency was found to occur over the most critical range, with the gates near the fully raised position. The reliability can, however, be increased if the chance of "faildown" or closing failure modes can be reduced by maintaining the opening tendency over the full range of gate position, and if the opening tendency is controllable.

Alternatives for increasing the opening tendency could include either fixed or variable buoyancy. It was concluded that the extent of buoyancy should be controlled and be variable as required. The recommended means of providing the buoyancy would be the installation of air lift bags that would be permanently installed in the structure of the gates.

Variable buoyancy was considered to be a practical, low cost option, which would be provided by the installation of a number of airlift bags inside each gate. The maximum buoyant forces of the airlift bags would be selected to create an opening moment exceeding the closing moment by at least 25%, including an allowance for friction. As an initial concept, the airlift bags proposed for use would be similar to those used for underwater recovery activities and would be specifically designed for this application. The bags would be permanently installed and "attached" to the main structural gate members to obtain a maximum practical opening moment. A pressure regulator on each supply line would permit the air pressure in each "bag" to be maintained at a constant pressure. A manual valve on each line would be used to relieve air from each bank. The existing compressed air system is proposed as the source of compressed air for this system. It is estimated that the bags could be fully inflated in about two hours using this compressor.

4.4.2 Improvement to Existing Components

Necessary improvements that have been identified in previous investigations include:

- Repair or replacement of the corroded piston rods.
- Replacement of the corroded cylinder cross head guide surfaces.
- Repair or replacement of corroded anchors for the cylinder crosshead guides.

It is recommended that these be completed as soon as practical as a part of the final design.

Spare Parts

A stock of spare parts should be maintained for critical components.

4.4.3 Erosion Protection in the Zone of Influence of the Inlet Control Structure

The earth fill embankments associated with the Floodway Inlet Control Structure are shown in Figure 4.4-4. Erosion and bottom scour in the protective apron area immediately downstream of the Floodway Inlet Control Structure resulted in major remediation to the downstream apron and to the adjacent toe of the abutment embankments in 2000. Some remediation of the downstream slope protection that was eroded in 1997 by the relatively high velocity return currents was carried out at the same time.

Since that time, concerns have been expressed regarding the adequacy of the upstream and downstream slope protection in the area of the abutments. This concern stems from the findings of previous limited site investigations and the knowledge that design criteria for riprap and filter design have changed since the Floodway Inlet Control Structure was constructed. As a result, an investigation of the adequacy of the slope protection was included in the scope of the present studies.

The hydraulic conditions at the entrance to the Floodway Inlet Control Structure and at the exit were assessed from a perspective of erosion protection requirements to meet the operating needs for the expanded Floodway system and to withstand extreme flood events. Erosion protection requirements for the upstream and downstream slopes of the embankments adjacent to the Floodway Inlet Control Structure were assessed for the conditions corresponding to operation of the Floodway within the design criteria set out for the Floodway Expansion Project.

East and West Dyke - Upstream Protection

For conditions in which the upstream water levels exceed the top elevation of the Wing walls, El. 235.0 m (771.0 ft), the slopes can be exposed to high entrance velocities, parallel and transverse to the flow, as well as wave action. Outside the limits of the Wing wall, the upstream face of the East Dyke and West Dyke will be exposed to wave action at all floodway entrance water levels.

Based on the riprap assessment, upgrades to the erosion protection are planned on the upstream face of the East and West Dyke at the Floodway Inlet Control Structure. This zone is deficient in size in an area adjacent to the Floodway Inlet Control Structure piers.

The recommended upstream riprap remediation locations are shown on Figure 4.4-4. The treatment will extend for a distance of 5 m (16.4 ft) from the face of the pier, where the size of the riprap is inadequate. The existing riprap and bedding material will be removed through to the embankment shell to provide sufficient depth for placement of the new riprap and a new geotextile filter.

For the remaining area, to the end of the Wing walls, the material will be removed and be replaced with riprap, which is compliant with the current standards. Following removal of oversize riprap, the filter material will be replaced and a geotextile filter cloth will be installed before placement of the riprap.

Figure 4.4-4
Floodway Inlet Control Structure and Earthfill embankments view from the west side of the Red River

East Dyke and West Dyke Slope Protection Downstream Face

The existing erosion protection on the downstream face of the East Dyke and the West Dyke at the Floodway Inlet Control Structure is deficient in thickness and gradation. During the 1997 Flood the slope protection downstream of the Wing walls was undermined locally and failed as a result of high velocities and downstream eddies.

Significant quantities of riprap were placed on the downstream face of the slope to prevent the possibility of progressive failure of the slope at that time. A major remediation of the downstream apron erosion and slope protection at the Floodway Inlet Control Structure was completed in January 2000. The remediation work consisted of reconstruction of the subgrade to support rows of concrete-filled fabriform bags.

The existing erosion protection on the downstream face of the East Dyke and the West Dyke at the Floodway Inlet Control Structure was checked against the riprap requirements associated with the velocities corresponding to maximum expected flow conditions through the Inlet Control Structure.

Using the same criteria as the upstream slope protection, excluding wave action the riprap was found not to meet current standards.

Based on this assessment, the existing riprap above the abutment walls will be removed and replaced with a geotextile filter under a riprap conforming to current design standards. The riprap beyond the walls will be extended as to provide additional protection for extreme high tailwater conditions.

4.4.4 Dam Safety Reveiw

A Dam Safety Investigation was based primarily on the Canadian Dam Association, Dam Safety Guidelines. The investigation covered the area from the TransCanada Highway to the West Dyke. A desktop review, from a dam safety perspective, of the West Dyke design (completed by others) was also undertaken.

4.4.5 Floodway Discharge Facilities

- The Floodway discharge facilities include the Floodway Channel and the Inlet Control Structure. The overall arrangement

4.5 FLOODWAY OUTLET UPGRADES

Details pertaining to Floodway Outlet Upgrades are derived from the most current design information as of June 2004. This information is contained in the document "Preliminary Engineering Report: Appendix D Outlet Drainage Structures and Syphons Pre-Design".

The Floodway Outlet Structure and Channel will require considerable modification in order to accommodate the increased design discharge of the expanded Floodway. Portions of the existing Outlet Structure will be incorporated into the new structure to reduce capital costs. The channel will be designed to convey the flow of water into the Red River at an acceptable velocity with a minimal amount of excavation.

- The potential for erosion along the riverbanks downstream of the Outlet Structure was assessed for flood events up to the 700 year event. Erosion protection/stability enhancement options were developed and cost estimates were prepared for the various mitigation options.
- The conceptual design of the Outlet Structure and Channel was refined to enhance the efficiency of the Outlet Structure, while maintaining satisfactory energy dissipation of flows, and minimizing potential erosion on the river and Outlet Channel. This task was performed using a state of the art numerical model.
- A hydraulic (physical) model of the preferred design of the Outlet Structure and Channel was constructed and used to confirm the numerical model results and increase the confidence level for further refinement of the design.
- Pre-design drawings, cost estimates, and a construction schedule were prepared for the Outlet Structure and Channel.

4.5.1 Design Assumptions

Various assumptions and alternatives considered for the Floodway Outlet Structure include:

- The existing structure is in acceptable condition and portions of the structure will be utilized in the new structure. The expanded Outlet Structure will incorporate the rollway concrete and portions of the west wall. The rollway and apron slab will extend east of the existing structure.
- New rollway surface over the existing rollway will be post-tensioned to the original structure. Alternatively, the new rollway layer may be anchored to the existing rollway by dowelling new reinforcing into the old rollway portion. This alternative may provide some costs savings and shall be considered during the final design.
- The length of the upstream channel facing wall will be increased to contain the fill of the raised embankments with slopes assumed similar to the existing structure (1v:2h).
- Nominal bedrock elevation outside the original excavation is assumed to vary between el. 217.5 metres (713.6 ft) and el. 219.5 metres (720.1 ft), as defined by the boreholes.
- A system of 150 mm (6 inch) diameter drains in the apron slab similar to the drains in the existing apron slab shall be provided to relieve the uplift pressures underneath the Outlet Structure. Monitoring and maintenance of the system of drains would be difficult, and therefore the effectiveness of the drains shall be neglected.
- With the system of drains described above, double corrosion protected post-tensioned rock anchors shall be used to anchor the apron slab against unbalanced uplift and vertical water loads. It is unlikely that any further reduction in the number of post-tensioned anchors is possible as this would impact on the spacing of the anchors and thereby impact on the structural capacity of the slab. Alternatively, designs such as increased slab thickness or the use of concrete piles bonded to rock to counter the unbalanced forces would significantly increased the capital cost of the structure. The option of using larger size drains for relieving the uplift pressures, which could be monitored, may reduce or eliminate the need for post-tensioned anchors. This alternative shall be considered during the final design.

The preferred design concept for the Outlet Structure was identified by the design consultant as a 100 metres (328.1 ft) wide structure, complete with energy dissipation appurtenances. This concept was selected early in the second iteration of design in order to provide sufficient time for the construction and testing of a physical model. Numerical analyses showed this option to perform the best, and the physical model also performed well, verifying the selected design.

However, additional numerical and cost analyses undertaken since the Iteration 2 design cycle indicate further economies in structure design may be possible. These economies would be realized by building a smaller structure, and accepting that the reduced hydraulic performance of the structure may lead to a greater risk of channel degradation and long term maintenance expenses. Since the Iteration 3 design, design consultants have been given the opportunity to more rigorously test the performance of the numerical model against actual data collected during the physical model tests. The very good match obtained provides additional confidence that the model can be used for this advanced design support. With this gain in confidence, it is possible to use the numerical model to assess more aggressive outlet structure designs. However, this must be done in consultation with MFEA to ensure the level of risk posed by the various alternatives is acceptable to the Authority. With this in mind, it is recommended that possible modifications to the design be considered prior to final design (within PDEA2).

The objective of these modifications would be to reduce the cost of the Outlet Structure without jeopardizing its intended performance. It is therefore proposed that the following modifications to the Outlet Structure be numerically analyzed and, if favorable, the preferred design be tested in the hydraulic model:

- Assess impacts on the upstream Floodway Channel for reduced structure width of 80 and 90 m (260 TO 295 ft). There is a concern that these narrower widths may result in unacceptable shear stresses on the bed under smaller flood events. This analysis would be undertaken utilizing the final HEC-RAS model developed in the PDEA2 channel studies.
- Assuming bed shear stresses are acceptable, review the performance of the 80 metres option under 1:200, 1:500, and 1:700 flood events with average and low Assiniboine River flows. If this proves to be unacceptable, review the 90 metres (295 ft) option under similar flow conditions.
- Reduce the upstream to downstream length of the stilling basin. This would involve moving the end sill and possibly the baffle blocks a few meters upstream. The resulting stilling action would not be quite as good as the Base Case Design and exit velocities from the basin would be slightly greater. However, it is noted that from test results to date that the exit velocities are below that which would likely result in scouring of the downstream bedrock.
- Increase the approach depth to the structure down to bedrock. The Floodway Channel upstream of the Outlet Structure consists of glacial till overlying the limestone bedrock. For the Base Case Design the approach depth relative to the crest of the ogee is quite shallow. For this design the glacial till would have to be protected from scour using riprap. By deepening the channel to bedrock the requirement for protective riprap would be significantly reduced. A deeper approach would have the additional benefit of increasing the discharge coefficient of the structure by 2 to 3 percent.
- Increased spacing between baffles.
- Elimination of one row of baffles, or elimination of chute blocks.

Further cost reductions could be achieved in the final design as a result of more detailed engineering of the structure. The following is a list of items that should be investigated but should not limit the final design process:

- Height of backfill adjacent to the downstream wing walls could be reduced. This would result in a reduction of the applied loads which would result in a reduction of concrete quantity.
- Optimization of the wing wall cross-sections could also result in a reduction in concrete quantity and cost.
- Potential utilization of more components of the existing structure specifically the modifications to the west wing wall could result in cost savings.
- Surface preparation and the use of passive dowels could reduce the anchoring costs for the new rollway surface over the existing rollway.

Figures 4.5-1 and 4.5-2 present the pre-design layout of the proposed Outlet and related structures.

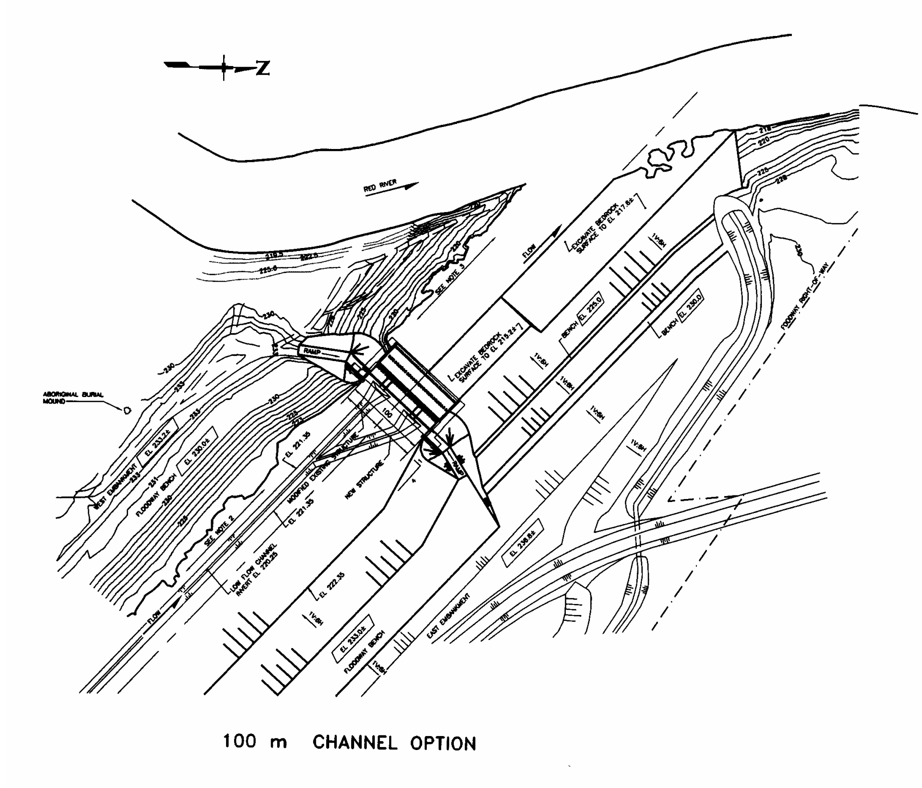


Figure 4.5-1
Floodway Outlet General Arrangement

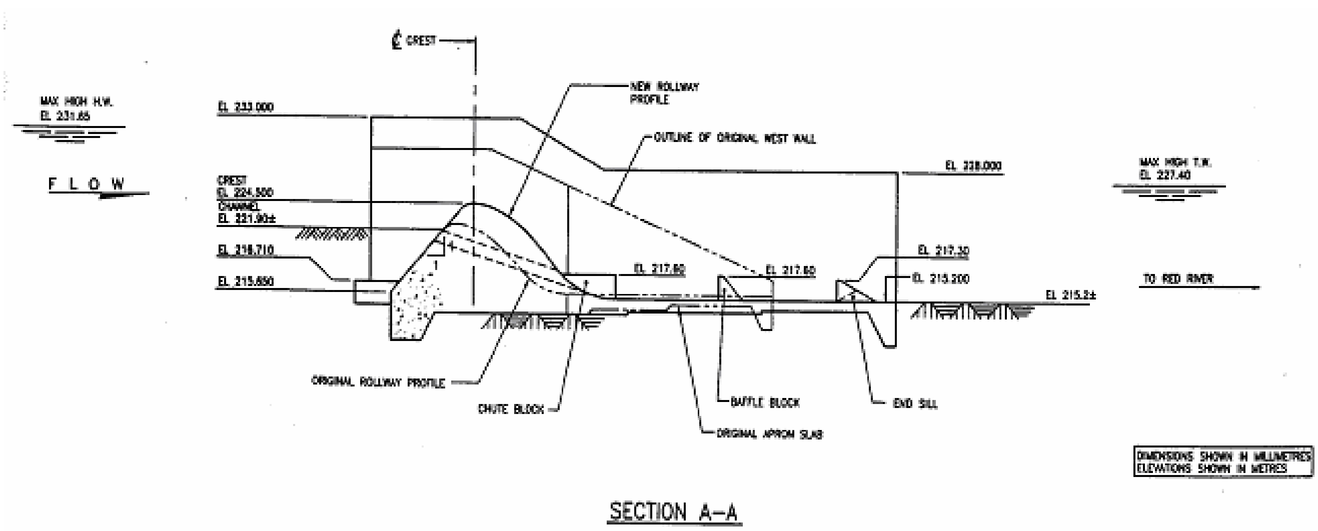


Figure 4.5-2
Floodway Outlet Spillway General Arrangement

The following information presents key pre-design hydraulic design criteria for the Outlet Structure. More detail is provided in Report (Preliminary Engineering Report: Appendix D Outlet Drainage Structures and Syphons Pre-Design).

Design Discharge

- Design Discharge = 3960 m^3/s (140,000 cfs).

Design Water Levels

- 700 year event
  - Design headwater level (energy level) el 231.65 metres (760 ft)
  - Design Tailwater level = 227.4 metres (746 ft).

- During Construction
  - Phase 1
    - Headwater level 222.9 metres (731.3 ft) (based on 1:5 yr summer event).
    - Tailwater level 221.5 metres (726.7 ft) (based on 1:20 yr summer event, Red River Water Level).
  - Phase 2
    - Headwater level 224.9 metres (737.9 ft) (based on 1:5 yr summer event).
    - Tailwater level 221.5 metres (726.7 ft) (based on 1:20 yr summer event, Red River Water Level).

Stilling Basin / Apron

- The downstream wing walls shall be of sufficient height to fully contain the hydraulic jump, thereby preventing the formation of strong lateral side currents that may draw material into the basin/apron of the structure. This criteria could be relaxed, however, there is a risk of damage to the structure through abrasion.
- The basin length shall be designed to contain the hydraulic jump.
- The downstream velocities shall not exceed:
  - 1.0 m/s (3.3 ft/s) over unprotected grassed slopes within the Outlet Channel
  - 3.0 m/s (10 ft/s) over riprap protected slopes
  - 5.0 m/s (16 ft/s) over bedrock

  The limit of velocity over bedrock is based on experience/observations with the existing structures, where erosion occurred in 1997 with velocities of about 8 m/s (26.2 ft/sec) (280 cfs).

Tailwater Stage-Discharge Rating Curve

- The tailwater level is variable and depends on the Floodway discharge and total Red River discharge downstream of the Floodway Outlet.

The length of the structure (upstream face of rollway to downstream edge of apron slab) is 60.2 metres (197.5 ft). The crest of the rollway is 224.5 metres (736.5 ft), with chute blocks on the downstream side of rollway, baffle blocks on the apron slab and an end sill at the end of the apron slab.

- The new rollway formed over the existing rollway was designed against sliding and flotation due to self weight of rollway, and hydrostatic and uplift head. Post-tensioned anchors were added to meet the design criteria. Alternatively, surface preparation of the contact surface and the use of passive dowels could be incorporated in the final design to eliminate the post-tensioned anchors.
- The concrete volume in the apron slab was minimized by the use of post-tensioned anchors. The slab is designed for uplift, varying from headwater at the upstream face to tailwater at the downstream of the slab. Alternatively, the apron slab could be designed as a gravity structure eliminating the post-tensioned anchors but would significantly increase the cost of the structure.
- The baffle blocks are anchored to bedrock to resist hydrodynamic and impact forces.
- The new training walls are sized and designed based on stability and structural requirements of the design criteria.
- Upstream facing west walls are modified making the best use of the existing structure, and strengthened using counterfort walls to increase the wall capacity to contain the fill of the higher embankment.
- The embankments have been designed with a semi-pervious core, filter zones rockfill shell, and riprap cover. The semi-pervious core may consist of a compacted till or silty sand.
- The Outlet Channel slopes have been designed to meet the design criteria, and to minimize the amount of excavation.

4.5.2 Outlet Construction

A preliminary construction sequence has been prepared to accomplish the proposed modifications of the Floodway Outlet Structure and Channel. To minimize winter construction and the added costs for such items as heating and hoarding the construction sequence will be performed over two years. The construction could be performed over one year but would increase the construction cost.

The two-year construction sequence consists of three construction phases. Each phase of the construction is described in the Report (Preliminary Engineering Report: Appendix D Outlet Drainage Structures and Syphons Pre-Design [KGS/Acres/UMA 2004c]).

Phase 1 will be performed in the summer of year 1 after passing of the spring flood. The existing structure will remain operational during this construction phase in order to pass a summer flood through the Floodway Channel. The resulting maximum headwater level is 222.9 metres (731.3 ft). A 1:20 year Red River summer flood event was assumed during the construction sequence resulting in a tailwater level of 221.5 metres (726.7 ft).

Phase 2 will be performed in the summer of year 2 after passing the spring flood. The new east portion of the rollway will remain operation during this construction Phase in order to pass a summer flood through the Floodway Channel.

Phase 3 will be performed in the late fall to mid-winter of year 2 following the completion of Phase 2 work. Minimal flows are expected during this time of year and would be dealt with by sandbag dikes and minimal **dewatering**.

4.6 BRIDGES

Details pertaining to Bridge Designs are derived from the most current information as of June 2004. This information is contained in the document "Preliminary Engineering Report: Appendix A Bridges and Transportation Pre-Design" (Dillon/NDLea 2004).

The bridge structures requiring modification under an expanded Floodway Channel scenario include:

Highway Bridges:

1. St. Mary's Road
2. Provincial Trunk Highway (PTH) 59 South – Southbound (S/B) Structure
3. Provincial Trunk Highway (PTH) 59 South – Northbound (N /B) Structure
4. Trans-Canada Highway (TCH) No. 1 East
5. PTH 15
6. PTH 59 North
7. PTH 44

Railway Bridges:

1. CPR Emerson
2. CNR Sprague
3. GWWD Railway
4. CNR Redditt
5. CPR Keewatin
6. Central Manitoba Railway (CEMR) Pine Falls

The bridge and transport consultants have reviewed costs for modifications to these bridges and conducted traffic evaluations for each floodway crossing. The consultants have also conducted site condition assessments at each bridge, and reviewed the land use in the vicinity of each structure, and adjacent infrastructure that may be impacted during bridge modifications.

Table 4.6.1
Summary of Bridge Condition Assessment Reports

Bridge Location	Condition of Deck	
	Estimated Remaining Life	Recommended Remedial Action[2]
1. St. Mary's Road	4 – 6 yrs	Deck Replacement
2. PTH 59S S/B	5 – 8 yrs	**
3. PTH 59S N/B	Bridge Constructed in 1996/97 and Meets Current Codes	
4. TCH No. 1E	5 – 8 yrs	**
5. PTH 15	5 yrs	Deck Replacement
6. PTH 59N	10 yrs	Deck Replacement
7. PTH 44	15 – 18 yrs	Deck Replacement

Notes: 1. Substructure rating is not evaluated.
2. From Bridge Condition Assessment Report

4.6.1 Highway Bridges

Life cycle cost analyses for the highway bridges were conducted, assuming a service life of 50 years for all existing highway bridge structures, except the PTH 59S N/B structure constructed in 1997, which has a service life of 75 years.

4.6.1.1 Highway Bridges Pre-Design Process

Pre-Design of Highway Bridge crossings involved a review of Bridge Condition Assessment Reports. It was determined that the condition of all bridge decks required either replacement or major rehabilitation. In addition, the pre-cast, prestressed concrete girders for all existing highway bridges (except PTH 59S Northbound Structure, constructed in 1997) are deficient in shear capacity to carry the proposed design live loads. An evaluation of Level of Service for each Floodway crossing was also conducted. This evaluation took into consideration the existing and projected traffic volumes, roadway functional classification/design criteria, recommended design criteria and collision analysis.

Further analyses indicated that there are major structural deficiencies in the strength of the existing piers and abutments for existing highway bridges. The Channel consultants determined that raising all bridge structures above the 1:700 year water level provided significant hydraulic and channel cost benefits, therefore all bridge concepts developed were subsequently based on raising and lengthening the bridges.

While initial design iteration work considered retrofit and replacement alternatives for each bridge, further design iteration life-cycle analyses concluded that all highway bridges, with the exception of PTH 59S northbound structure, should be replaced. The PTH 59S northbound structure would be raised and lengthened.

4.6.1.2 Highway Bridges Design Criteria Summary

Several general design criteria were applied in the preliminary highway bridges preliminary design process. The general design data and criteria are as follows:

Design Codes

- All bridge modifications including retrofits and replacvements are designed in accordance with AASHTO LRFD Bridge Design Spefication (latest edition) plus interims.
- MTGS Tansportation Planning Manual (Basic Design Standards) for geometrics for affected roadways, with secondary reference to the Transportation Association of Canada (TAC) Geometric Design Guide for Canadian Roads.

Design Loading

- For Trans Canada Highway No. 1 East:
  - Modified AASHTO MSS27 (HSS30) Design Truck Loading.
  - AASHTO MS27 (HS30) Lane Loading.
  - AASHTO LRFD HL “93” Loadings.
- For all other bridges””
  - Modified AASHTO MSS22.5 (HSS25) Design Truck Loading.
  - AASHTO MS22.5 (HS25) Lane Loading.
  - AASHTO LRFD HL “93” Loadings.
- Service Life:
  - Bridges are based upon a life span (including maintenance) of 75 years.
  - Traffic projections are based on a 20 year horizon.

Floodway Channel Geometry

- Based on the latest information provided by KGS (April 22, 2004).

Hydraulic Data

- The High Water Level (HWL) represents the 1 in 700 year design flood event.
- Channel design flow is 3960 m3/s (140,000 cfs).
- Design HWL for Highway Bridges is:
  - St. Mary’s Road = 237.130.
  - PTH 59 South = 236.870.
  - TCH No. 1 East = 236.250.
  - PTH 15 = 235.460.
  - PTH 59 North = 233.990.
  - PTH 44 = 231.570.
- Channel velocity for highway bridges:
  - Velocity = 1.52 m/s at St. Mary’s Road, PTH 59 South , TCH No. 1 E, and PTH 15.
  - Velocity = 2.13 m/s at PTH 59 North and PTH 44.
- Freeboard Clearance for Highway Bridges:
  - All bridges use 300 mm freeboard to HWL to underside of girders at abutments, then increased by a minimum 0.35% longitudinal grade to centerline of bridge.

- Ice impact and ice jams:
  - St. Mary's Road Bridge is designed for ice pressures due to an ice jam for a maximum water level of El. 237.744m (780 ft).
  - All other bridge piers are designed to accommodate ice loads due to impact of a solitary, freely moving ice floe. Maximum size of floe assumed to be 15 metres diameter by 0.6 metres thickness. The maximum stage for which this is considered possible is 230.4 metres (756 ft) at Floodway Inlet.

Discussion of each bridge crossing site, the site-specific design decisions and the resulting preliminary design is provided for each highway bridge crossing as follows:

4.6.1.3 St. Mary's Road PR 200

Figure 4.6-1 presents an aerial view of PR 200 Highway Bridge in June of 2004.

Figure 4.6-1
St. Mary's Road PR 200 Highway Bridge Crossing

During the iteration process, it became apparent that replacement of this structure was the best solution on a life cycle cost analysis basis. The issue arose as to whether or not this structure should be replaced at its existing elevation or at an elevation above the 1:700 year HWL. The logic to replace the structure above the 1:700 year water level was that the additional strengthening of the superstructure to resist the potential of lateral forces due to water, ice, and debris forces would be largely offset by the cost of raising the structure. The second argument was being able to maintain traffic on St. Mary's Road during construction. The third most compelling argument was the significant hydraulic advantage of raising the bridge, that would result for flood events below the 1:700 year design water level.

A total of four alternative replacement bridge alignments and roadway approaches were considered. In consultation with Manitoba Transportation and Government Services (MTGS), an alignment locating a new bridge and roadways east of the existing highway alignment beginning at Fraser Road, north of the Floodway, and extending to Greenview Drive south of the expanded Floodway was chosen as the best overall alternative in terms of geometrics and property disruption. The width of the structure was also reviewed at this time and although the MTGS standard width for a Provincial Road is 9.600 m, it was deemed justified to use a width of 10.900 metres because of the high volume of bicycle traffic across the bridge. The decision of whether to use a standard bridge rail or a combination pedestrian/traffic rail was deferred to final design.

Roadway Classification

The following are the findings for the roadway and bridge evaluation at the PR 200 Bridge crossing of the Floodway along St. Mary's Road:

- St. Mary's Road (PR 200) is classified a Collector A roadway, and is currently a two lane undivided roadway.
- St. Mary's Road is currently and will continue to operate at a LOS B throughout the 20-year design horizon.
- No widening of the roadways or structure is needed at this location.
- No roadway improvements on either side of the existing structure are expected, as the structure crossing will not be relocated.

Alignment/Geometrics

The alignment of a new crossing of the Expanded Floodway is shown on Figure 4.6-2, which was chosen in consultation with MTGS and offers the best solution in terms of overall highway alignment, access control, and minimal effect on existing residences. The new alignment will allow the existing crossing to remain open to traffic during construction of the new structure.

The proposed alignment will locate the new bridge and roadways east of the existing highway alignment beginning at Fraser Road, north of the Floodway, and extending to Greenview Drive south of the Expanded Floodway. Immediately south of Fraser Road, the proposed alignment departs from the existing alignment, with a 470 metres curve. The alignment becomes tangent at Chrypko Drive and passes between two residential properties before crossing the Floodway. A 370 metres tangent portion of roadway is provided on the north side of the new bridge structure. The new roadway and bridge structure cross the Floodway within the curved portion of the channel at approximately a 6^o skew. The proposed roadway alignment continues south of the new structure with a 110 metres tangent followed by a 500 metres curve to the west. The proposed alignment crosses to the west side of the existing highway approximately 350 metres south of the existing structure and reconnects to the existing St. Mary's Road alignment at Greenview Drive. This proposed roadway alignment would require the intersections at Courchane Road and Chrypko Drive to be relocated, connecting with St. Mary's Road at the end of curve north of the Floodway. The longitudinal grade over the bridge is 2% with the high point at midspan and a vertical curve length of 420 m. Lateral crossfall of the decks will be 2% from centreline to each shoulder. The bridge deck will be 12.100 metres wide consisting of two lanes at 3.700 m (12 ft), two shoulders at 1.750 metres each, and two curbs at 600 mm (2 ft) each. This cross section allows for additional shoulder width to accommodate cyclists. The PR 200 and Courchane Road cross section will consist of two lanes at 3.700 metres with 2.000 metres shoulders.

Minimum private property required for the construction of the new approaches has been estimated at 3.75 ha. Actual property to be acquired may be more depending on expropriation. Existing Floodway property required for the new approaches has been estimated at 6.00 ha, but no cost has been assigned to the project for this property.

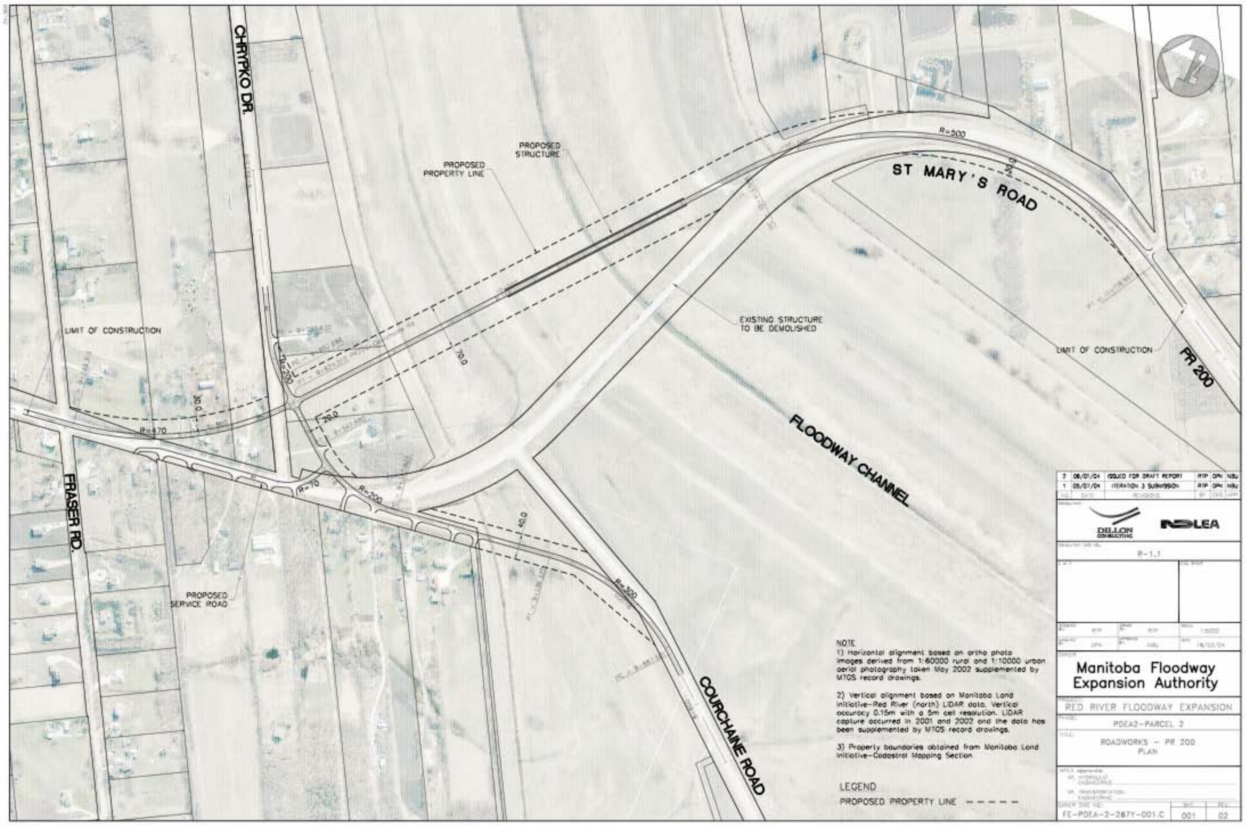


Figure 4.6-2
Alignment of new PR 200 St. Mary's Road Bridge Crossing

Superstructure

Seven equal spans of 43.5 metres are contemplated for a total length of 304.5 m. The superstructure will consist of five lines of 2000 mm precast prestressed concrete NU girders spaced at 2400 mm on centres. The reinforced concrete deck could be 225 mm thick continuous over the piers for live load. The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at the time of tender.

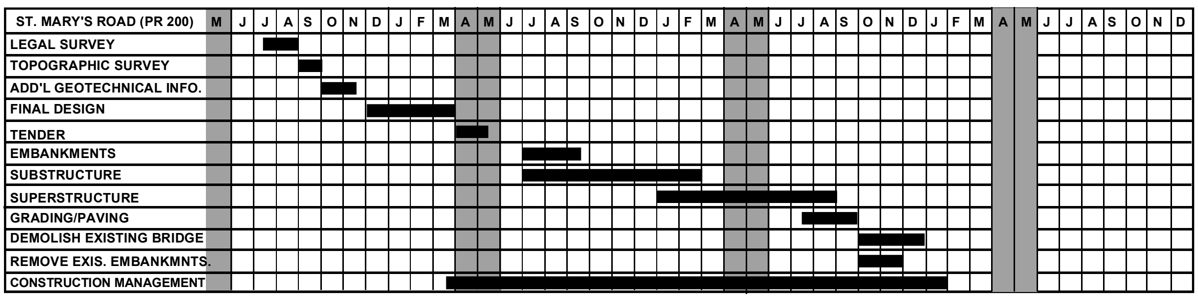
The existing structure has conduits suspended below the deck, some or all of which are occupied by MTS. The new structure should have six - 100 mm diameter conduits suspended below the deck to accommodate MTS. One - 38 mm diameter conduit will be considered in one curb for potential future lighting.

Substructure

The abutments would be reinforced concrete gravity-type founded on 405 mm hexagonal precast prestressed concrete piles driven to refusal. The piers would also be reinforced concrete supported on 405 mm hexagonal precast prestressed concrete piles driven to refusal. Deep clays at this site warrant the use of the concrete piles which are less expensive than steel piles and excessive ground water release is not a concern.

Construction and Workforce Scheduling

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-3 provides a conceptual timeline for major tasks associated with replacement of the existing St. Mary's Road Bridge and related site preparation tasks.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS			3	3	3	3	3	5	5	5	5	2	1	5	27	27	23	20	20	20	40	40	25	25	25	25	23	30	15	20	20	15	2	2											487
PERSON DAYS (21 DAYS/MTH)																																													10,227

Source: Dillon/NDLea 2004

Figure 4.6-3

Conceptual Timeline: St. Mary's Road Bridge Replacement

4.6.1.4 PTH 59 South

Figure 4.6-4 presents an aerial view of the existing PTH 59 South crossing.

Figure 4.6-4
Aerial View of the Existing PTH 59 South Crossing

Based on the life cycle cost analysis, the existing southbound bridge is recommended to be replaced and the existing northbound bridge is recommended to be raised and lengthened. To accommodate the new bridge elevations, the existing PTH 59 roadway profiles will be raised and Prairie Grove Road beneath the north end of the structure will be realigned. Manitoba Transportation and Government Services agreed with this functional design concept and it was therefore used for the preliminary design.

<u>Roadway Classification</u>

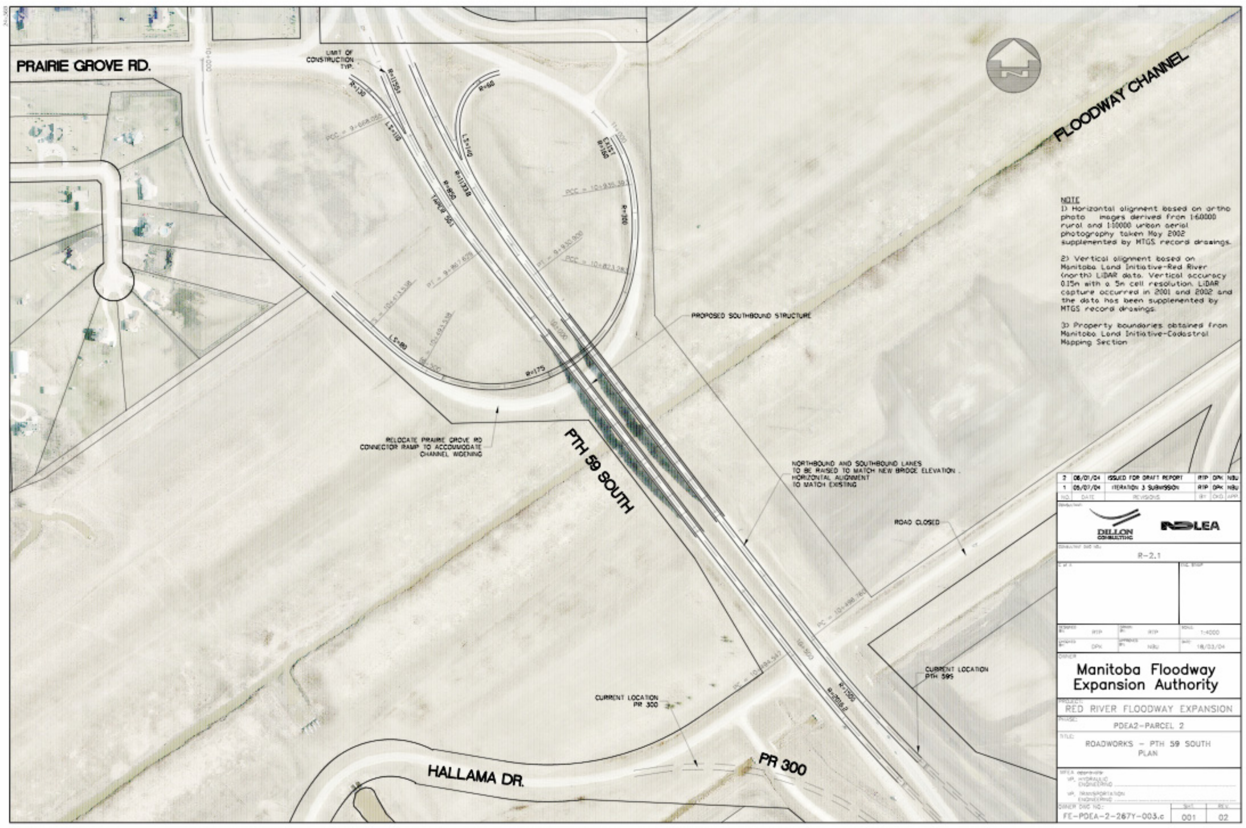
The following are the findings for the roadway and bridge evaluation at the PTH 59 South crossing of the Floodway:

- PTH 59 South is classified as an Expressway and is currently a four-lane divided roadway.
- PTH 59 South is currently operating at a LOS A and will operate at a LOS “B” at the end of the 20 year design horizon.
- This is an acceptable LOS and no widening of the roadways or structures are recommended at this location.

- Roadway and ramp improvements are anticipated at this location as the structures are being raised in elevation. Other than these tie-in works and improvements, no re-alignment of the roadway on either side of the structures is anticipated.

Alignment/Geometrics

The horizontal alignment of both the northbound and southbound structures will remain as per existing conditions, as shown in Figure 4.6-5. The longitudinal approach grades to the bridges will be 2% at either end, with the exception of the south approach southbound being 1.6%. Lateral crossfall of the decks will be 2.0% for the southbound structure and 1.5% for the northbound structure taken from centreline to both shoulders. In order to meet minimum stopping site distances, the roadways will have to be reconstructed a minimum of 380 metres on either side of the structures.



Source: Dillon/NDLea 2004

Figure 4.6-5

Horizontal Alignment of northbound and southbound PTH 59 S Structures.

The existing bridge deck of the northbound structure is 11.900 wide consisting of two lanes at 3.700 metres each, two shoulders at 1.700 metres each, and two curbs at 550 mm each. The cross section of the proposed additional end spans will match the existing structure. The bridge deck of the proposed southbound structure will be 12.100 metres wide consisting of two lanes at 3.700 metres each, a median shoulder of 1.500 m, and a gutter shoulder of 2.000 metres plus two curbs at 600 mm each.

Due to the channel widening, the Prairie Grove ramp will need to be realigned and will require a radius modification to the section immediately below the structures. The curve radius will change from 160 metres to 175 m.

To accommodate traffic during construction, the bridge improvements will be carried out one bridge at a time so the adjacent structure can be used to convey traffic. Crossover detours will be constructed during this time.

Superstructure

The proposed southbound bridge is nine spans of 27.870 metres and two spans of 27.795 metres for a total length of 306.420 m. The superstructure would consist of five lines of 1600 mm deep precast prestressed concrete NU girders spaced at 2400 mm on centres. The reinforced concrete deck would be 225 mm thick continuous over the piers for live load. The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at time of tender.

The proposed northbound bridge will consist of the existing nine spans, seven spans at 27.870 metres and two spans at 27.795 m, plus new additional end spans of 27.870 metres each for a total length of 306.420 m. The new additional simple end spans superstructure will consist of five lines of 1600 mm deep NU precast prestressed concrete girders spaced at 2400 mm on centre. The SU-2 and SU-11 pier caps will be stepped to accommodate the difference in girder depths. The reinforced concrete deck slab of the two new end spans would be 200 mm thick, complete with a 75 mm asphalt overlay (to match the existing deck slab). The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at time of tender.

The existing southbound structure has six conduits suspended below the deck, some or all of which are occupied by MTS. The MTS plant will have to be temporarily relocated during replacement of the southbound structure. The new southbound structure should have six – 100 mm diameter conduits suspended below the deck to accommodate MTS. One - 38 mm diameter conduit should be considered in one curb of the southbound structure for potential future lighting.

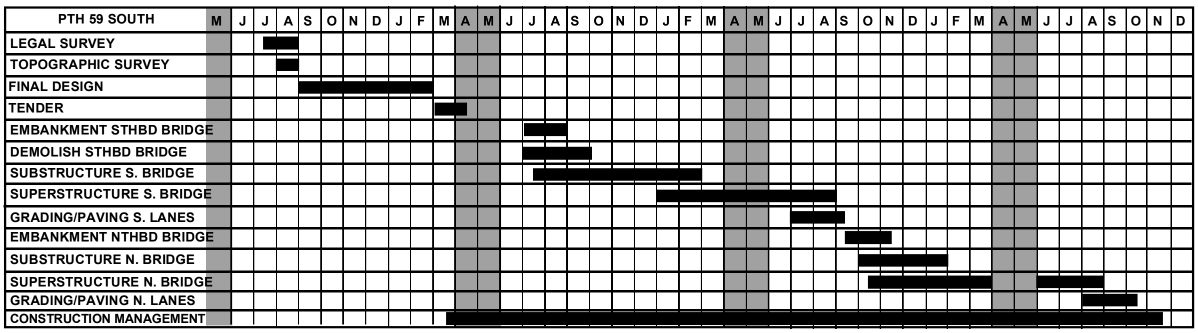
Substructure

The abutments for both the southbound and northbound structures would be reinforced concrete gravity-type founded on 405 mm hexagonal precast prestressed concrete piles driven to refusal. The three new piers for the northbound structure and all piers for the southbound structure would be reinforced concrete supported on 405 mm hexagonal precast prestressed concrete piles driven to refusal. Deep clays at this site warrant the use of the concrete piles which are less expensive than steel piles and excessive ground water release is not a concern. The existing piers for the northbound structure would be extended with reinforced concrete.

Typical abutment, pier, and pier extension details are shown on Drawing FE-PDEA-2-255C002.c. The existing pier of the northbound structure would be reinforced with concrete skirt encasements to accommodate the expanded channel profile.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F
PERSON MONTHS			3	6	5	5	5	5	5	5	2	2		5	30	37	30	20	20	20	40	40
PERSON DAYS (21 DAYS/MTH)																						

ESTIMATED WORKFORCE	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS	25	25	25	20	30	30	15	37	42	40	40	25	25			20	20	30	15	15	2	2	770
PERSON DAYS (21 DAYS/MTH)																							16,200

Source: Dillon/NDLea 2004

Figure 4.6-6

Conceptual Timeline: PTH 59 South Bridge Replacement

4.6.1.5 Trans-Canada Highway No. 1 East

Figure 4.6-7 presents an aerial view of the existing TCH No. 1 East crossing.

Figure 4.6-7
Aerial View of the Existing TCH No. 1 East Crossing

From the outset of the design iteration process, this bridge was considered as a candidate for replacement above the 1:700 year HWL (recommended in the SAFE Report). The results of the Condition Assessment Findings/Load Rating from PDEA1 and a life cycle cost analysis confirmed that the structure should be replaced. During the Iteration 1 process, an evaluation of traffic level of service at this bridge site concluded that four traffic lanes would be required up to the 20-year design horizon and thereafter widening to six lanes would be necessary. Two new bridge structures were recommended, one of which would facilitate a detour crossing during construction. Manitoba Transportation and Government Services concurred with this evaluation and recommended functional design.

In the Iteration 2 Draft Report, further evaluation was carried out for the bridge crossing as well as its relationship to the close proximity of the TCH No. 1E/PTH 100 interchange and a future interchange of TCH No. 1E with PR 207. The results of the evaluation determined that the length of the acceleration lane from PTH 100 northbound to TCH No. 1E eastbound across the Floodway bridge is substandard. Also, the length of the existing deceleration lane from TCH No. 1E westbound to PTH 101 northbound west of the Floodway bridge is substandard. To address the substandard acceleration and deceleration lanes, it was recommended that the eastbound bridge be constructed as a three-lane structure and that a flare be provided on the westbound structure to accommodate the transition of a deceleration lane.

Manitoba Transportation and Government Services reviewed the proposed structure layout and asked that consideration be given to providing a full three lanes on the westbound structure rather than retrofitting the structure in the future. This revision was carried forward to the preliminary design.

Roadway Classification

The following are the findings of the roadway and bridge evaluation at the TCH No. 1E bridge crossing of the Floodway:

- TCH No. 1E is classified as an Expressway and is currently a four-lane divided roadway.
- TCH No. 1E is currently operating at a LOS "B," a LOS "C" at the end of the 20-year design horizon, and a LOS "D" within the life span of the bridge structure.
- The LOS "C" rating is acceptable for traffic volumes but as the LOS will drop to "D" within the life span of the bridge structures, it is recommended that provisions be made to accommodate widening the Floodway crossing to a future six-lane facility with two separate three-lane structures.
- Significant roadway and ramp improvements will be required to accommodate any roadway or structural widening at this location.
- The construction of a future six-lane bridge across the Floodway at TCH No. 1E will not negatively impact future roadway improvements at this location. Any future roadway works can be accommodated with the bridge structure proposed at the Floodway and a future interchange of TCH No. 1E and PR 207 to the east of the Floodway.

Alignment/Geometrics

The proposed alignments for the bridges and associated roadways at this site are shown on Figures 4.6-8 and 4.6-9.

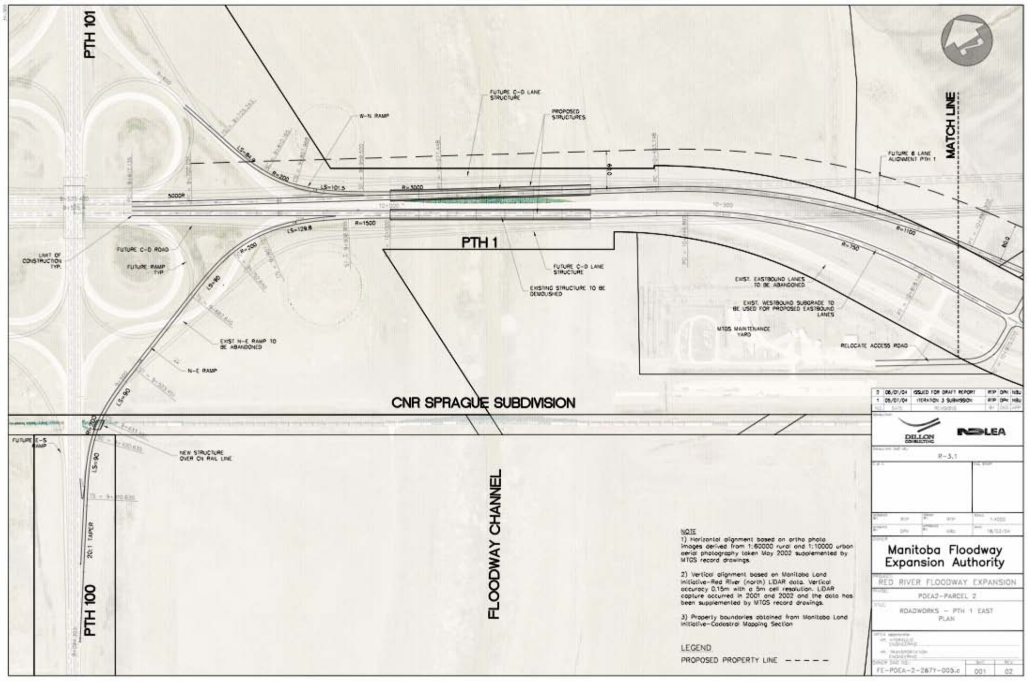


Figure 4.6-8
Bridges and Roadworks Alignment – TCH No. 1 East

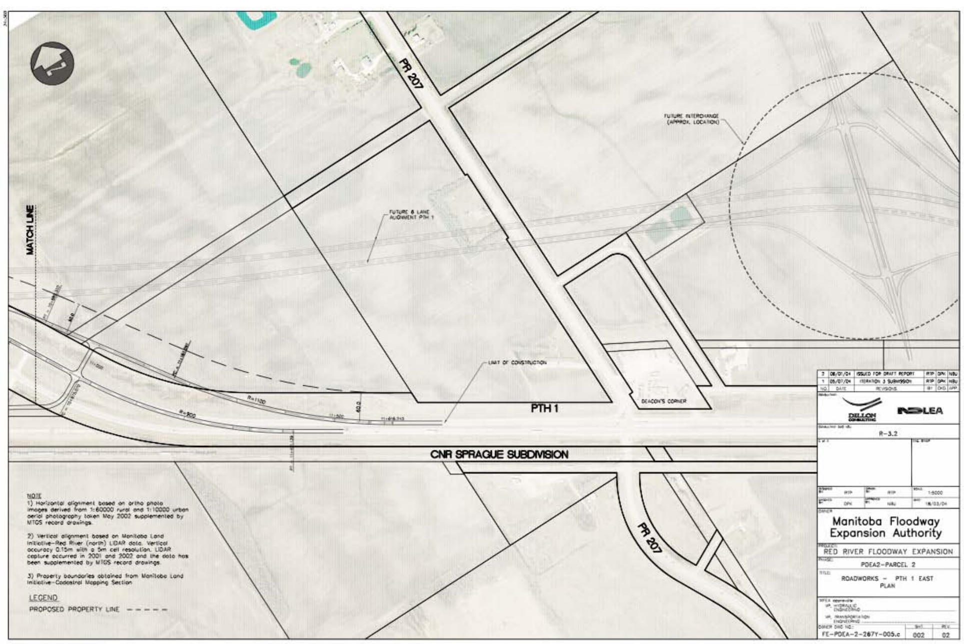


Figure 4.6-9
Bridges and Roadworks Alignment – TCH No. 1 East.

Eastbound TCH No. 1E will generally stay on the existing alignment between the PTH 100/101 overpass and the Expanded Floodway. East of the Floodway structure, the roadway alignment will cross onto the existing westbound lanes with a 750 metres radius curve and utilize the existing westbound lanes before crossing back over to the existing eastbound lanes with a 900 metres radius. The existing eastbound lanes will be abandoned from the east tie-in to the Floodway bridge. Realignment of TCH No. 1E westbound will begin at approximately 70 metres east of the PTH 100 overpass and move north with a 5000 metres radius curve to match the alignment of the new westbound Floodway structure. The alignment will cross over the Floodway on tangent with a skew angle to the Floodway of approximately 1o - 43'. East of the Floodway bridge the roadway alignment will curve south with a 1100 metres radius curve, continue with a tangent and tie into the existing westbound lanes with a 1100 metres radius curve. The existing ramp from PTH 100 northbound to TCH No. 1E eastbound will have to be realigned in order to meet current geometric design standards. Improvements of this ramp will start on PTH 100 approximately 320 metres south of the rail overpass and terminate on TCH No. 1E eastbound approximately 100 east of the new bridge. The new ramp will include a 20:1 exit taper off PTH 100, a new rail overpass and a 500 metres long parallel lane entrance terminal that will extend across the eastbound bridge. This lane will serve as the third lane when TCH No. 1E is upgraded to a future six-lane facility. The proposed exit terminal from PTH 1 westbound to PTH 101 northbound will be a parallel type starting at the east end of the new westbound structure and terminate on the existing ramp approximately 350 metres beyond the west end of the proposed bridge structure. This lane will serve as the third lane when TCH No. 1E is upgraded to a future six-lane facility.

Each of the bridge decks will be 15.800 metres wide consisting of three traffic lanes at 3.700 metres each, a median shoulder of 1.500 m, and a gutter shoulder of 2.000 metres plus two curbs at 600 mm each. The westbound bridge will require a 1.000 metre flare of the deck at the northwest corner to accommodate the geometrics of the deceleration lane. Lateral crossfall of the decks will be 2% from median to shoulder. The approach roadways will be 11.900 metres wide consisting of two 3.700 metres lanes, a median shoulder of 1.500 m, and a gutter shoulder of 3.000 m. The median will be a raised type west of the structure and depressed east of the structure. Ramp widths will be 4.800 metres with a 1.500 metres shoulder on the left and a 3.000 metres shoulder on the right.

Private property required for the construction of the new approaches has been estimated at 4.35 ha. Existing Floodway property required for the new approaches has been estimated at 2.00 ha, but no cost has been assigned to the project for this property.

Superstructure

The preliminary bridge designs consist of twin three-lane bridges, each with seven equal spans of 43.5 metres centre to centre of bearings, for a total length of 304.5 m. The superstructures of each bridge would consist of seven lines of 2000 mm deep precast prestressed concrete NU girders spaced at 2250 mm on centres. The reinforced concrete deck would be 225 mm thick continuous over the piers for live load. The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at time of tender.

There are no conduits in the existing structure. One of the new structures should have six – 100 mm diameter conduits suspended below the deck for future use. Each of the new structures should be provided with one - 38 mm diameter conduit in one curb for lighting.

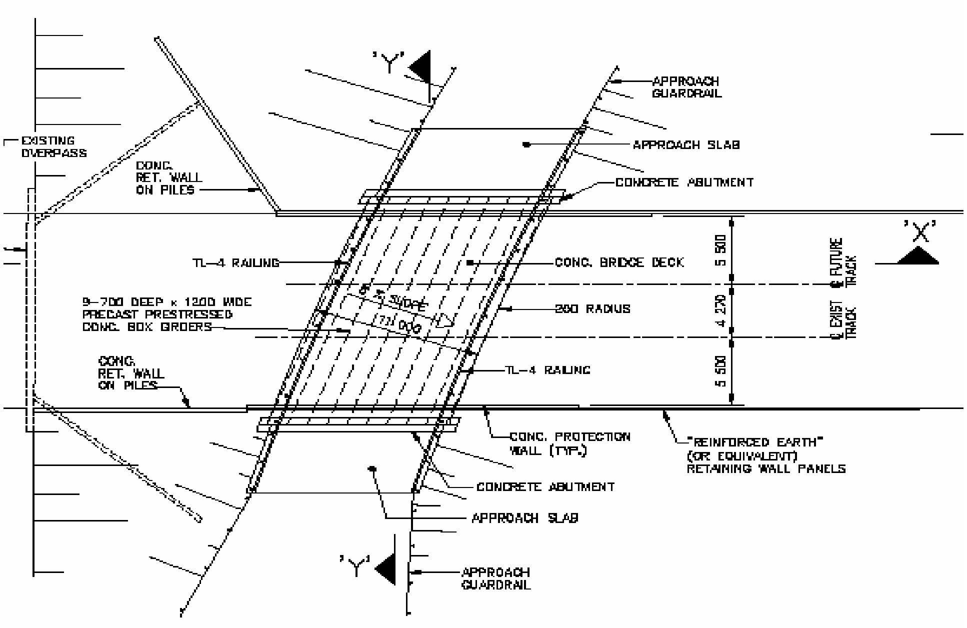
A gas main crosses north/south approximately 30 m± west of the proposed new west abutments and therefore should not require relocation.

Substructure

The abutments would be reinforced concrete gravity-type founded on 405 mm hexagonal precast prestressed concrete piles driven to refusal. The piers would also be reinforced concrete supported on 405 mm hexagonal precast prestressed concrete piles driven to refusal. Deep clays at this site warrant the use of concrete piles which are less expensive than steel piles and excessive ground water release is not a concern.

Rail Overpass Ramp Structure

A new rail overpass ramp structure to accommodate the northbound PTH 100 to eastbound TCH No. 1E movement is shown in concept design only in Figure 4.6-10. The concept design was developed to provide overall costing of the geometrics necessary at this interchange to meet current design standards. The width of the structure would be 10.890 mm and the single span length would be 18.000 m. The deck width will be made up of one 4.800 metres lane, a 1.500 metres shoulder on the left, a 3.000 metres shoulder on the right, and two curbs at 795 mm each.

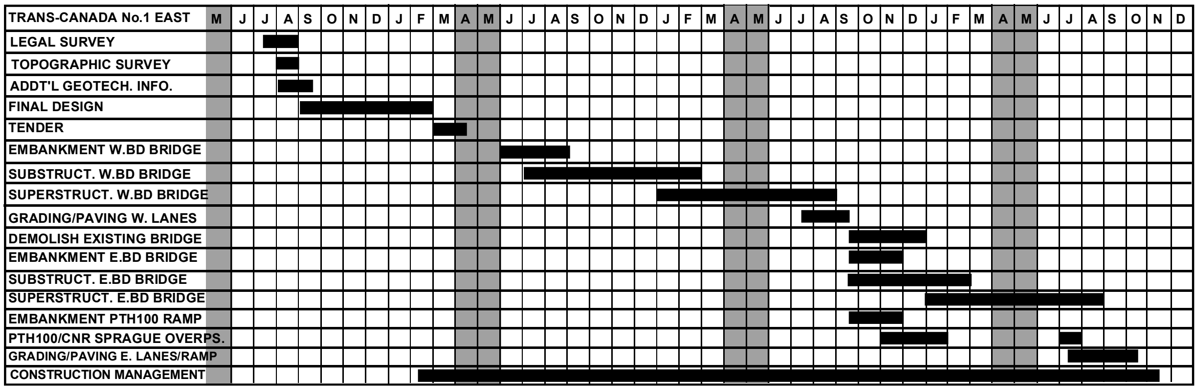


Source: Dillon/NDLea 2004

Figure 4.6-10
Rail Overpass Ramp Structure.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation location. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-11 presents the conceptual construction timeline for the Trans Canada Highway No. 1 East crossing.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS			3	9	9	5	5	5	5	5	2	1		9	27	27	20	20	20	20	40	40	25	25	25	20	30	30	30	45	65	50	60	40	25	30	20	20	32	30	15	15	2	2	910
PERSON DAYS (21 DAYS/MTH)																																													19,100

Source: Dillon/NDLea 2004

Figure 4.6-11

Conceptual Timeline – TCH No. 1 East Crossing

4.6.1.6 PTH 15

Figure 4-6.12 shows an aerial view of the existing PTH 15 Highway Bridge structure.

Figure 4-6.12
Aerial View of the Existing PTH 15 Highway Bridge Structure

In the Iteration 1 Draft Report, the length and height of the existing bridge was determined to be adequate to accommodate the 1:700 year HWL. However, the results of the Condition Assessment Findings/Load Rating from PDEA1 and a life cycle cost analysis concluded that the structure should be replaced. An evaluation of the traffic level of service at this site concluded that four traffic lanes were required across the Floodway.

In the Iteration 2 Draft Report, cost estimates and preliminary design were developed based on providing twin two-lane bridges across the Floodway. Manitoba Transportation and Government Services reviewed and concurred with the proposed recommendation.

Toward the end of the Iteration 3 process, the Province of Manitoba announced their intention for the twinning and grade separation of PTH 101 immediately to the west of the Floodway crossing. To accommodate the grade separation of PTH 101 will require the flaring of the westbound PTH 15 structure for a westbound to northbound ramp at PTH 101 and may also require a flare of the west end of the eastbound structure. Since this development occurred late in the iteration process, an attempt has been made in this Draft Report to show a possible solution to the structure flare of the westbound bridge and quantify the associated costs.

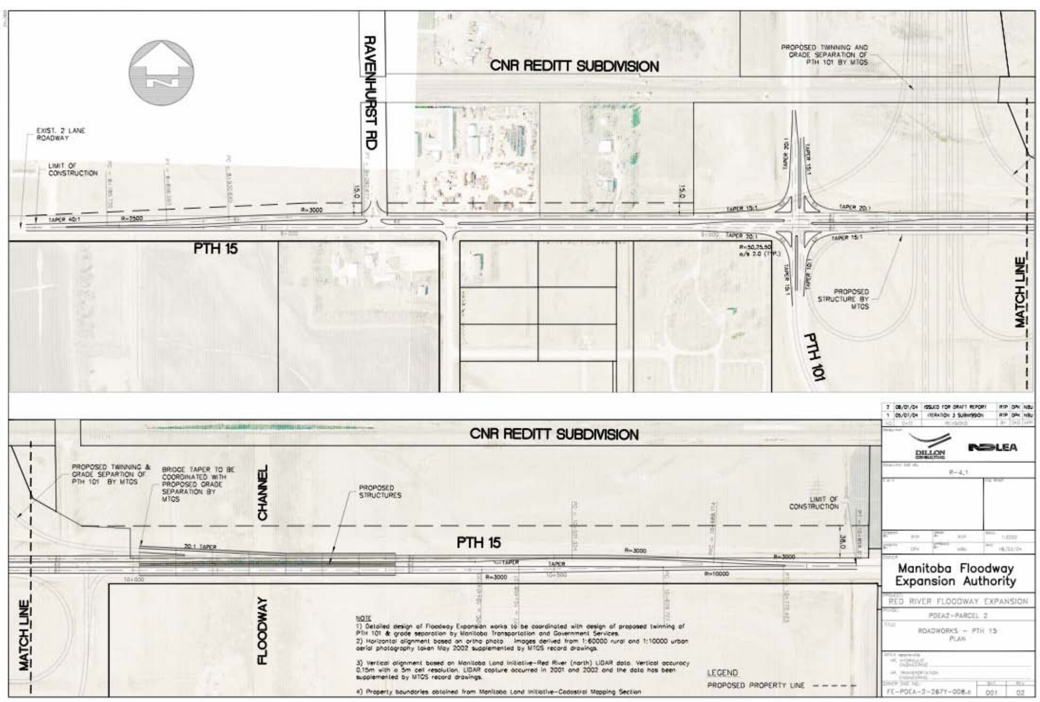
Roadway Classification

The following are the findings for the roadway and bridge evaluation at the PTH 15 bridge crossing of the Floodway:

- PTH 15 is classified as a Primary Arterial and is currently a two-lane undivided roadway.
- PTH 15 has a high collision rate and provisions should be made for a four-lane facility at this location.
- PTH 15 is currently operating at a LOS "D" which is not acceptable and provisions should be made for a four-lane facility at this location.
- On the assumption that a four-lane facility will be constructed, PTH 15 will operate at a LOS "B" for most of the 20-year design horizon, but will drop to a LOS "C" at the end of the design period.

Alignment/Geometrics

The alignment of a new crossing of the Expanded Floodway is shown in Figure 4.6-13.



Source: Dillon/NDLea 2004

Figure 4.6-13
Horizontal alignment of new PTH 15 Crossing.

The eastbound lanes will remain as per the existing alignment. The new westbound lanes will parallel the eastbound lanes at an offset of 13.400 metres centre to centre of roadways. East of the structures, the four-lane roadway would transition back to a two-lane facility using the existing eastbound lanes. Roadway geometry west of the structure is shown on the drawings as a potential short-term solution only. The twinning and grade separation of PTH 101 will likely occur concurrently with the construction of the new bridges across the Expanded Floodway, and therefore, temporary and final alignments will be coordinated during detailed design.

The bridge deck of the proposed structures will be 12.100 metres wide, each consisting of two lanes at 3.700 metres each, a median shoulder of 1.500 m, and a gutter shoulder of 2.000 m, plus two curbs at 600 mm each. The approach roadways will be 11.900 metres wide consisting of two 3.700 metres lanes, a raised median and a gutter shoulder of 3.000 m. Lateral crossfall of the decks will be 2% from median to shoulder.

To provide a westbound to northbound ramp onto PTH 101, the westbound structure will be flared 8.700 metres at the west abutment.

Private property required for the construction of the new approaches has been estimated at 1.00 ha. Existing Floodway property required for the new approaches has been estimated at 3.35 ha, but no cost has been assigned to the project for this property.

Superstructure

These are twin two-lane bridges, each with seven equal spans of 43.5 metres centre to centre of bearings, for a total length of 304.5 m. The superstructures of each bridge would consist of five lines of 2000 mm deep precast prestressed concrete NU girders spaced at 2.400 metres on centre. The reinforced concrete deck would be 225 mm thick continuous over the piers for live load. The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at the time of tender.

The existing structure has conduits suspended below the deck, some or all of which are occupied by MTS. The new westbound structure should have six – 100 mm diameter conduits suspended below the deck to accommodate MTS. The MTS plant would be relocated from the existing eastbound structure. One – 38 mm diameter conduit should be considered in one curb of each new structure for potential future lighting.

A gas main crosses north/south approximately 20 m± west of the proposed new west abutments and therefore should not require relocation.

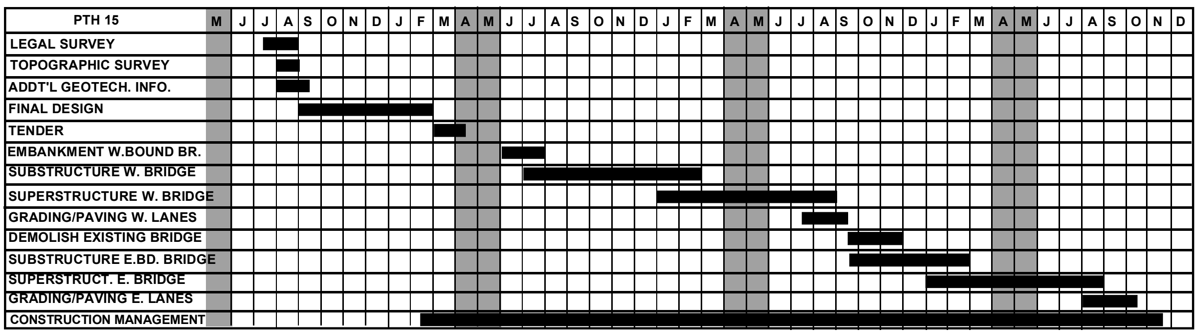
Substructure

The abutments would be reinforced concrete gravity-type founded on HP 310 x 110 steel piles driven to refusal. The piers would also be reinforced concrete supported on HP 310 x 110 steel piles driven to refusal. Ground water is anticipated to be a potential problem at this site and therefore the steel piles would cause less disturbance to the clay **overburden** than concrete piles.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress.

Figure 4.6-14 provides conceptual scheduling associated with construction of the new PTH 15 crossing and associated project elements.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M
PERSON MONTHS			3	9	5	5	5	5	5	5	2	2	
PERSON DAYS (21 DAYS/MTH)													

ESTIMATED WORKFORCE	J	J	A	S	O	N	D	J	F	M	A	M
PERSON MONTHS	12	22	20	20	20	20	20	40	40	25	25	25
PERSON DAYS (21 DAYS/MTH)												

ESTIMATED WORKFORCE	J	J	A	S	O	N	D	J	F	M	A	M
PERSON MONTHS	20	30	30	22	30	30	20	40	40	25	25	25
PERSON DAYS (21 DAYS/MTH)												

ESTIMATED WORKFORCE	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS	20	20	30	15	15	2	2	780
PERSON DAYS (21 DAYS/MTH)								16,300

Source: Dillon/NDLea 2004

Figure 4.6-14
Conceptual Scheduling for PTH 15 Crossing

4.6.1.7 PTH 59 North

Figure 4.5-15 presents an aerial view of the existing PTH 59 Bridge Crossing configuration.

Figure 4.6-15
Aerial View of the Existing PTH 59 Bridge Crossing Configuration

In the Iteration 1 Draft Report, this bridge was recommended to be replaced based on the results of the Condition Assessment Findings/Load Rating from PDEA1 and a life cycle cost analysis. A level of service evaluation was carried out and the results of a previous functional design study undertaken by Manitoba Transportation and Government Services were reviewed. These studies indicated that in the future, a six-lane facility, including a second bridge is required at this location. The new bridge structure should be built south of the existing bridge to carry northbound traffic.

In the Iteration 2 Draft Report, the Channel Consultant confirmed that the elevation of the existing bridge was sufficiently high enough to be above the 1:700 year HWL, but that widening of the channel would require the bridge to be approximately 50.0 metres longer. The proposed channel widening would also require a realignment of the Oasis Road connector at the east end of the bridge. This realignment necessitated the addition of entrance and exit tapers at the east end of the proposed southbound and northbound bridges. Future widening of the proposed structures was recommended to the inside (median) of each structure. Manitoba Transportation and Government Services reviewed the concepts and asked that the option of constructing parallel decelerating and acceleration lanes over the entire structure be considered.

The acceleration and deceleration lanes were reviewed. The conclusion for the acceleration lane on the southbound structure was that the costs of providing this lane as compared to the taper were similar and since it provided a greater degree of safety, the parallel acceleration lane was carried forward to the preliminary design. The same result for the deceleration lane on the northbound structure could not be proven and therefore the preliminary design carried forward in the Iteration 3, Draft Report included a taper as previously recommended.

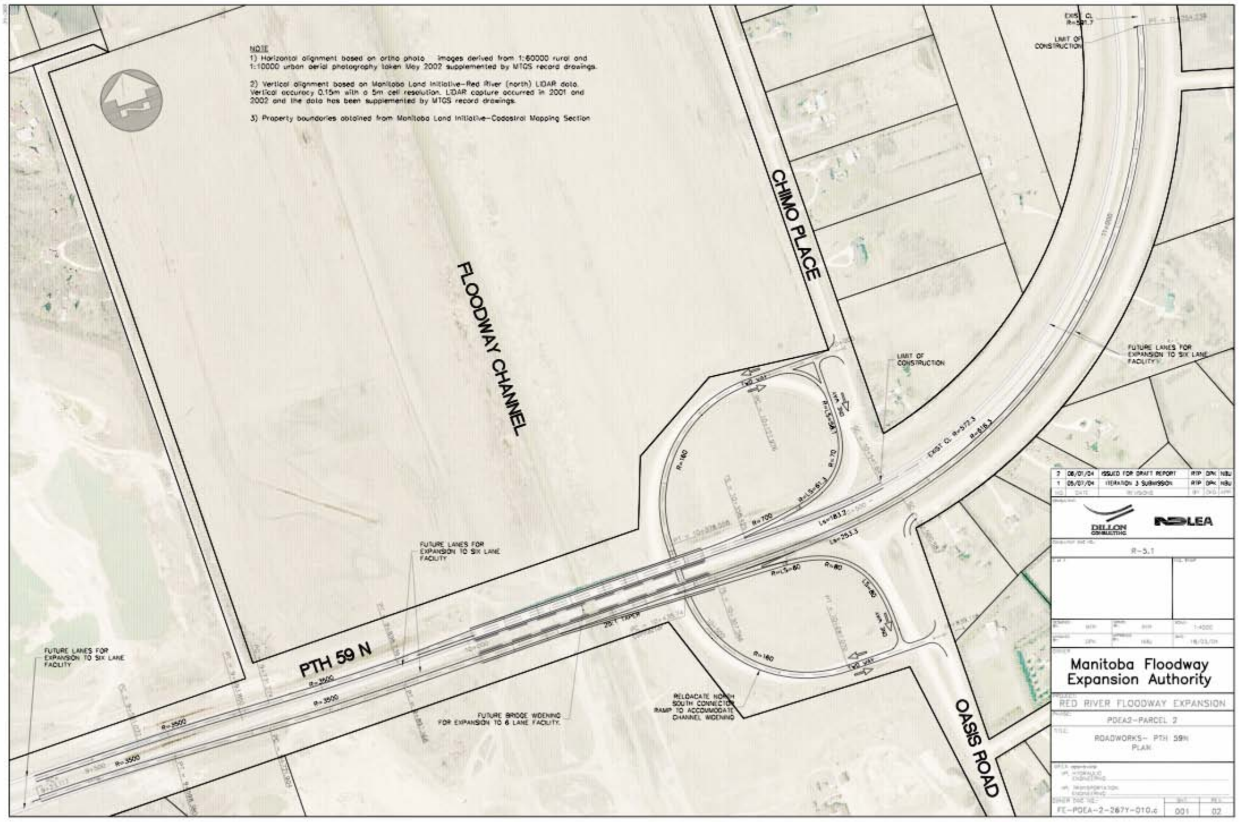
Roadway Classification

The following are the findings for the roadway and bridge evaluation at PTH 59 North, bridge crossing of the Floodway:

- PTH 59 North is classified as an Expressway and is currently a four-lane divided roadway.
- PTH 59 North, is currently operating at a LOS “B” for most of the 20-year design horizon, but will drop to a LOS “C” at the end of the design period.
- The LOS “C” rating is acceptable for traffic in an urban area but as the LOS will drop to “D” within the life span of the bridge structures. It is suggested that consideration be given to accommodate a future six-lane bridge facility at this location.
- Studies for the upgrading of PTH 59 North have been undertaken and concur with this recommendation.
- Roadway improvements will need to be carried out at this location.

Alignment/Geometrics

The alignment of a new crossing of the Expanded Floodway is shown Figure 4.6-16. The proposed alignment closely follows the design and recommendations from MTGS functional design dated February 2000 at this location with the exception of the southbound entrance terminal at the north east corner of the bridge. East of the bridge, proposed southbound lanes will shift south with reverse 3500 metres radii. The Floodway crossing would be on tangent to just east of the structure. The new northbound lanes will parallel the southbound lanes at an offset of 29.6 metres centre to centre of roadways. A long spiral and 620 metres radius will slowly tie into the existing curve. Southbound traffic would generally remain in the same location, however, would be shifting north to accommodate future expansion to six lanes.



Source: Dillon/NDLea 2004

Figure 4.6-16
PTH 59 N Horizontal Alignment

The exit terminal from PTH 59 northbound to Oasis Road will be a direct taper utilizing a 25:1 taper to exit the roadway. This will require the east end of the northbound bridge to be widened. The entrance terminal on the southbound structure will be a parallel lane, continuing across the bridge. It will terminate approximately 100 metres from the west end of the bridge. Due to the channel widening, the Oasis Road connector will be realigned and will require a radius modification. The Oasis Road connector will operate as a two-way roadway as recommended in the functional design report.

The bridge deck of the proposed southbound structure will be 15.800 metres wide consisting of three traffic lanes at 3.700 each, a median shoulder of 1.500 m, and a gutter shoulder of 2.000 m, plus two curbs at 600 mm each. The bridge deck of the proposed northbound structure will be 12.100 metres wide for most of the length consisting of two traffic lanes at 3.700 metres each, a median shoulder of 1.500 m, and a gutter shoulder of 2.000 m, plus two curbs at 600 mm each. The east end of this structure will be flared by 7.950 metres to accommodate the exit ramp. The flare will start at 102.350 metres from the west end of the bridge. The approach roadways will be 11.900 metres wide consisting of two 3.700 metres lanes, a median shoulder of 1.500 m, and a gutter shoulder of 3.000 m. and a depressed median. Ramp widths will be 4.800 metres with a 1.500 metres shoulder on the left and 3.000 metres shoulder on the right. The north-south connector road will consist of two traffic lanes of 3.700 metres each with 2.000 metres shoulder on each side.

Superstructure

The southbound structure is a three-lane bridge and the northbound structure is a two-lane bridge, with a flare at the east end. Each bridge has seven equal spans of 43.5 metres centre to centre of bearings, for a total length of 304.5 m. The superstructure of the southbound bridge would consist of seven lines of 2000 mm deep precast prestressed concrete NU girders spaced at 2250 mm on centre. The superstructure of the northbound bridge would consist mainly of five lines of 2000 mm deep precast prestressed concrete NU girders spaced at 2400 mm on centres, with additional girders being added at the east end to accommodate the flare of the exit ramp. To accommodate the flare will require one additional girder line for Span SU-4 to SU-5; two additional girder lines for Span SU-5 to SU-7; and three additional girder lines for Span SU7 to SU-8.

The reinforced concrete decks would be 225 mm thick continuous over the piers for live load. The decks would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at time of tender.

Steel-reinforced elastomeric bearing pads would be utilized with fixed bearings at Pier SU-5 and expansion bearings at the other piers and abutments. Two multi-seal expansion joints would be provided, one at each abutment. The bridge guardrail will be MTGS Standard TL-4 railing.

The existing structure has conduits suspended below the deck, some or all of which are occupied by MTS. The new southbound structure should have four – 100 mm diameter conduits suspended below the deck for future use. The MTS plant would be relocated from the existing structure and buried across the Expanded Floodway at this location. One – 38 mm diameter conduit should be considered in one curb of each new structure for potential future lighting.

A gas main crosses north/south approximately 9 m± west of the proposed new west abutments and therefore would not require relocation, but may require protection during construction of the abutments.

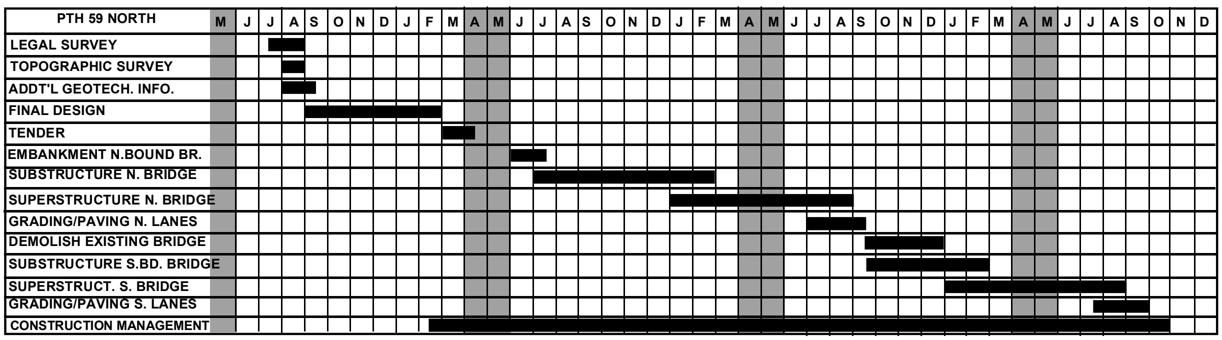
Substructure

The abutments would be reinforced concrete gravity-type founded on HP 310 x 110 steel piles driven to refusal. The piers would also be reinforced concrete. The foundations for Piers SU-2 and SU-7 would be founded on HP 310 x 110 steel piles. The foundations within the channel will likely encounter ground water problems. The underlying soil strata which overlies the limestone bedrock is insufficient in depth to resist hydraulic uplift forces from the ground water aquifer which lies within the limestone bedrock. For this reason, a potential piping problem exists at the proposed bridge foundations. This piping condition could wash material from the piles and remove lateral support. A ground water release of approximately 300 Imperial gallons per minute is currently occurring in the area of the existing bridge.

For the above reasons, it would appear reasonable to consider a foundation treatment of the bedrock in an attempt to limit the impact of bridge construction on the ground water regime and to ensure long-term performance of the bridge foundations. A **grouting** operation should be investigated for those piers within the Floodway channel. The recommended option would be the injection of a cement grout into the upper 3 metres of limestone bedrock and filling of the joints of the bedrock through an area approximately 3 metres wide by 12 metres long for each pier. This is termed defragmentation or closing of joints from inside the pier base footprint outward. This would be accomplished with two rows of grout holes spaced at 1.5 metres both longitudinally and transversely, with the installation of grout pipes, followed by injection grouting. This grouting should significantly reduce piping action along driven piles. The piers would then be founded on HP 310 x 110 steel piles driven to the bedrock surface.

<u>Construction and Workforce Schedule</u>

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS			3	9	5	5	5	5	5	5	2	2		12	22	20	20	20	20	20	40	40	25	25	25	20	30	30	22	30	30	30	40	40	25	25	25	20	25	30	25	2	2		790
PERSON DAYS (21 DAYS/MTH)																																													16,600

Source: Dillon/NDLea 2004

Figure 4.6-17

Conceptual Timeline: PTH 59 North Bridge

4.6.1.8 PTH 44

Figure 4.6-18 presents an aerial view of the existing PTH 44 crossing configuration.

Figure 4.6-18
Aerial View of the Existing PTH 44 Crossing Configuration

In the Iteration 1 Draft Report, the results of the Condition Assessment Findings/Load Rating from PDEA1 and a life cycle cost analysis concluded that this structure should be replaced. An evaluation of the traffic level of service at this site concluded that two traffic lanes were adequate across the Expanded Floodway.

In the Iteration 2 Draft Report, in consultation of the Channel Consultant, the bridge at this site was recommended to be raised above the 1:700 year HWL and lengthened slightly. The bridge was recommended to be replaced on its existing alignment using a low level detour crossing of the Floodway immediately to the south. MTGS reviewed this recommendation and concluded that the risk of losing the low level crossing could not be tolerated since the resulting highway user costs would be too high. The examination of a new alignment was requested.

In the Iteration 3 process, two new alignments of PTH 44 were examined. One alignment was just north of the existing bridge and the other alignment was just south of the existing bridge. In consultation with

MTGS, the alignment just south of the existing bridge was chosen and presented as a preliminary design in the Iteration 3 Draft Report.

Prior to submission of this Technical Appendix, the possibility of providing a medium level detour of PTH 44 across the Floodway using an ACROW Panel bridge was reviewed in an attempt to reduce the associated roadway costs. The cost saving did not appear to justify the risk of the detour and therefore the preliminary design in this Technical Appendix remains as presented in the Iteration 3 Draft Report. This issue may warrant further study during the final design phase of the project.

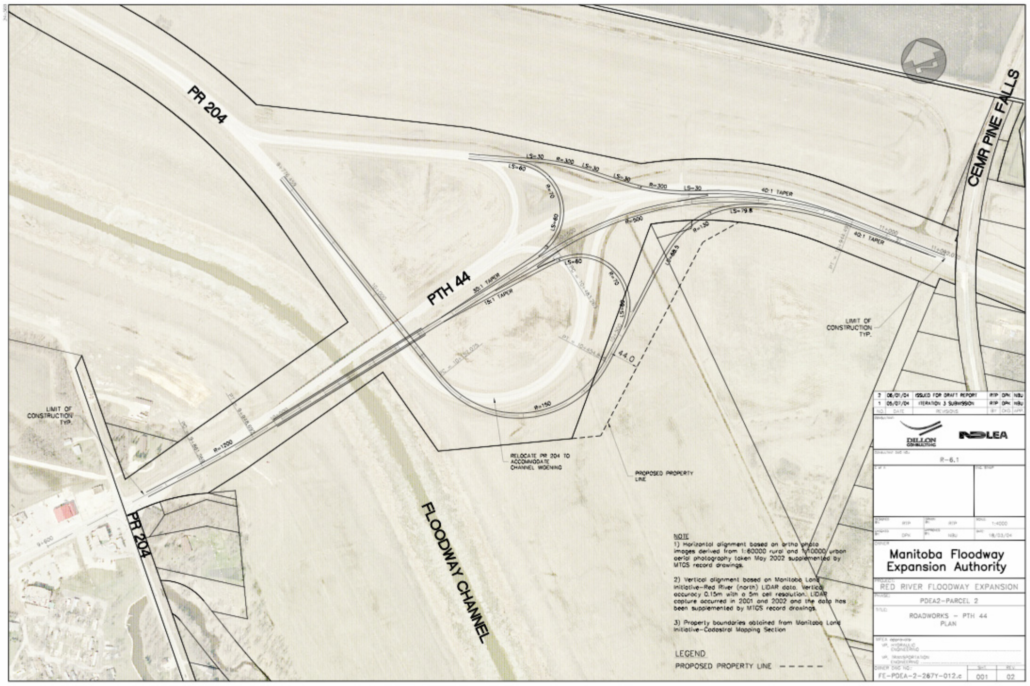
Roadway Classification

The following are the findings for the roadway and bridge evaluation at the PTH 44 Bridge crossing of the Floodway:

- PTH 44 is classified as an Expressway and is currently a two-lane undivided roadway
- PTH 44 is currently, and will continue, operating at a LOS “A” throughout the 20-year design horizon.
- No widening of the roadways or structure is needed at this location.
- No roadway improvements on either side of the existing structure are expected.

Alignment/Geometrics

The alignment of a new crossing of the Expanded Floodway is shown Figure 4.6-19.



Source: Dillone/NDLea 2004

Figure 4.6-19
Horizontal Alignment for new PTH 44 Crossing.

Beginning approximately 40 metres east of PR 204, the proposed alignment continues tangent for approximately 90 m. It departs from the existing alignment, with a 1200 metre curve. A 40 metre tangent portion of roadway is provided on the west side of the new bridge structure. The new roadway and bridge structure cross the Floodway within the curved portion of the channel at approximately a 7o skew. The proposed roadway alignment continues south of the new structure with a 230 metre tangent followed by a 500 metres curve to the south before tying into the existing alignment approximately 180 metres north of the CEMR rail crossing. This proposed roadway alignment will require the relocation of the PR 204 north-south underpass connector road, PR 204 southbound to PTH 44 westbound and eastbound ramp, PTH 44 eastbound to PR 204 northbound ramp, PR 44 eastbound to PR 204 northbound ramp and PTH 44 westbound to PR 204 northbound ramp. The longitudinal approach grades over the bridge are 2% west of the structure, 1.7% east of the structure with the high point just east of midspan. The vertical curve length is 400 m. Lateral crossfall of the deck will be 2% from centreline to each shoulder. The bridge deck will be 10.900 metres wide consisting of two lanes at 3.700 m, two shoulders at 1.750 metres each, and two curbs at 600 mm each. PTH 44 and PR 204 north south connector road cross section will consist of two lanes at 3.700 metres with 2.000 metres shoulders. Ramp widths shall be 4.8 metres with a 1.500 metres shoulder on the left and 2.0 metres shoulder on the right. PR 204 north/south connector road shall consist of two traffic lanes 3.700 metres each, with a 2.000 metres shoulder on each side.

Private property required for the construction of the new approaches and ramps has been estimated at 0.55 ha. Existing Floodway property required for the approaches and ramps has been estimated at 1.75 ha, but no cost has been assigned to the project for this property.

Superstructure

Six equal spans of 43.5 metres centre to centre of bearings for a total length of 261.0 m are planned. The superstructure would consist of five lines of 2000 mm precast prestressed concrete NU girders spaced at 2400 mm on centres. The reinforced concrete deck would be 225 mm thick continuous over the piers for live load. The deck would be constructed using high performance concrete (HPC) with stainless clad or MMFX reinforcing steel based on availability at time of tender.

Steel-reinforced elastomeric bearing pads would be utilized with fixed bearings at Pier SU-4 and expansion bearings at the other piers and abutments. Two multi-seal expansion joints would be provided, one at each abutment. The bridge guardrail will be MTGS Standard TL-4 railing.

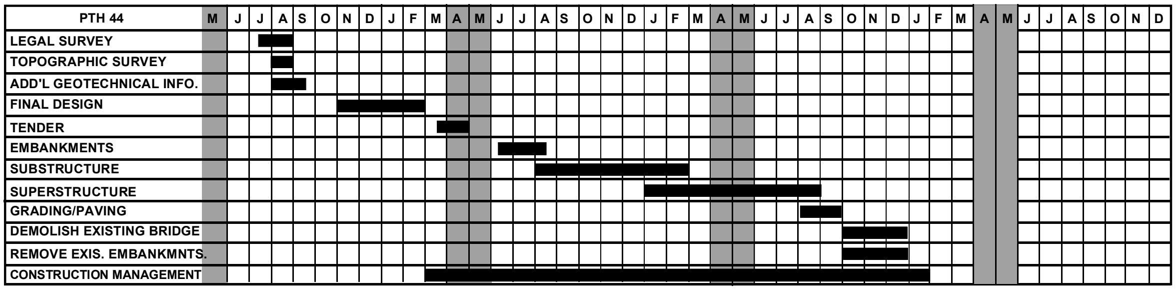
The existing structure has conduits suspended below the deck, some or all of which are occupied by MTS. The new structure should have four - 100 mm diameter conduits suspended below the deck to accommodate MTS. One - 38 mm diameter conduit should be considered in one curb for potential future lighting.

Substructure

The abutments would be reinforced concrete gravity-type founded on HP 310 x 110 steel piles driven to refusal. The piers would also be reinforced concrete. The foundations for Piers SU-2 and SU-6 would be founded on HP 310 x 110 steel piles while Piers SU-3, SU-4, and SU-5 would be spread footings bearing on the till. Ground water release should not pose a problem at this site.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and Contractor's ability to access the foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-20 presents a conceptual schedule for completion of the bridge replacement works for PTH 44.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS			3	9	3		5	5	5	5	1	2		12	12	17	20	20	20	20	40	40	25	25	25	20	20	30	15	20	20	20	2	2											470
PERSON DAYS (21 DAYS/MTH)																																													9,800

Source: Dillon/NDLea 2004

Figure 4.6-20
Conceptual Schedule for PTH 44 Bridge Replacement Program.

4.6.2 Railway Bridges

4.6.2.1 Railway Bridges Design Criteria Summary

Several general design criteria and assumptions were applied in the preliminary railway bridges preliminary design process. The general design data and assumptions are as follows:

Design Codes

- AREMA, as modified by standard CN/CP design practices or site-specific CN/CP requirements.
- MTGS Transportation Planning Manual (Basic Design Standards) for geometrics for affected roadways, with secondary reference to the Transportation Association of Canada (TAC) Geometric Design Guide for Canadian Roads.

Design Loading

- Live Loads for Railway Bridges:
  - Cooper E60 for GWWD and CEMR bridges.
  - Cooper E70 for detour bridges; increased to Cooper E80 for longitudinal forces only.
  - Cooper E80 for CPR bridges.
  - Cooper E90 for CNR bridges, reduced to Cooper E80 for longitudinal forces only.

- Service Life:
  - Bridges are based upon a life span (including maintenance) of 75 years.
  - Traffic projections are based on a 20 year horizon.

Floodway Channel Geometry

- Based on the latest information provided by KGS (April 22, 2004.

Hydraulic Data

- The High Water Level (HWL) represents the 1 in 700 year design flood event.

- Channel design flow is 3960 m3/s (140,000 cfs).
- Design HWL for Railway Bridges is:
  - CPR Emerson = 236.930
  - CNR Sprague = 236.300
  - GWWD Railway = 235.924
  - CNR Redditt = 235.430
  - CPR Keewatin = 234.836
  - CEMR Pine Falls = 231.855
- Channel velocity for Railway bridges:
  - Velocity = 1.52 m/s at CPR Emerson, CNR Sprague, GWWD Railway, and CNR Redditt.
  - Velocity = 2.13 m/s at CPR Keewatin and CEMR Pine Falls.
- Freeboard Clearance for Railway Bridges:
  - All bridges use 300 mm freeboard to HWL to underside of girders.
- Ice impact and ice jams:
  - All bridge piers are designed to accommodate ice loads due to impact of a solitary, freely moving ice floe. Maximum size of floe assumed to be 15 metres diameter by 0.6 metres thickness. The maximum stage for which this is considered possible is 230.4 metres (756 ft) at Floodway Inlet.

Discussion of each bridge crossing site, the site-specific design decisions and the resulting preliminary design is provided for each highway bridge crossing as follows:

4.6.2.2 CPR Emerson

Figure 4.6-21 presents an aerial view of the existing CPR Emerson railway crossing.

Figure 4.6-21
Existing CPR Emerson Rail Crossing.

The successive iterations of PDEA2 have each reaffirmed the recommendation contained in the SAFE Report that this bridge should be replaced with a new bridge. The SAFE Report recommendation to replace the bridge was based on consideration of the cost to retrofit versus the cost of replacement. During the iteration process, we determined that the significant change in vertical geometry caused by raising the girders above the HWL results in railway operational requirements being the governing criterion in the decision to replace the bridge.

The Iteration 2 Draft Report considered three (3) concepts for this site. Two (2) of the concepts involved modifying the bridge on its existing alignment, either by raising the existing girders or replacing them with new spans, all above the HWL. The third concept involved replacing the bridge with a new bridge on a new alignment, at the same higher elevation. The modification concepts were determined either to be deficient in terms of remaining fatigue service life or unacceptable due to severe conflicts with railway operational requirements.

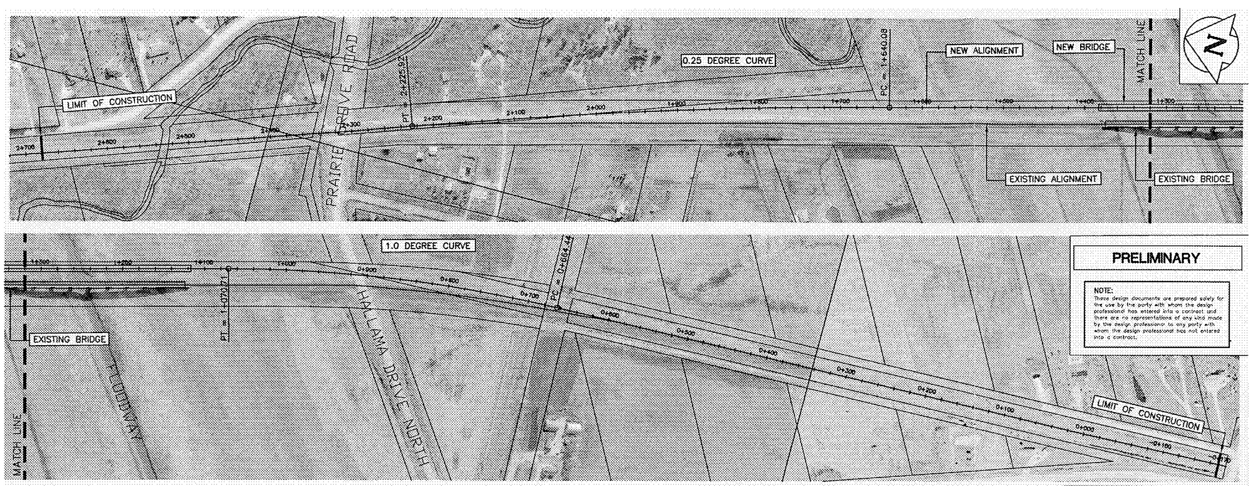
During Iteration 3, the railway company imposed a restriction of 0.3% on the run-out gradient for this site, which significantly increased the scope of associated rail and roadwork.

The existing bridge is located on a long curved section of track, and the natural choice is to realign the track to the east, that is, back towards its original pre-floodway alignment. The existing bridge will remain in service until construction of the new bridge is complete, after which time it will be demolished.

The fatigue-related problems and relatively low load rating for the girders, as reported by the PDEA1 consultant in the Bridge Condition Assessment Reports, means that the existing bridge spans cannot be re-used for long-term loading. However, they are candidates for use in the short-term as part of temporary detour structures at other sites. In fact, cost estimates for the three bridges that will utilize a temporary detour are based on using the worn spans from this bridge as the detour superstructure.

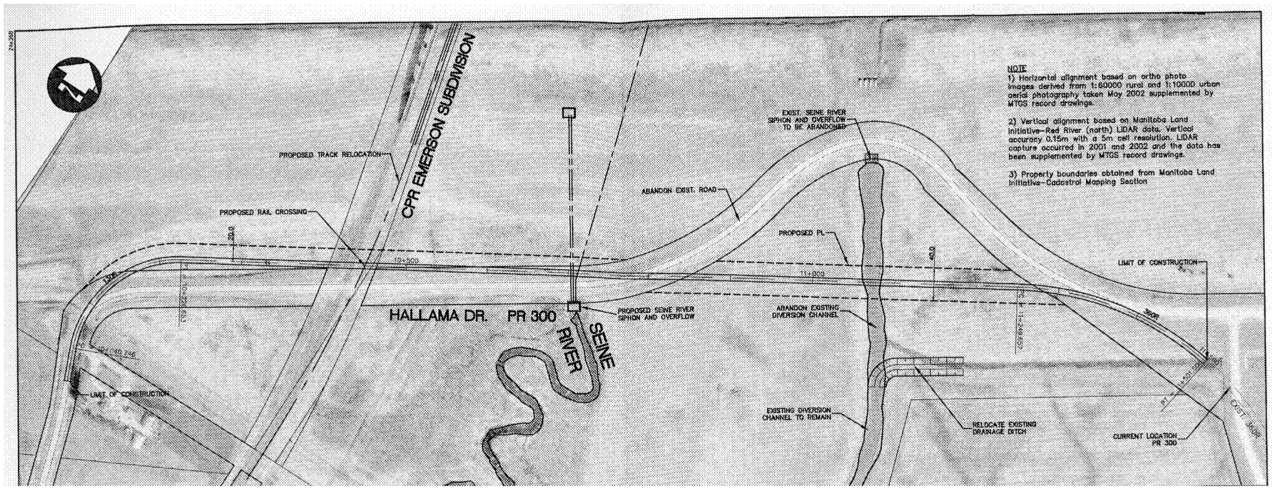
<u>Alignment/Geometrics</u>

Plan information for the recommended rail work is shown Figure 4.6-22. Associated roadworks for the CPR Emerson location are presented in Figure 4.6-23.



Source: Dillon/NDLea 2004

Figure 4.6-22
Plan for CPR Emerson Crossing.



Source: Dillon/NDLea 2004

Figure 4.6-23
Roadworks for CPR Emerson Crossing.

The new bridge will be located to the east of the existing bridge, at a horizontal offset of 20 metres [65’]. The base-of-rail will be raised 2.3 metres [7.5’], and the new bridge will have an overall length of about 267 metres [875’].

Starting at the north tie-in, the horizontal realignment will consist of a 586 metres [1925’] long 0.25^{o} curve, a 569 metres [1870’] long tangent section (which contains the bridge), a 406 metres [1330’] long 1.0^{o} curve, and a short tangent section to the south tie-in. The overall length of track affected by the horizontal realignment is 1665 metres [5460’]. All distances are approximate.

Vertical profile adjustment will start approximately 1315 metres [4315’] north of the bridge, and finish approximately 1300 metres [4265’] south of the bridge. The overall length of track affected by the vertical profile adjustment is approximately 2880 metres [9450’]. However, in the absence of detailed track survey information, it is possible that the vertical geometry tie-in points could vary significantly from those reported here.

The section of track located between the limit of construction and the tie-in at each end of the work must remain in service at all times. Therefore, it will be necessary to incrementally raise these parts of the profile under traffic prior to tieing in the realigned sections of track.

The track already sits several feet above the surrounding landscape as it passes through this area, so the impacts of the planned raising on adjacent property and roads are significant. To the north, Prairie Grove Road needs to be raised 1.05 metres [3.5’], and the private crossing of the track 200 metres south of Prairie Grove Road needs to be raised 1.6 metres [5.2’]. The existing Seine River culvert crossing to the north of Prairie Grove Road is considered to be sufficiently long to accommodate the 0.4 metres [1.3’] raise in base-of-rail at that location. Prairie Grove Road will need to be reconstructed from approximately 190 metres west of the existing rail crossing to approximately 170 metres east of the crossing. The horizontal alignment will generally remain in the same location. Approach grades at the crossing will be

2.5% on the west side and 3.0% on the east side, with a 110 metres long vertical curve. Approach grades at the two private approaches west of the tracks will not be affected. The private approach east of the track will need to be realigned and shifted east in order to minimize the approach grade. The Seine River crossing just east of the tracks will require modifications. The existing crossing consists of a 1.2 metres diameter corrugated steel culvert with cast-in-place concrete headwalls that support a chain-link fence. The culvert will need to be extended approximately 10 metres to accommodate the widened roadway embankment. These modifications may require significant revetment and stabilization work.

To the south, Hallama Drive North needs to be raised 2.3 metres [7.5'] at the rail crossing. Starting from the west, Hallama Drive North will be relocated from the existing curve west of the rail crossing to just west of PTH 59 southbound, tieing into the recently realigned section of PR 300. The alignment in the vicinity of the tracks will be shifted north to provide for a larger radius at the west tie-in. This will also enable the existing roadway to remain in operation during construction. Approach grades at the new railway crossing will be 2.5%, with a 250 metres long vertical curve to accommodate minimum stopping distance. This work will significantly improve sight lines and safety at the rail crossing.

When the Floodway was built, the railway alignment was changed from a sweeping curve to a curve-tangent-curve, with the bridge located on the tangent section. A new right-of-way was established for the railway and, according to the railway company, the old railway right-of-way was left in place. If this is the case, the proposed realignment can likely be accommodated within the Existing Floodway and/or railway rights-of-way. If needed, the horizontal offset of the new alignment can be reduced by several metres to minimize any encroachment beyond the existing railway right-of-way, although this would have implications on the track tie-ins.

Superstructure

The superstructure of the new bridge will consist of 11 equal spans of simply supported ballasted through plate girders, each 24.2 metres [79.5'] long. The design will incorporate the standard CN cross-section for this type of bridge, including internal trainman's walkways, and steel plate deck. Bearings will be the spherical type.

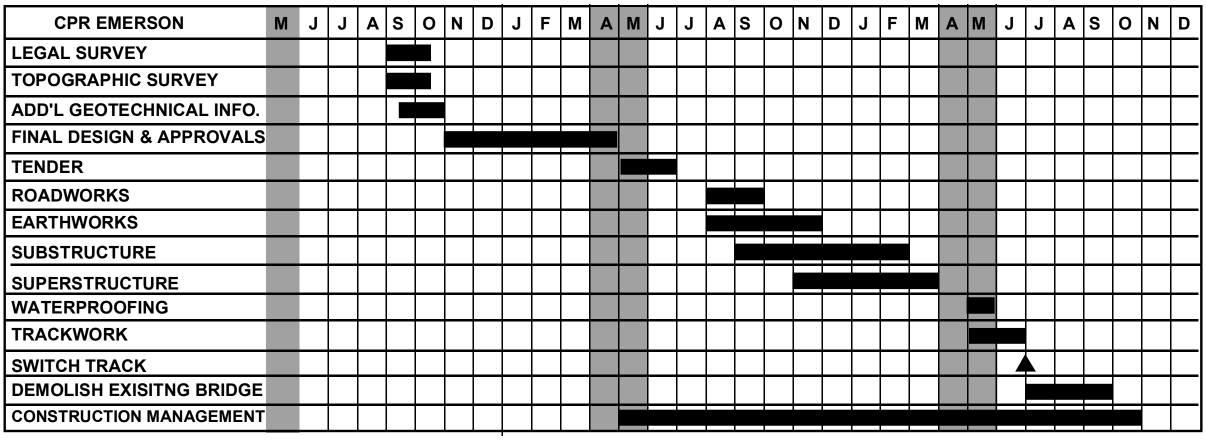
Substructure

Abutments and piers will be constructed of reinforced concrete, founded on steel H-piles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure.

The height of the exposed face of the abutments will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the pier foundation locations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Conceptual scheduling of bridge and roadworks is presented in Figure 4.6-24.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS					8	5	6	6	6	6	6	6	8	8	5	40	65	50	60	40	40	40	15	5	30	25	15	15	15	5			530
PERSON DAYS																																	11,130

Source: Dillon/NDLea 2004

Figure 4.6-24

Conceptual scheduling of bridge and road works for CRP Emerson.

4.6.2.3 CNR Sprague

Figure 4.6-25 presents and aerial view of the existing CNR Sprague rail crossing.

Figure 4-6.25
Existing CNR Sprague rail crossing.

The base concept contained in the SAFE Report called for this bridge to be replaced with a new bridge on an offset alignment, based on consideration of the cost to retrofit versus the cost of replacement. During the iteration process, it was determined that the significant constraint to sight lines imposed by the adjacent PTH 101 overpass, and a strong preference on the part of the railway company not to accept a permanent localized deviation in track alignment, dictates that the bridge needs to be modified on its existing alignment.

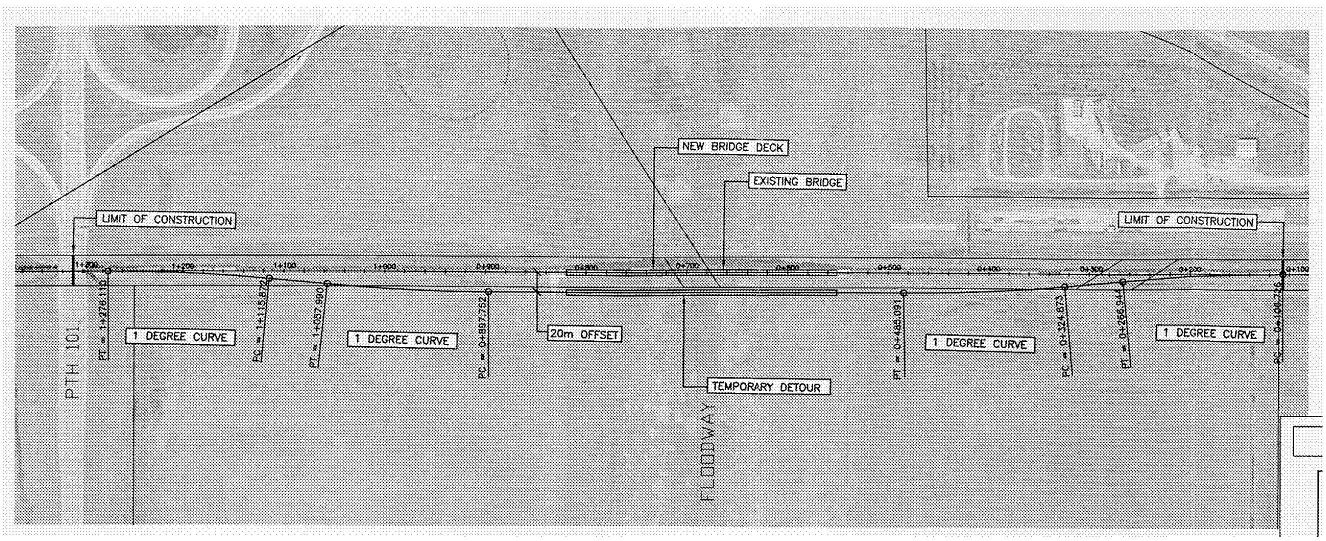
The Iteration 2 Draft Report considered three (3) concepts for this site. Two (2) of the concepts involved modifying the bridge on its existing alignment, either by raising the existing girders or replacing them with new spans, all above the HWL. The third concept involved replacing the bridge with a new bridge on a new alignment, at the same higher elevation. One of the modification concepts was determined not to be viable with regard to vertical geometry, and the plan to realign the bridge was subsequently found to be unacceptable to the railway company.

Therefore, the existing bridge will be modified on its existing alignment, and a temporary detour structure will be constructed to the south of the existing bridge to enable railway operations and the construction work to proceed without interruption. The existing superstructure and two of the piers will be replaced, and new abutments will be built.

The original design loading for the existing deck plate girders was Cooper E60. Therefore, the existing girders do not meet the required Cooper E70 rating for use in the short-term as part of a temporary detour structure at other sites (upgrading is not economical, although this may need to be revisited in the final design if these existing spans can be released before those at CPR Emerson).

<u>Alignment/Geometrics</u>

Plan information for the recommended rail work is shown in Figure 4.6-26. There is no associated roadwork at this site.



Source: Dillon/NDLea 2004

Figure 4.6-26
CNR Sprague Bridge Modifications Plan

The alignment of the existing bridge will be retained, and the base-of-rail will be raised 0.5 metres [1.6']. The new bridge will have an overall length of about 289 metres [942'].

The temporary detour will be located to the south of the existing bridge, at a horizontal offset of 20 metres [65']. The vertical profile of the detour will replicate the vertical geometry of the existing track. The horizontal alignment of the detour will take the form of a shoo-fly, about 1170 metres [3835'] long, with 1.0^{o} curves. The detour structure will be located on tangent track.

Vertical profile adjustment of the existing track will start approximately 490 metres [1610'] west of the bridge, and finish approximately 300 metres [985'] east of the bridge. The overall length of track affected by the vertical profile adjustment is approximately 1075 metres [3525']. However, in the absence of detailed track survey information, it is possible that the vertical geometry tie-in points could vary from those reported here.

There are no impacts on existing roads due to the vertical profile adjustment at this site.
It appears that the proposed alignment of the temporary detour can be accommodated within the Existing Floodway and/or railway rights-of-way.

Superstructure

The new superstructure will consist of 11 spans of simply supported ballasted through plate girders, with individual span lengths to match the spacing of the existing substructure units. The existing end spans will be replaced with longer girders to accommodate the proposed channel geometry and raised base-of-rail, so new abutments will be required. The girders will be haunched at the piers, to a depth equal to two-thirds of the basic girder height.

The design will incorporate the standard CN cross-section for this type of bridge, including internal trainman's walkways, and steel plate deck. Bearings will be the spherical type, and shock transmission units (lock-up devices) will be used at each expansion joint.

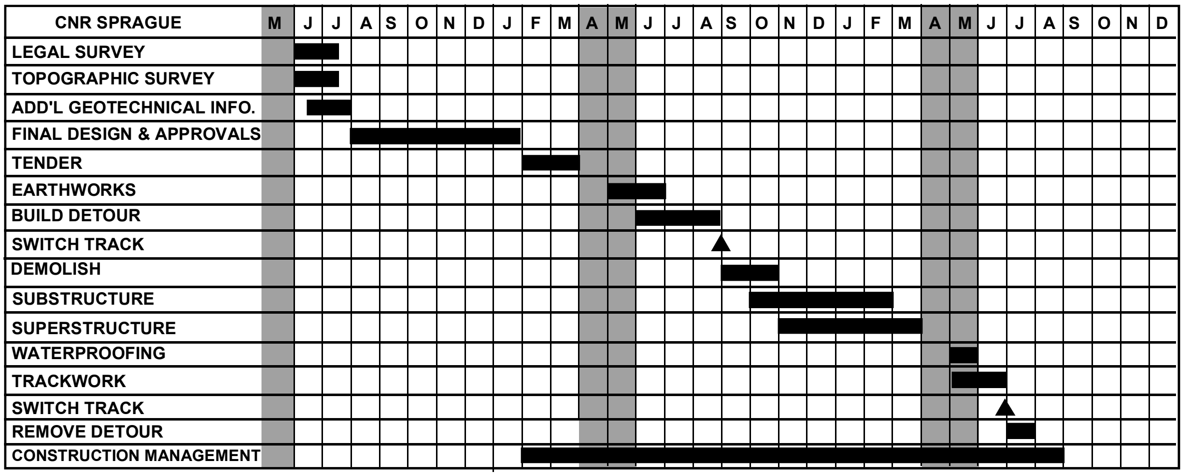
Substructure

The existing abutments will be demolished, and replaced with new reinforced concrete abutments set 9 metres [30'] behind the old ones. The new abutments will be founded on steel Hpiles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure. The height of the exposed face of the new abutments will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.

Proposed changes to the channel cross-section will expose the entire pilecap and upper section of the piles at Piers SU-2 and SU-9. Therefore, these two piers and their pilecaps will be demolished, and reconstructed at a lower elevation on the existing piles to provide suitable frost protection. The remaining piers will have their tops demolished and rebuilt about 1.2 metres [4'] higher and 3.2 metres [10.5'] longer using reinforced concrete, to accommodate the replacement superstructure. The pilecaps at Piers SU-1, SU-6, SU-8, and SU-10 will require additional frost protection.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the piers and foundations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-27 presents a conceptual scheduling for work associated with the CNR Sprague rail crossing.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS		8	7	5	5	5	5	5	5	8	8	5	15	35	25	25	15	35	35	35	35	35	15	5	30	25	15	5					451
PERSON DAYS																																	9,471

Source: Dillon/NDLea 2004

Figure 4.6-27

Conceptual Scheduling: CNR Sprague Rail Crossing

4.6.2.4 Greater Winnipeg Water District (GWWD)

Figure 4.6-28 presents an aerial view of the existing GWWD Rail Bridge adjacent to the Deacon Reservoir facility.

Figure 4-6.28
Existing GWWD Rail Bridge

The base concept contained in the SAFE Report called for the existing spans of this bridge to be raised approximately 2.4 metres [8'] on the existing alignment, based on consideration of the cost to retrofit versus the cost of replacement. During the iteration process, it was determined that the significant impact on PTH 101 to the west, and the relatively large amount of associated rail work in both directions, justified changing the existing concrete I-girder superstructure to shallower through plate girders.

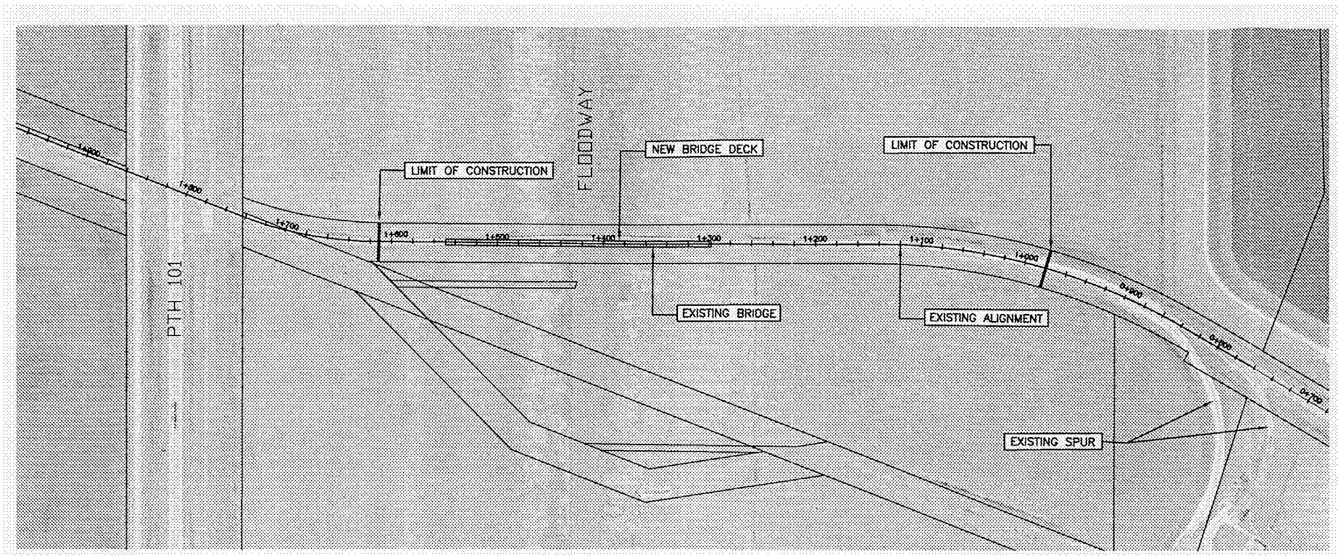
The Iteration 2 Draft Report considered only the original SAFE Report concept for this site, but this was changed to the through plate girder concept in the Iteration 3 Draft Report.

Therefore, the existing bridge will be modified on its existing alignment. The bridge will need to be closed for a period of four months to enable the modification work to proceed without interruption. During this time, the existing superstructure, two piers, and an abutment will be replaced, a new pier and abutment will be built, and two new spans will be added. Additional costs for alternative arrangements for the transportation of goods during the bridge closure period have been included in the cost estimate.

GWWD has stated that the modification work for this bridge must be complete by the end of 2006 so as not to interfere with planned development work at the Deacon Reservoir site.

Alignment/Geometrics

Plan information for the recommended rail work is shown in Figure 4.6-29. There is no associated roadwork at this site. The alignment of the existing bridge is being retained. The base-of-rail will be raised 0.6 metres [2.1'], and the new bridge will have an overall length of about 303 metres [993'].



Source: Dillon/NDLea 2004

Figure 4.6-29
Plan for GWWD Rail Crossing

Vertical profile adjustment of the existing track will start approximately 250 metres [820'] west of the bridge, and finish approximately 260 metres [850'] east of the bridge. The overall length of track affected by the vertical profile adjustment is approximately 815 metres [2675']. However, in the absence of detailed track survey information, it is possible that the vertical geometry tie-in points could vary from those reported here.

There are no impacts on existing roads due to the vertical profile adjustment at this site.

Superstructure

The new superstructure will consist of 13 spans of simply supported ballasted through plate girders, with individual span lengths to match the spacing of the existing substructure units, and two new spans at the east end of the bridge. The girders will be haunched at the piers and west abutment, to a depth equal to two-thirds of the basic girder height.

The design will incorporate the standard CN cross-section for this type of bridge, including internal trainman's walkways, and steel plate deck. Bearings will be the spherical type, and shock transmission units (lock-up devices) will be used at each expansion joint.

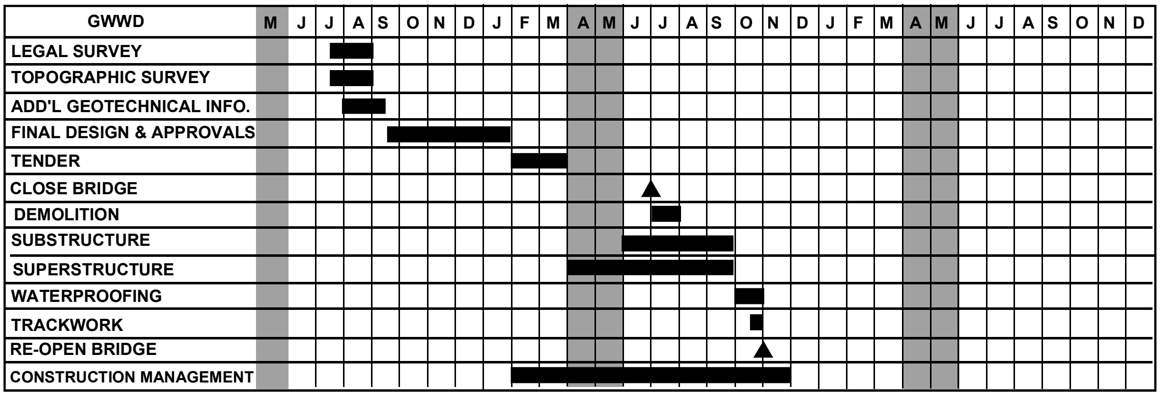
Substructure

The existing west abutment will be modified (strengthened and widened) to accommodate the new superstructure. The east abutment will be demolished, and replaced with a new reinforced concrete abutment set 52 metres [170'] behind the old one. The new abutment will be founded on steel H-piles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure. The height of the exposed face of the new abutment will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.

Proposed changes to the channel cross-section on the east side will expose the entire pilecap and upper section of the piles at Pier SU-1, and will expose all but 0.3 metres [1'] of the pilecap at Pier SU-2. Therefore, these two piers and their pilecaps will be demolished and reconstructed at a lower elevation on the existing piles to provide suitable frost protection. Additional piles will be driven to refusal and a new pier will be built on these and the existing piles at the location of the existing east abutment. A completely new pier will be built between the old and new west abutment locations. The remaining piers will have their tops demolished and rebuilt about 0.7 metres [2.4'] higher and 4 metres [13'] wider using reinforced concrete, to accommodate the replacement superstructure. The pilecap at Pier SU-5 will require additional frost protection.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the piers and foundations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-30 presents conceptual scheduling information for the GWWD Rail Crossing.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS			3	10	5	5	5	5	5	8	8	15	15	55	70	55	55	25	5														349
PERSON DAYS																																	7,329

Source: Dillon/NDLea 2004

Figure 4.6-30
Conceptual scheduling for GWWD Rail Crossing

Figure 4.6-31 presents an aerial view of the existing CNR Redditt Rail Bridge.

Figure 4.6-31
Existing CNR Redditt Rail Crossing (shown at left)

The base concept contained in the SAFE Report called for this deck plate girder bridge to be left in place, with new longer end spans, based on consideration of the cost to retrofit versus the cost of replacement. This arrangement left the girders partially submerged during the design flood event.

The Iteration 2 Draft Report considered three (3) concepts for this site, all of which retained the bridge on its existing alignment. In addition to the partially submerged SAFE Report concept, the other two (2) concepts involved either raising the existing deck plate girders by 1.5 metres [5'] or replacing them with new through plate girder spans with no change in base-of-rail elevation, each above the HWL. The PDEA2 Lead Consultant subsequently identified significant savings in channel excavation costs associated with raising all of the Floodway bridges above the HWL. Given the significant impact on PTH 101 to the west and the relatively large amount of rail work in both directions associated with raising the existing girders, the recommendation for this site is to replace the existing superstructure with new through plate girders.

The Iteration 2 Draft Report also contemplated the use of a change-out strategy at this site, as opposed to using a detour structure. This plan subsequently did not meet with railway company approval.

Therefore, the existing bridge will be modified on its existing alignment, and a temporary detour structure will be constructed to the north of the existing bridge to enable railway operations and the construction work to proceed without interruption. The existing superstructure and four of the piers will be replaced, the existing abutments will be replaced with new piers, and new abutments will be built.

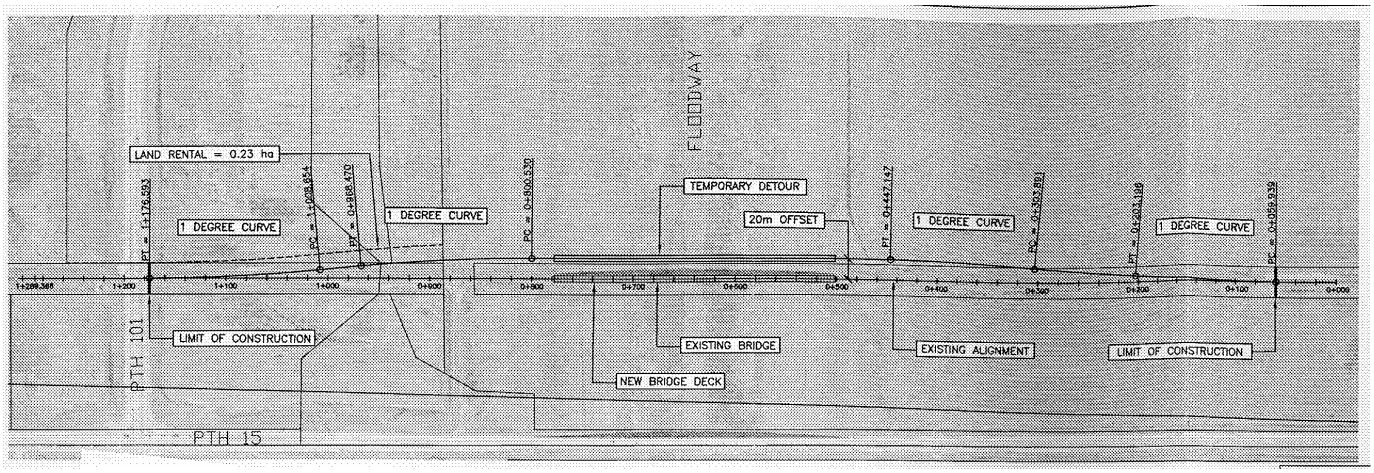
Depending on property ownership, it may be necessary to rent a small amount of up to three parcels of land (total 0.23 ha) at the west end of the bridge for about 15 months for occupation by the temporary detour.

The original design loading for the existing deck plate girders was Cooper E60. Therefore, the existing girders do not meet the required Cooper E70 rating for use in the short-term as part of a temporary detour structure at other sites (upgrading is not economical, although this may need to be revisited in the final design if these existing spans can be released before those at CPR Emerson).

CN has scheduled conversion of the existing bridge superstructure to a ballasted deck in 2005, which coincides with the need to replace the existing ties. If the bridge modifications associated with the Floodway expansion are not carried out in 2005, CN will spot change the ties instead. According to CN, the cost of this work is chargeable to the Province. Since worn spans will not have been salvaged from other bridges in the program in time for use in a detour at this site in 2005, it is unlikely this bridge will be modified until after the CN cut-off date.

Alignment/Geometrics

Plan information for the recommended rail work is shown in Figure 4.6-32.



Source: Dillon/NDLea 2004

Figure 4.6-32
Plan details for CNR Redditt

The alignment of the existing bridge is being retained, as is the base-of-rail elevation. The new bridge will have an overall length of about 305 metres [1001'].

The temporary detour will be located to the north of the existing bridge, at a horizontal offset of 20 metres [65']. The vertical profile of the detour will replicate the vertical geometry of the existing track. The horizontal alignment of the detour will take the form of a shoo-fly, about 1115 metres [3660'] long, with 1.0^{o} curves. The detour structure will be located on tangent track.

It appears that the proposed alignment of the temporary detour can be accommodated within the Existing Floodway and/or railway rights-of-way, with the exception of three parcels at the west end of the bridge that may require rental of a total of about 0.23 **ha** of land for approximately 15 months.

Superstructure

The new superstructure will consist of 11 spans of simply supported ballasted through plate girders, with individual span lengths to match the spacing of the existing substructure units. The existing end spans will be replaced with longer girders to accommodate the proposed channel geometry. The girders will be haunched at the piers, to a depth equal to two-thirds of the basic girder height.

The design will incorporate the standard CN cross-section for this type of bridge, including internal trainman's walkways, and steel plate deck. Bearings will be the spherical type, and shock transmission units (lock-up devices) will be used at each expansion joint.

Substructure

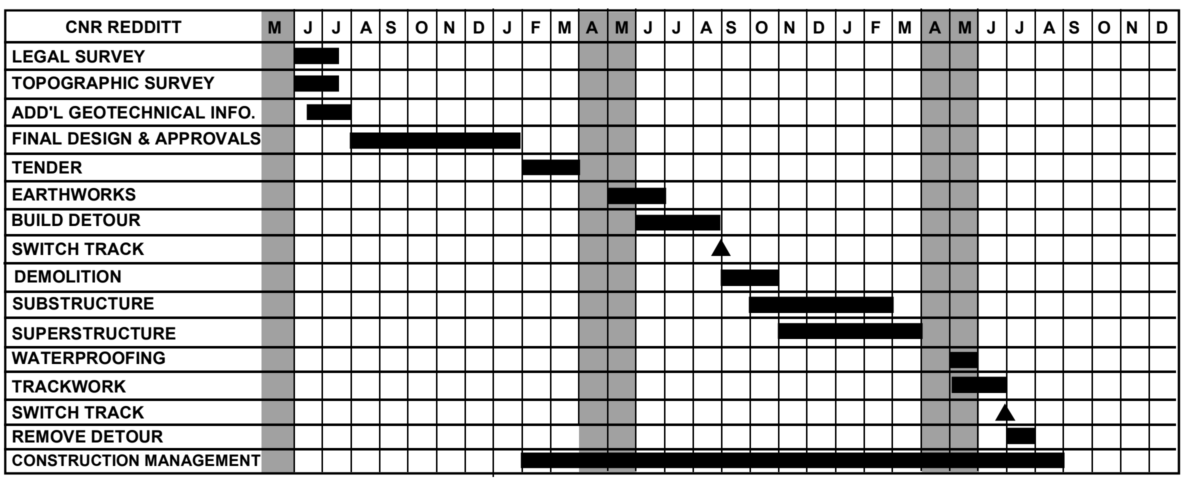
The existing abutments will be demolished, and replaced with new reinforced concrete abutments set 15 metres [49'] behind the old ones. The new abutments will be founded on steel H-piles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure. The height of the exposed face of the new abutments will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.

Proposed changes to the channel cross-section will expose the entire pilecap and upper section of the piles at Piers SU-1, SU-2, SU-9, and SU-10. Therefore, these four piers and their pilecaps will be demolished, and reconstructed at a lower elevation on the existing piles to provide suitable frost protection. The remaining piers will have their tops demolished and rebuilt about 0.6 metres [2'] higher and 3.6 metres [12'] wider using reinforced concrete, to accommodate the replacement superstructure. The pilecap at Pier SU-6 will require additional frost protection.

Defragmentation of the bedrock through injection grouting is recommended for the piled foundations at this site, and this has been included in the cost estimate.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the piers and foundations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-33 presents conceptual scheduling for CNR Redditt construction works.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS		8	7	6	6	6	6	6	6	8	8	5	15	35	25	25	15	35	35	35	35	35	15	5	30	25	15	5					457
PERSON DAYS																																	9,597

Source: Dillon/NDLea 2004

Figure 4.6-33
Conceptual scheduling for CNR Redditt Rail Crossing

4.6.2.5 CPR Keewatin

Figure 4.6-34 presents an aerial view of the existing CPR Keewatin rail crossing.

Figure 4.6-34
Existing CPR Keetwatin Rail Crossing.

The base concept contained in the SAFE Report called for this deck plate girder and concrete cradle bridge to be left in place, based on consideration of the cost to retrofit versus the cost of replacement. This arrangement left the girders partially submerged during the design flood event.

The Iteration 2 Draft Report considered three (3) concepts for this site, all of which retained the bridge on its existing alignment. In addition to the partially submerged SAFE Report concept, the other two (2) concepts involved either raising the existing superstructure by 1.2 metres [4'] or replacing it with similar but less deep spans with no change in base-of-rail elevation, each above the HWL. The PDEA2 Lead Consultant subsequently identified significant savings in channel excavation costs associated with raising all of the Floodway bridges above the HWL. Raising the existing superstructure proved not possible due to the constraint imposed by the adjacent PTH 101 overpass structure to the west, so the recommendation for this site is to replace the existing superstructure with one that is similar but less deep.

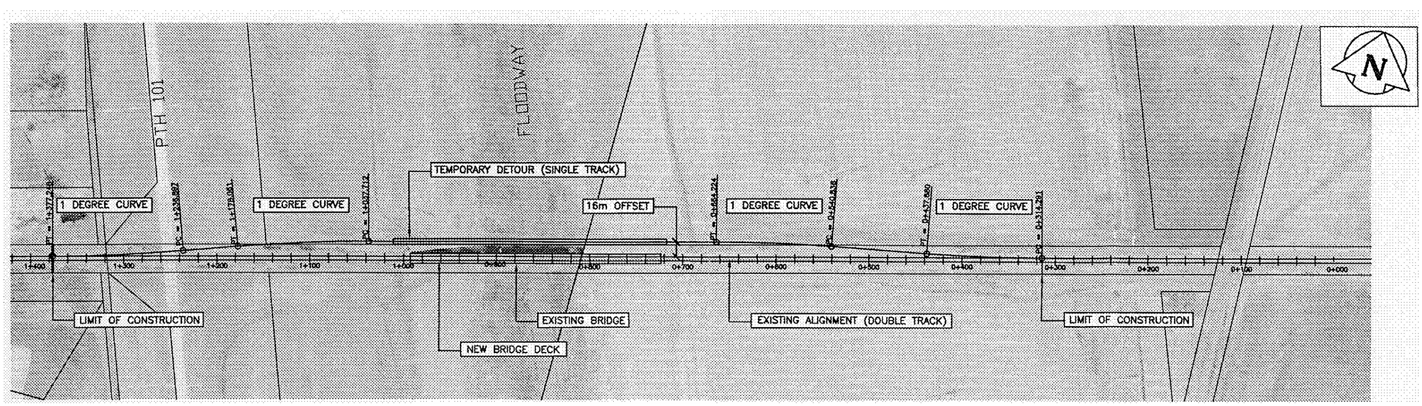
The Iteration 2 Draft Report also contemplated the use of a change-out strategy at this site, as opposed to using a detour structure. This plan subsequently did not meet with railway company approval.

Therefore, the existing double-track bridge will be modified on its existing alignment, and a temporary single-track detour structure will be constructed to the north of the existing bridge to enable railway operations and the construction work to proceed without interruption. The existing superstructure will be replaced, the existing abutments will be replaced with new piers, new abutments will be built, and two new spans will be added.

In order to accommodate the construction, CP will install a power-controlled crossover to the east of the detour so that both tracks can be utilized for staging trains. The cost of this crossover has been included in the cost estimate.

Alignment/Geometrics

Plan information for the recommended rail work is shown in Figure 4.6-35. There is no associated roadwork at this site.



Source: Dillon/NDLea 2004

Figure 4.6-35
Plan detail for CPR Keewatin Rail Crossing

The alignment of the existing bridge is being retained, as is the base-of-rail elevation. The new bridge will have an overall length of about 293 metres [962'].

The temporary detour will be located to the north of the existing bridge, at a horizontal offset of 16 metres [52.5']. The vertical profile of the detour will replicate the vertical geometry of the existing track. The horizontal alignment of the detour will take the form of a shoo-fly, about 1065 metres [3495'] long, with 1.0^{o} curves. This geometry takes account of the proposed twinning structure at PTH 101. The detour structure will be located on tangent track.

It appears that the proposed alignment of the temporary detour can be accommodated within the Existing Floodway and/or railway rights-of-way.

Superstructure

The new superstructure will consist of 13 spans of simply supported steel deck plate girders and composite concrete cradle, with two trainman's walkways. Individual span lengths will match the spacing of the existing substructure units, and new end spans will be added to the bridge.

Bearings will be the spherical type, and shock transmission units (lock-up devices) will be used at each expansion joint.

Substructure

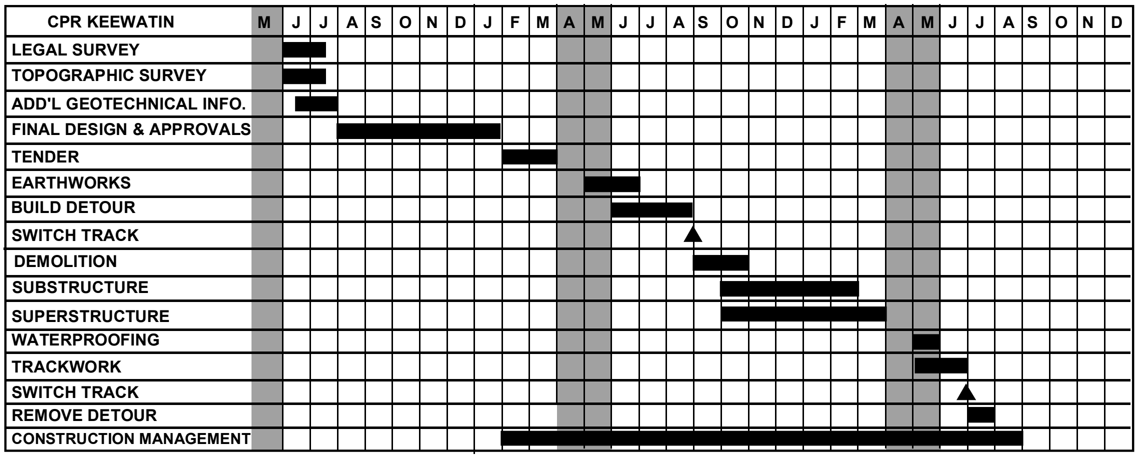
The existing abutments will be demolished, and replaced with new reinforced concrete abutments set 13 metres [42'] behind the old ones. The new abutments will be founded on steel H-piles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure. The height of the exposed face of the new abutment will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.

Proposed changes to the channel cross-section will not adversely affect any of the existing pier pilecaps. Additional piles will be driven to refusal at the location of the existing abutments, and new piers will be built on these and the existing piles at an elevation that will provide suitable frost protection. The existing piers will have their tops demolished and rebuilt about 1.2 metres [4'] higher and 1.2 metres [4'] longer using reinforced concrete, to accommodate the replacement superstructure. The pilecap at Pier SU-5 may require additional frost protection.

Defragmentation of the bedrock through injection grouting is recommended for the piled foundations at this site, and this has been included in the cost estimate.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the piers and foundations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-36 presents conceptual scheduling for the CPR Keewatin Rail Crossing construction.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS		8	7	8	8	8	8	8	8	8	8	5	15	35	25	25	15	45	35	35	35	35	15	5	30	25	15	5					479
PERSON DAYS																																	10,059

Source: Dillon/NDLea 2004

Figure 4.6-36

Conceptual scheduling for CPR Keewatin Rail Crossing

4.6.2.6 CEMR Pine Falls

Figure 4-6.37 presents an aerial view of the existing CEMR Pine Falls Rail Crossing.

Figure 4.6-37
Existing CEMR Pine Falls Rail Crossing

The base concept contained in the SAFE Report called for this deck plate girder bridge to be left in place, with new longer end spans, based on consideration of the cost to retrofit versus the cost of replacement. This arrangement left the girders partially submerged during the design flood event.

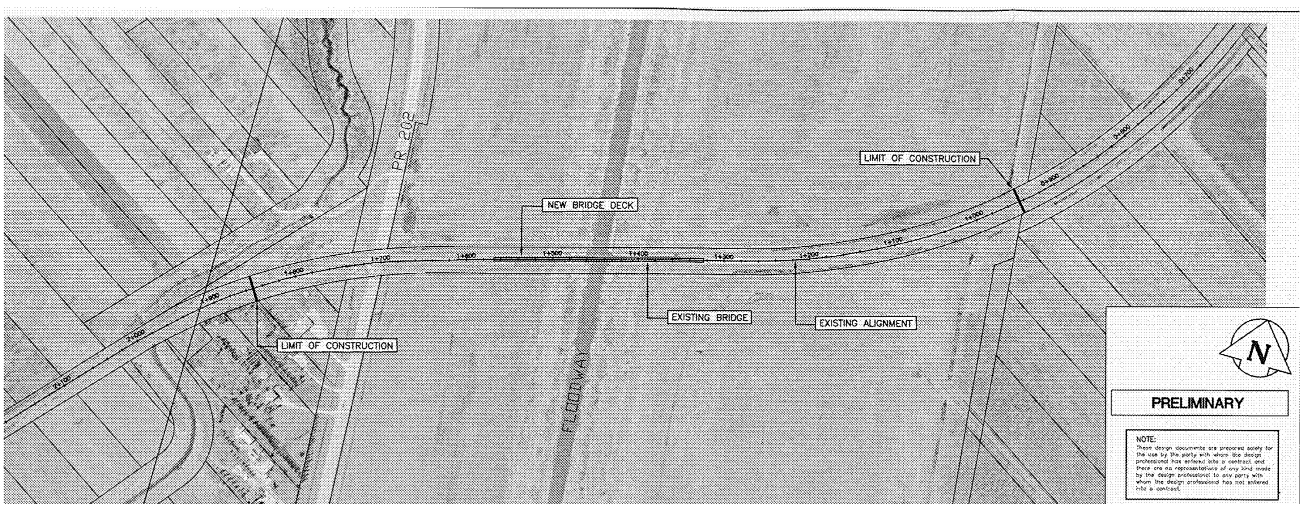
The Iteration 2 Draft Report considered raising the existing girders 2.3 metres [7.5'] to be above the HWL, and the PDEA2 Lead Consultant subsequently identified significant savings in channel excavation costs associated with raising all of the Floodway bridges above the HWL. During Iteration 3, IT WAS determined that the significant impact on PR 202 to the west and the relatively large amount of associated rail work in both directions caused by raising the existing girders justified changing the existing superstructure to shallower through plate girders. The Iteration 3 Draft Report also considered replacing the bridge on a new alignment. However, the cost of associated rail work remained high, which led to the recommendation to replace the existing superstructure with new through plate girders on the existing alignment.

Therefore, the existing bridge will be modified on its existing alignment. The bridge will need to be closed for a period of four months to enable the modification work to proceed without interruption. During this time, the existing superstructure will be replaced, the end spans will be lengthened, and two new piers and two new abutments will be built. Additional costs for alternative arrangements for the transportation of goods during the bridge closure period have been included in the cost estimate.

The original design loading for the existing deck plate girders was Cooper E60. Therefore, the existing girders do not meet the required Cooper E70 rating for use in the short-term as part of a temporary detour structure at other sites (upgrading is not economical, although this may need to be revisited in the final design if these existing spans can be released before those at CPR Emerson).

Alignment/Geometrics

Plan information for the recommended rail work is shown in Figure 4.6-38.



Source: Dillon/NDLea 2004

Figure 4.6-38
Plan detail for CEMR Pine Falls Rail Crossing

The alignment of the existing bridge will be retained, and the base-of-rail will be raised 0.5 metres [1.6']. The new bridge will have an overall length of about 256 metres [839'].

Vertical profile adjustment of the existing track will start approximately 285 metres [935'] west of the bridge, and finish approximately 375 metres [1230'] east of the bridge. The overall length of track affected by the vertical profile adjustment is approximately 915 metres [2990']. However, in the absence of detailed track survey information, it is possible that the vertical geometry tie-in points could vary from those reported here.

PR 202 will have to be raised about 0.5 metres [1.6']. Existing stopping site distance is substandard due to the abrupt rail crossing approach grades. In addition, there are several private driveways located in close proximity of the crossing, resulting in substandard site distance to those entering the roadway from the approaches and those traveling on PR 202. In order to achieve minimum design site distances, maintain traffic during construction and improve private access, it is recommended that PR 202 be realigned to the east from about 500 south of the crossing to about 600 north of the crossing. A new service road south of the crossing will consolidate access from private approaches with a new access point about 250 metres south of the crossing. Approach grades to the crossing will be 1.2%, with a 260 metres long vertical curve.

Superstructure

The new superstructure will consist of 11 spans of simply supported ballasted through plate girders, with individual span lengths to match the spacing of the existing substructure units, and new longer end spans to accommodate the proposed channel geometry and raised base-of-rail.

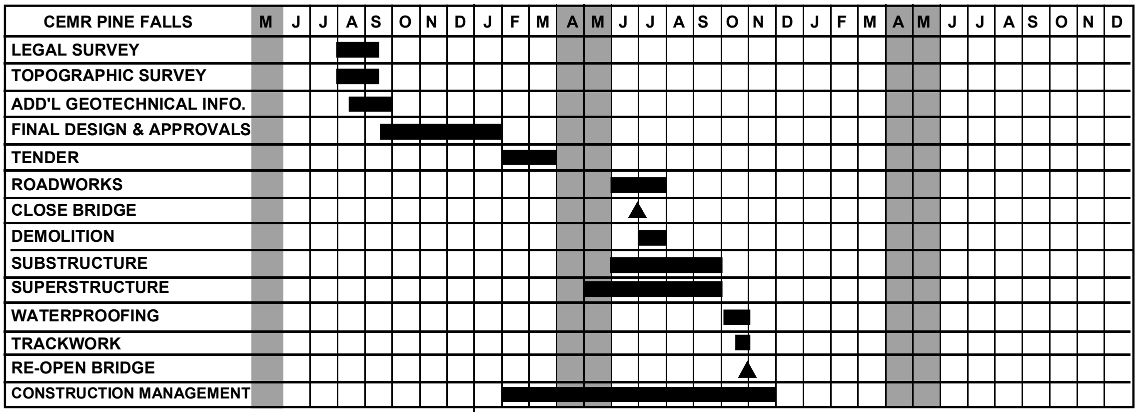
The design will incorporate the standard CN cross-section for this type of bridge, including internal trainman's walkways and steel plate deck. Bearings will be the spherical type, and shock transmission units (lock-up devices) will be used at each expansion joint.

Substructure

The existing abutments will be demolished, and replaced with new reinforced concrete abutments set 6 metres [20'] behind the old ones. The new abutments will be founded on steel H piles driven to refusal. Batter piles will be used to resist lateral earth pressure and longitudinal forces transmitted from the superstructure. The height of the exposed face of the new abutment will be in the order of 0.9 metres [3'], and fill slopes in the immediate vicinity of the abutment wingwalls will be increased to 3h:1v.
Piers SU-1 and SU-10 will be abandoned, and replaced with new piers located 5.8 metres [19'] in front of the existing piers. The remaining piers will have their tops extended about 2.1 metres [7'] higher and 2.4 metres [8'] wider, to accommodate the replacement superstructure.

Construction and Workforce Schedule

The construction schedule will be subject to the influences of weather and the Contractor's ability to access the piers and foundations. Seasonal operation of the Floodway in April/May or potentially during the summer months may significantly affect progress. Figure 4.6-39 presents a conceptual schedule for completion of the CEMR Pine Falls Rail Crossing modifications.



ESTIMATED WORKFORCE	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	TOTAL
PERSON MONTHS				8	10	5	5	5	5	8	8	5	15	70	85	55	55	25	5														369
PERSON DAYS																																	7,749

Source: Dillon/NDLea 2004

Figure 4.6-39
Conceptual scheduling – CEMR Pine Falls Rail Crossing

4.7 CITY OF WINNIPEG BRANCH AQUEDUCTS

The City of Winnipeg is served by two aqueducts conveying drinking water to the Deacon Reservoir from the source at Shoal Lake. The Branch I Aqueduct was originally constructed in 1918-1919 while the Branch II Aqueduct was constructed in 1959. In 1966 both Branch Aqueducts were relocated to their current location in the vicinity of the Floodway to accommodate the original Floodway construction. Both aqueducts will require reconstruction to accommodate the Expanded Floodway Channel.

Branch I connects the Main Aqueduct and the Deacon Reservoir to the McPhillips and McLean Distribution System Reservoirs. Branch II connects the Main Aqueduct and Deacon Reservoir to the McLean and Wilkes Avenue Reservoirs. Both Branch I and Branch II were redesigned to cross the Existing Floodway Channel. Based upon initial cross sections of the expanded Floodway Channel conceptual designs, both Branch Aqueducts will require considerable modification as they will either be exposed in the proposed widened channel or have their current depth below ground surface reduced to an unacceptable level.

It is imperative that the re-construction of the Branch Aqueducts at the Floodway crossing carefully and fully consider all aspects of short and long term stability of the Branch I and Branch II Aqueducts. The Branch Aqueducts convey all of the City of Winnipeg's water supply from the Deacon Reservoir and as such, are considered critical infrastructure. There is a finite capacity within the three distribution reservoirs within the City limits to provide a water supply in the event of an unplanned loss of service of one or both Branch Aqueducts. There is also limited capacity within the distribution reservoirs to provide balancing storage during time periods when the Branch Aqueducts will be removed from service for reconstruction across the Floodway channel. Even with all upgrading proposed to be undertaken to the Branch I Aqueduct between now and the commissioning of the Water Treatment Plant (WTP) in 2007, there is not enough supply capacity in the Branch I Aqueduct to meet average daily demand within the

City under all operating conditions. In situations when the Branch II Aqueduct is removed from service, the entire City can be operating in a depleting storage mode under certain demand scenarios.

A set of design criteria for Iteration #1 has been submitted by the consultant. The criteria include:

- Minimum finished height of cover (depth of soil overlaying top of Aqueduct) of 1.8 metres.
- Realignment of the Branch I Aqueduct back to its original alignment prior to the Floodway construction (this represents the shortest possible length of realignment).
- Realignment of the Branch II Aqueduct to an alignment parallel to its pre-Floodway alignment but sufficiently offset from the existing pipe to maximize the time period that the existing Branch II Aqueduct may remain in service during construction.
- Vertical alignment selection and relocation considerations that prevent siting the Aqueduct in an area that is susceptible to slope movement.

The realignment options for Branch II of the Aqueduct are heavily influenced by operating considerations as City of Winnipeg requirements dictate that this branch can only be removed from service for relatively short timeframes during specific operating windows. The Branch I Aqueduct can and has been removed from service in the past for longer timeframes and is less constrained by reconnection considerations. Nonetheless, the City of Winnipeg restricts construction on both Aqueducts to limited construction windows that must be considered in the design for realignment of the Branch Aqueducts.

Both Branch Aqueducts will require replacement in the lower channel of the Floodway, replacement in the Floodway Channel sideslope, replacement in the natural grade areas of the Floodway rights-of-way, and installation and testing of air release and access chambers. Preliminary geotechnical analyses have indicated that work in the lower channel of the Floodway will require extensive **dewatering** and foundation stabilization to facilitate construction as well as temporary diversion of the existing watercourse. Results from a 1965 drilling program indicate that dense till contact in the vicinity of the Branch II Aqueduct appears to be substantially lower than in the vicinity of the Branch I Aqueduct. Results from this drilling program suggest that dense till will be encountered in the vicinity of the Branch I Aqueduct with channel deepening. Dense till will not be encountered in the vicinity of the Branch II Aqueduct until the channel is deepened and additional 3m or more beyond the depth stated in the proposed SAFE Study conceptual design.

Several design Iterations were undertaken to identify the impact that varying channel geometries would have on the Branch Aqueducts. Through this iterative process, channel geometry changes to accommodate other project areas (i.e. channel hydraulics, bridge design etc.) could be evaluated in terms of feasibility, cost and construction scheduling/sequencing.

A total of five aqueduct reconstruction approaches were assessed by the design consultants. Each option involved variances in channel side slope in the region of the aqueducts, and different branch aqueduct alignments. Design options descriptions and assessments are available in the "Preliminary Engineering Report: Appendix D: Outlet, Local Drainage and Syphons Pre-Design".

The draft Iteration 3 Report concluded that the most economical alternatives for the combined Branch I /GWWD Bridge are Case 1 and Case 5, both estimated at $5.8 Million.

Case 1 assumes that the 1v:9h channel side slopes are carried past the bridge to eliminate the need for any slope stabilization works. This requires that the bridge be widened an additional 20 m on each side. Although in this case there is no requirement for slope stabilization, there is a cost ($1.8M) for extending the bridge. There are also likely to be costs associated with raising the grade at the PTH 101 rail crossing to maintain a maximum rail grade of 0.5%. Total costs are estimated to be $5.8 Million (exclusive of the Branch II reconstruction).

Case 5 assumes the original Aqueduct alignment but widening the east side of the channel by 40 m with 1v:9h sideslopes carried past the bridge. No slope stabilization is required and there will be no impact on the railway grade crossing PTH 101. The bridge will have to be lengthened by 40 m on the east side ($1.8 Million) Costs for Case 5 are estimated to be $5.8 Million (exclusive of the Branch II reconstruction).

Both of these alternatives share the advantage that there are no slope stabilization works required and it is unnecessary to realign the Branch I Aqueduct from the original (Iteration 2) alignment. From a design perspective, alternatives that avoid slope stabilization measures (i.e., rock columns) are preferred given the degree of difficulty associated with constructing these works and the possibility of post-construction slope movements as the strength of the rock fill is mobilized. Discussions with the City of Winnipeg Water and Waste Department also suggest they would prefer to maintain the existing 1v:9h slopes if possible and from the perspective of hydraulic efficiency, prefer the original alignment of the Branch I Aqueduct. However, when the two alternatives are compared, there may be a significant cost advantage associated with Case 5 once the cost of raising the railway grade crossing at PTH 101 are taken into account for Case 4. Scheduling of this work around the City of Winnipeg's work for the new Water Treatment Plant at Deacon Reservoir and other planned work is also more favorable for Case 5.

Recommended alternatives for the Branch II Aqueduct are based on Alternative 3a from the Iteration 2 process, that being widening of the channel base by 40 m, approximately half on each side of the channel, and constructing 1v:9h side slopes. This channel section would be maintained approximately half way to the Branch 1 crossing, at which point the channel would transition to the 40 m east side only widening at Branch 1 and GWWD Bridge. Pre-design drawings for the selected alternative are shown on the attached Drawings 1 (Branch I crossing) and 2 (Branch II crossing).

Both re-alignments would likely involve a construction program commencing from the centre of the Floodway working outwards towards the existing bends at the limits of the 1966 relocations. The pipe would be temporarily terminated a sufficient distance from the existing horizontal bends beyond the crest of the Floodway that would preclude movement of the existing thrust blocks. After successful pressure testing, the new pipe segment would be connected to the existing pipes by removing the existing horizontal bends and installing PCP closure sections.

The timing of the Branch I and II Aqueduct reconnection operations are sensitive to both prevailing water demand and to weather. In this regard, works should not be scheduled during the winter months.

Construction during winter months poses many concerns relating to the ability to effectively compact backfill material around the Aqueduct Branches, working with frozen material, workmanship and the risk associated with unanticipated occurrences. Reconnection works cannot be scheduled during periods of high water demand (i.e., summer demands) as working on the Regional Water Supply system under high demand conditions presents an unreasonable risk in terms of assuring a reliable supply of water for the duration of reconnection operations.

The month of October provides the best balance between lower average daily demands and favourable prevailing weather conditions for reconnection operations. Average daily water demand in October for the past 6 years is on the order of 225 megalitres per day (MLD). Maximum weekly demands (97.5% probability that the value will not be exceeded) are approximately 245 MLD.

To provide sufficient water supply to the City of Winnipeg during reconnection operations the Branch I and II Aqueducts cannot be taken out of service at the same time. Further, Aqueduct relocation schedules must also be integrated with the construction of the proposed Water Treatment Plant and other City of Winnipeg Regional Water Supply upgrading programs. Based on Preliminary discussions with the City of Winnipeg, it appears feasible that both Aqueduct reconnections could be accommodated in 2006, subject to a review on all City Regional Infrastructure projects. In order to facilitate 2006 reconstruction, detailed design and Aqueduct shutdown schedule will need to be finalized in 2005.

4.8 SEINE RIVER SYPHON AND OVERFLOW STRUCTURES

The current Seine River Inverted Syphon consists of two components. An inverted syphon that carries **riparian** flow under the Existing Floodway; and four overflow outlet pipes that release excess water into the Floodway during Seine River high flood conditions.

The intake for the overflow and the syphon are contained in one reinforced concrete intake structure located south of Hallama Drive, and the East Embankment.

The syphon portion of the intake consists of a drop structure with energy dissipation blocks that connects to a 1.5 metres (5 ft) diameter corrugated metal pipe. The pipe carries riparian flow under the Floodway and releases it through a concrete outlet structure into the Seine River on the northwest side of the Floodway. There is a concrete inspection manhole in the pipe located approximately at the low flow channel. This pipe has a capacity of approximately 4 m^3/s (130 cfs) under normal conditions, and 10 m^3/s (350 cfs) under extreme flood conditions.

The overflow portion of the intake consists of an energy dissipation basin that connects to four 2.4 metres (8 ft) diameter corrugated metal pipes. The pipes are approximately 90 metres (290 ft) long and extend under the East Embankment of the Floodway and release directly into the low flow channel. Under extreme conditions, it is estimated that a flow of 42 cms (1500 cfs) could enter the Floodway through these pipes.

In 1999, the Seine River Inverted Syphon under went a rehabilitation to maintain proper operation of the syphon pipe. The main component of the rehabilitation involved the installation of a 1.2 metres (4 ft) diameter plastic pipe inside of the original metal pipe.

4.8.1 Pre-Design

Following the Iteration 3 report, the Floodway Channel optimization resulted in less widening than was determined in Iteration 1 and 2. It was found that only minimal changes from the original Floodway Channel cross-section were necessary at the Seine River Inverted Syphon. This was due to refinements in the pre-design of the CPR bridge located just upstream of the syphon and the PTH 59 bridges located just downstream of the syphon.

As a result, all of the original syphon components will remain in place. As with the previous planned new structure, gates will be required to prevent water from the Floodway from flowing back up the overflow pipes into Grande Pointe. Therefore, a backflow control structure will be required.

From a review of the hydraulics of the Seine River and Floodway Channel and incorporating the newly constructed Grande Pointe flood protection structures, it was determined that two of the four existing overflow pipes could be abandoned. It was decided to incorporate abandoning these pipes as part of the backflow control structure.

The backflow structure will be constructed at a suitable location in the East Embankment. The two existing pipes to be used for the overflow will be intercepted and cast into the new concrete structure that would contain the gates. The two abandoned pipes will be intercepted at the same location and a concrete plug installed.

To meet the design criteria regarding the historical problems associated with debris blocking the syphon inlet trashrack, modifications to the inlet structure are planned. It is planned to construct a new trashrack system upstream of the syphon and overflow inlets. This will assure that flows will enter the syphon and not the overflow unless the capacity of the syphon is exceeded.

4.8.2 Construction

The work will be completed during a four to six month period during the summer. As the planned modifications do not require modification of the syphon pipe, riparian flows during the summer will not be affected during construction.

The sequencing of the construction is not dependent on other works, therefore the year in which the work is performed is not critical.

4.9 LOCAL DRAINAGE INLETS

Drawing DWG 004c (19) in Appendix 4.0 (KGS/Acres/UMA 2004) provide a useful overview of the surface drainage infrastructure along the Floodway Channel.

Drainage infrastructure, in the form of Local Drainage Inlets, are located on both sides of the Existing Floodway. These inlets convey discharge from local land drains or river diversions into the Existing Floodway Channel. The single exception is the Seine River Syphon, which conveys flow underneath the Floodway Channel. Due to the proposed deepening and widening of the Floodway Channel, the drainage inlets are affected and will require modification to accommodate the channel expansion, or if justified, complete replacement.

Drainage inlet structures requiring modification include:

- Replacement / Rehabilitation of drainage structures that discharge local runoff into the Floodway, including enhancement of the discharge capacity to comply with a 1 in 100 year design inflow where practical
- Centreline Drain – replacement
- North Bibeau Drain – replacement
- Cook's Creek Diversion – repair (retains 1 in 50 year capacity)
- Springfield Road Drain – replacement
- Shkolny Drain – replacement
- Ashfield Drain – replacement
- Transcona Storm Sewer Outlet

The Kildare Trunk and Country Villa Estates drop structures are both located on the west side of the Floodway Channel and convey surface runoff from urban developments. Consequently, the designs for these structures are based upon requirements for urban drainage and are different that agricultural drains. The stormwater within the catchment of the Kildare Trunk Sewer is managed using modern runoff management principles where new development must be designed with no increase in the rate of runoff from the development. This contemporary design should preclude the requirement for capacity upgrades except for the infrastructure required to protect the trunk system and provide some level of drainage during periods of high levels in the Existing Floodway Channel.

The Country Villa Estates drop structure was designed and constructed in 1998 and is expected to meet contemporary standards and not require a capacity upgrade.

4.9.1 Agriculture or Rural Drainage Drop Structures

Rural drainage drop structures convey flow from agricultural land and had a common basis of design during the original Floodway construction. They were examined together to ensure consistency in the design of structure replacement or modification required due to the Floodway expansion. These structures include:

- Centreline Drain Drop Structure
- North Bibeau Drain Drop Structure
- Cooks Creek Diversion Drop Structure
- Springfield Road Drain Drop Structure
- Shkolny Drain Drop Structure

- Ashfield Drain Drop Structure

Structural inspections of the structures were undertaken as part of the design process and are included in the Appendix D of the Preliminary Engineering Report (KGS/Acres/UMA 2004).

4.9.1.1 Development of Design Criteria

The basic design criterion for the drainage structures is to provide an appropriate level of service for the reasonable future and to be consistent with past design practices. The document “Red River Floodway – Notes on Drainage Design”, A. Kulak, 1963, identifies that the discharge frequency criteria proposed for the original Floodway included: Drains – 50 years; Culverts – 75 years or 1.20 times the drain discharge and Drop Structures – 100 years or 1.36 times the drain discharge.

This was confirmed by a comparison of the capacity of the existing structures with design flows computed using the 1963 Runoff Formula.

The results indicated that all of the drainage structures, except the Ashfield Drain, have an existing capacity that is greater than the 1% flow computed using the 1963 Runoff Formula. The hydraulic capacity of the drop structures were then compared to the design flows computed using contemporary hydrologic runoff formula. This analysis indicates that only the Springfield Road drop structure would meet the original intent of providing hydraulic capacity for the 100-year runoff event. All of the other structures have a hydraulic capacity less than the 1:33 year event except the Cooks Creek Diversion drop structure has the capacity to convey the 50-year return period flow.

The assumptions used in the design of the rural drainage structures were developed over the course of the three design iterations. Summaries of the design assumptions used are presented in the Preliminary Engineering Design - Appendix D (KGS/Acres/UMA 2004a).

The assumptions used for the preliminary design included:

- The target design life for all structures was to be in excess of 50 years
- Reconstruction of all the rural drainage drop structures except the Cooks Creek Diversion structure, which will be repaired
- Using the 1% flow event as the required hydraulic capacity for all rural drainage outlets/drop structures to be reconstructed
- Lowering the invert elevation of the drop structure inlets by approximately 0.6m (2ft) during reconstruction to allow the approach drains to be graded to a lower invert
- Using the 2% flow event as the required hydraulic capacity for all primary drains to be reconstructed upstream of the drop structures. The drain improvements would be restricted to the Floodway right-of-way and would include gradient control/ energy dissipation structures at the right-of-way boundary
- Use a 0.03% longitudinal slope for the reconstructed drains whenever possible

- Constructing grade transition sections on the primary drains at the boundary of the Floodway right-of-way.

Cost estimates were prepared for replacement of the structures using the assumptions given above and unit prices based on recent industry estimates.

The lowering of the invert of the replacement inlet structures and approach channels would enable future municipal drainage improvements. Preliminary design drawings of the proposed structures and channel alignments are shown in the Preliminary Engineering Report (KGS/Acres/UMA 2004c), Appendix D. The cost of the replacement drop structures, approach channel excavation, culverts, energy dissipation weirs and channel erosion protection were estimated based on design quantities and unit prices based either on suppliers quotes or previous local experience. More detailed rationale for the design considerations and cost estimates is presented in the Preliminary Engineering Report, Appendix D.

The preliminary designs presented are for replacement of the existing rural drain drop structures with new structures designed to accommodate the 1 in 100 year flow event and a service life in excess of 50 years. The cost estimate assumed a relatively inexpensive repair of Cooks Creek Diversion to restore its functionality although its life expectancy will be less than the other structures.

The preliminary design cost estimate includes channel improvements within Floodway right-of-way for the major agricultural drains, but not including the spoil embankment toe drains unless coincident with the agricultural drain.

The reconstruction of the drainage drop structures must be scheduled to coincide with the adjacent Floodway Channel earthworks in order to minimize the cost of mobilization and depth of excavation required for the installation of the drop structure pipes, outlet structure and outlet drain. The new drop structures will be constructed during the summer and the existing structures removed once the new ones are in place.

4.9.2 Country Villa Estates Drain Outlet Structure

Country Villa Estates Drain was recently designed (February, 1998) by Stanley Consulting Group Ltd. (presently Stantec). The drain services an urban subdivision and has a ditch with beehive grate and catch basin with a 600 mm concrete pipe running beneath the Floodway embankment and discharging into the Low Flow Channel.

The catch basin drains two ditches located upstream of and adjacent to the Floodway embankment. A flap gate and a positive gate control located in manhole chambers on the top of the spoil embankment to protect the drain from back-up of Floodway flows.

The design criteria proposed for the drainage structures is to provide an appropriate level of service for the foreseeable future and to be consistent with past design practices. Given the recent design and construction of the Country Villa Estates Drain, the design capacity was accepted without further analysis at this time.

The Country Villa Estates outlet can be successfully modified given the "as new condition" and hydraulic capacity.

The proposed channel cross-section requires relocation of the outlet approximately 40 metres west of existing location. Removing pipe sections and moving the outfall further from the low flow channel can accommodate this.

4.9.3 Transcona Storm Sewer Outlet

The Transcona Storm Sewer Outlet (also referred to as the Kildare Avenue Outfall) is a 2895 mm (9.5 ft) diameter concrete tunnel sewer, outfall and gate chamber servicing an area of nearly 20 km^2 and a population of roughly 30,000. Considerable modifications are required due to the proposed excavation of the Floodway Channel including construction of a new gate chamber; and replacement of the pump discharge conduits; the downstream section of pipe; the energy dissipater and headwall structure.

The replacement gate chamber was designed to provide flood protection to 1m (3.3 ft) above the design water. The design included a sluice gate installed downstream of a flap gate, and provision for temporary pumping capability to allow the installation of permanent dewatering pumps units by the City of Winnipeg in the future.

Approximately 15 m (50 ft) of new 3000 mm (10 ft) diameter heavy gage Structural Plate Corrugated Steel Pipe (including two 2.0 m (6.6 ft) long slip joints) will be installed by open cutting upstream of the headwall structure. Permanent pumping units will not be installed as part of the Floodway Expansion Project, but the gate chamber/pump station structure was sized to house and support the required equipment in the future.

The flood pump station was designed with consideration of design flows, pump capacities, and station location and it was recommended that the total capacity be approximately 1.10 m^3/s (39 cfs). This is 10% greater than the large pump at the Deep Pond pump station further upstream and should consist of two 0.55 m^3/s (19 cfs) vertical mixed-flow pumps. The capacity of each pump will match the capacity of one of the small pumps at the Deep Pond pump station in order to avoid excessive cycling when only one pump is operating upstream.

In addition, the new gate chamber/pump station should be able to accommodate two 2250 Flygt Bibo submersible pumps until the City installs permanent units. The station has been configured as shown in drawings in Appendix D. Utilities in the vicinity of the proposed gate chamber are shown in Appendix D. A slope stability analysis was conducted and indicates that the minimum acceptable factor of safety of 1.5 occurs about 109 m (360 ft) west of the proposed toe of slope of the expanded Floodway Channel.

Modifications to the outfall, including installation of a new gate chamber should ideally take place between mid November to mid March when stormwater runoff is at a minimum. Even in this period, it will be necessary to maintain some outflow capacity using temporary by-pass piping or pumping due to the possibility of water entering the system due to watermain breaks or other unforeseen emergencies.

4.9.4 Deacon Drain Chamber and Aqueduct Underdrain Outfalls

The Deacon Drain Chamber is a 1.5m diameter outfall line constructed in 1997 as part of the Shoal Lake Aqueduct Rehabilitation Program. Two 200 mm diameter Aqueduct underdrain outfalls were installed shortly after the construction of the Shoal Lake Aqueduct to lower groundwater levels adjacent to the concrete aqueduct pipe, which were constructed of concrete susceptible to severe sulphate attack. These lines will require modification with the Project (Preliminary Engineering Report: Appendix D Outlet Drainage Structures and Syphons Pre-Design).

4.9.4.1 Deacon Drain Chamber

The 1500 mm diameter Deacon Drain Chamber outfall line, and the two 200 mm diameter Aqueduct underdrain outfalls, have been re-designed to accommodate the proposed Floodway Expansion. The locations of the outlet lines are approximately at Floodway Station 22+000 metres (see Drawings at the back of this Chapter).

The geometry used is based on the assumption that the Existing Floodway in the vicinity of the Branch I Aqueduct and GWWD Railway bridge will not be deepened, and widened approximately 40 metres to the east only. Side slopes will be constructed at a slope of 1v:9h, to provide the minimum required factors of safety for the Aqueduct crossings.

The Deacon Drain Chamber outfall was constructed in 1997, as part of the Shoal Lake Aqueduct Rehabilitation Program. As a drain line, it has numerous intended functions, including:

- to provide an emergency outlet for the Shoal Lake Main Aqueduct at the full flow Main Aqueduct discharge rate of 385 million litres per day (MLD);
- to provide a convenient low level drain for the of the Aqueduct and Deacon Reservoir complex;
- to provide an emergency outlet line for the Branch II Aqueduct Surge Tower in the Deacon compound;
- to provide site surface drainage and miscellaneous sump pits for the Deacon Booster Pumping Station compound.

Future uses of the line also include functioning as an emergency overflow for the clearwell of the proposed Water Treatment Plant (WTP) and for the Branch I Aqueduct Surge Tower. The Branch I Surge Tower is currently proposed to be in service by 2005/2006 while the WTP is scheduled to be in service by 2007. The Deacon Drain Chamber outlet is considered a key component of infrastructure that provides considerable operational flexibility and emergency protection for the Aqueduct and the Deacon Booster Pumping Station and Reservoir facility.

<u>Design</u>

Based on the proposed Floodway configuration, modification of the outfall line should be fairly straightforward. The existing cast-in-place outfall structure will require removal. Approximately 41 metres of the existing drain line will also require removal. A new cast-in-place outfall structure, similar to the current structure, will need to be constructed.

The new structure will be constructed such that it does not protrude into the flow channel. Stone riprap will be installed to an extent that normal outflow from the drain chamber outfall line will be adequately dissipated prior to overland flow to the low flow channel, and such that erosion will not occur at the base of the proposed embankments. Significant outflows from the outfall line are infrequent, and as such it is estimated that the cost of riprap protection of the outfall line to the low flow channel is in excess of the cost of repair of occasional erosion of the Floodway Channel.

Construction

Coordination with the Branch Aqueduct relocation and Floodway widening will require careful coordination.

4.9.4.2 Aqueduct Underdrain Outfalls

The Branch I Aqueduct is paralleled along its length by a clay tile underdrain system. The Aqueduct construction predates the advent of sulphate resistant cement. Shortly after construction of the Shoal Lake Aqueduct, portions of the Aqueduct between Mile 12.87 (Deacon) and 26.32 (PTH 12) were found to have severe sulphate attack. After this discovery, the Branch I portion was designed with an underdrain system, in order to lower ground water levels adjacent to the pipe, and portions of the main Aqueduct were retrofitted with underdrains to reduce the effects of sulphate-related degradation. The system has proven to be very effective as recent condition assessment programs has shown very little progressive sulphate degradation since the inception of the underdrain system.

During construction of the Existing Floodway, the underdrain system was effectively separated into two systems, east and west of the Floodway. Two independent outfalls were constructed, consisting of 200 mm clay tile pipe, with a corrugated steel pipe outlet section. The east outfall was abandoned with the construction of the Deacon Drain outfall in 1997, and the underdrain system rerouted to the newly constructed outfall structure. The west outfall remains in place.

Design

For purposes of the Floodway Expansion, it is expected that the east underdrain system will be terminated near the point of the proposed relocated Branch I Aqueduct, near the proposed crest of the Floodway embankment and redirected to the Deacon Drain outfall line. There is no need to provide an underdrain system for the proposed Aqueduct relocations, as the modern concrete pressure pipe is constructed of sulphate resistant materials. There is an existing drain line manhole beyond the crest of the proposed Floodway, which would serve as a convenient connection point. The 1500 mm drain line is significantly deeper than the underdrain system at this point. Construction would entail modification of an existing underdrain manhole or installation of a new manhole to such a depth as to be able to tunnel a new underdrain outlet under the existing Aqueduct. Approximately 13 metres of 200 mm pipe, consistent with the current underdrain size will be required. A backflow device will be installed in this line to protect the system from Floodway backup. There are a minimum of three existing underdrain manholes between the Floodway and Deacon, with rim elevations of 0.1 metres to 0.45 metres below the proposed 700 year flood level. These manholes should be adjusted to provide protection to the 700 year level, plus a reasonable freeboard allowance.

With the Iteration 2 Floodway geometry, the west underdrain outfall could potentially be salvaged, as the west Floodway embankment will remain relatively untouched. However, it is constructed of clay tile pipe, and has not been assessed for structural defects at this time. It will also require some modification at the proposed Branch I Aqueduct reconnection point. Due to these factors, it is recommended that the line be replaced to avoid disruptive repair work at a later date. This matter remains under review.

The west outfall line replacement would involve installation of a new underdrain manhole at the proposed Branch I Aqueduct connection point near the crest of the Existing Floodway, and extendion to at least the toe of the proposed 1v:9h Floodway slope, a distance of approximately 140 m. This is an increased length over the Iteration 1 and 2 geometries, due to the “east-only” widening of the Floodway. It is recommended that this pipe be installed perpendicular to the Floodway channel to economize on pipe length. There is an existing legal parcel (Parcel 12, Plan 10204) currently in place for the existing outfall line. This parcel would be convenient to use for this purpose. A short outlet pipe section is recommended at the exit point from the bank, to prevent damage of the outlet, as well as a minimal amount of stone riprap to dissipate energy prior to flowing overland to the low flow channel.

4.9.4.3 Construction

The construction activities for the underdrain outfalls would be coordinated with the earthworks for the expansion. The required works should not significantly affect the construction schedule.

During the course of the PDEA, there were many suggestions provided by the Rural Municipalities for increasing the number of drainage structures. This was considered beyond the scope of the Floodway Expansion. No additional outlets have been included in the Floodway Expansion Project.

4.10 UTILITY CROSSINGS

Details pertaining to Utility Crossings are derived from the most current design information as of June 2004. This information is contained in the document “Preliminary Engineering Report: Appendix E Utilities Crossings Pre-Design” (Stantec/Teshmont – 2004).

A number of utilities cross the Floodway Channel Corridor above and below the ground surface, and via utility conduits contained in bridge crossings. In addition, several utility lines pass through existing Red River Floodway rights-of-way in parallel to the existing channel. Utilities crossing and or running adjacent to the Existing Floodway Channel include:

- Manitoba Hydro Transmission Lines;
- Manitoba Hydro Distribution Lines;
- Imperial Oil - oil supply lines;
- Manitoba Hydro Natural Gas Lines;
- Manitoba Telecomm Services telecommunications lines;
- Manitoba Hydro fibreoptik lines;
- Municipal Utilities – water distribution mains and wastewater sewer conduits.

Drawings DWG 004c (19) in Appendix 4 provide useful perspective on the Floodway Channel and utility crossings.

Pre-design concepts for relocation/replacement/protection options were developed for all of the affected utilities. The basic principles followed in the development and evaluation of the preliminary design concepts were to maintain or return the same level of service and safety that existed prior to the expanded Floodway Channel.

For buried utilities, the minimum and maximum depths of cover as specified by the utilitiy owners would be maintained or provided for in the Expanded Floodway Channel as shown in Table 4.15-1.

Table 4.10-1
Burial Depth Summary for Direct Buried Oil Pipelines, Natural Gas Pipelines and Communication Cables

Utility	Minimum Cover	Maximum Cover
Imperial Oil	2 metres	Dependent on site conditions and the design engineer's discretion
Manitoba Hydro Natural Gas	1.5 metres	Any existing natural gas pipeline that will have more than 2.4 metres of cover after placement of embankment shall be relocated
MTS		
• Copper Cable	1.0 to 1.5 metres	To be confirmed by MTS
• Fibre Optic Cable	2.0 to 3.0 metres	To be confirmed by MTS
Manitoba Hydro Communications Department	1.5 metres	3.0 metres

For overhead utilities, the Manitoba Hydro Maintenance Department stated that a minimum access distance of 8 metres perpendicular and 10 metres parallel to the Expanded Floodway Channel top of side-slope would be required to adequately maintain transmission/distribution towers as specified in Table 4.15-2.

Table 4.10-2
Summary of Overhead Utilities Transmission & Distribution Requirements(Preliminary Engineering Report Appendix E: Utilities Crossings Pre-Design)

Utility	Perpendicular Access Distance	Parallel Access Distance
Manitoba Hydro Transmission and Distribution	8 m	10 m

For overhead utilities, no embankment placement is allowed to be placed under transmission or distribution lines without the prior approval of Manitoba Hydro.

4.10.1 Manitoba Hydro Transmission Lines

Fourteen existing and future transmission lines crossing the expanded Floodway and six existing and future transmission lines parallel to the Floodway have been identified. Consultants assigned to Utility Crossings have worked closely with Manitoba Hydro to minimize the number of towers potentially affected by the channel modifications.

Presently, with the expanded Floodway Channel pre-design, all affected towers are proposed to be relocated or protected with slope stabilization (Preliminary Engineering Report Appendix E: Utilities Crossings Pre-Design).

<u>Affected Transmission Lines</u>

The following transmission lines have been identified as being potentially affected by the expanded Floodway Channel:

- MH-4X*, R33V/R49R crossing south of PTH 59 (Bird's Hill) 230 kV double circuit steel lattice towers.
- MH-8X, GT1/ST2 crossing at East Transcona 115 kV double circuit steel lattice towers.
- MH-9X, WT34 crossing at East Transcona 115 kV double circuit steel lattice towers.
- MH-10X, ST5/ST6 crossing at East Transcona 115 kV double circuit steel lattice towers.
- MH-16X, VJ50/VT63 crossing at PTH 59 South double circuit 115 kV line on steel lattice towers.
- MH-18X, Future 500 kV ac lines (South Corridor).
- MH-19X, Future 500 kV ac lines (South Corridor).
- MH-20X, Future 230 kV ac line (South Corridor).
- MH-5P, Future 500 kV ac lines (South Corridor).
- MH-6P, Future 500 kV ac lines (South Corridor).
- MH-7P, Future 230 kV ac line (South Corridor).

*Note – Numbers refer to Manitoba Hydro labeling protocols.

For each transmission tower, a number of pre-design options were considered, including:

- Relocation and/or replacement to an assumed safe setback distance of 10 metres from the Expanded Floodway Channel top of side-slope.
- Protection with a berm and/or slope stabilization.
- Protection with a retaining wall and/or slope stabilization.
- Protection with slope stabilization.

Some crossings with special considerations are discussed below:

<u>MH-4X Crossing</u>

The MH-4X west tower handles significant power allocated to customers for export by Manitoba Hydro. Studies conducted by Manitoba Hydro indicate estimated costs associated with outage of the MH-4X 230 kV line can range between 0 and $440,000 per day. For the purposes of estimating relocation costs, an allowance of $250,000 per day was assigned for outages for this line.

The MH-4X west tower requires relocation and replacement. The proposed relocation and replacement option includes:

- Relocating and replacing the west tower #4 to a location west of the MH-2X line.
- Installation of a concrete caisson foundation.
- Installation of rider poles on each side of the MH-2X line.
- Obtaining an outage.
- Erecting a double circuit single tubular steel pole.
- Disconnecting the conductors from existing tower #4.
- Connecting the conductors to the new steel pole.
- Disconnecting and salvaging the existing tower #4.

<u>MH-7X West Tower and MH-15X West Tower</u>

The MH-7X and MH-15X west towers will not be affected by Floodway Channel expansion due to recent decisions by Channel Pre-Design consultants to employ minor realignment of the expanded channel in the vicinity of these two towers. Due to the channel realignment, berms and stabilization will not be required for these two towers.

<u>MH-8X, MH-9X, MH-10X and MH-16X 115 kV Towers</u>

These towers will be relocated. Previous designs called for consideration of protection of these towers with retaining walls, however relocation costs were found to be comparable to retaining wall costs, therefore relocation was selected as the preferred option.

MH-16X and MH-17X Towers

For the MH-16X and MH-17X towers, previous design iterations considered modifying the channel alignment, with minor changes to the upper slope and inclusion of slope stabilization with rock caissons, thereby removing the need for berm protection. In the detailed design phase, further investigations will be carried out to assess if tower relocation would be more cost effective.

Future Crossings

Costs associated with the impact on future 500 kV ac lines crossing the Floodway, MH-18X, MH-19X and MH-20X and the future 500 kV and 230 kV lines parallel to the Floodway (south corridor), MH-5P, MH-6P and MH-7P have not been assessed as they are not quantifiable at this time. Manitoba Hydro has stated that one line parallel to the Red River Floodway (South Corridor) is planned within the next ten years.

Scheduling of Proposed Works

For the proposed works that require tower relocation, the construction duration will be relatively short. Relocation and swtichover of the lines can be completed within 2-3 days. However, the planning, ordering of materials, installation of foundations will require a longer lead time, with a duration of nine months to a year. The longer the lead time, the higher the possibility of timing the outages to minimize costs,

4.10.2 Manitoba Hydro Distribution Lines

Four existing distribution lines (66 kV and below) crossing the Floodway and three existing distribution lines parallel to the Floodway have been identified. Close coordination with Manitoba Hydro Distribution Department has taken place to minimize the number of towers potentially affected by the channel modifications.

Affected Distribution Lines

The following distribution lines have been identified as being potentially affected by the Expanded Floodway Channel:

- MH-1X, RY-250 crossings at Garven Road, two 12 kV wood pole structure lines.
- MH-11X, Line 31, 66 kV wood pole crossing at St. Boniface Road.
- MH-17X, Lines 10 and 14, crossing at PTH 59 South double circuit 66 kV in steel lattice towers.
- MH-8P, 12.5 kV lines parallel to the Floodway channel north of PTH 59 north, along PR202.
- MH-9P, 12.5 kV lines parallel to the Floodway channel along Chrypko Road, perpendicular to the St. Mary's Bridge to the north.
- MH-10P, 12.5 distribution line parallel to the Floodway channel at the end of Navin Road.

MH-1X, MH-11X, MH-8P. MH-9P and MH-10P Towers

For the MH-1X, MH-11X, MH-8P. MH-9P and MH-10P towers, relocation of the structures will be required. The MH-1X and MH-11X existing wood pole structures adjacent to the expanded Floodway channel top of side-slope will be replaced with steel pole structures due to expanded span length.

Drawings providing detail to each Hydro utility crossing or parallel to the Floodway Channel Expansion are available in Annex D of "Preliminary Engineering Report: Appendix E Utilities Crossings Pre-Design".

4.10.3 Oil Utilities

Two parallel oil lines owned by Imperial Oil were identified. Imperial Oil has two oil lines crossing the Red River Floodway at station 13+800. These lines are 200mm and 250mm in diameter. The oil lines were originally installed with minimum cover, consequently any channel excavation will result in replacement of these lines.

Imperial Oil has expressed a preference to install new lines by trenchless methods across the width of the expanded Floodway channel excavation (outside toe to outside toe of embankments). The old lines will be abandoned in place and removed, if necessary, during channel excavation. No oil line valves will be located within the expanded Floodway channel.

Imperial Oil has stated that minimum pipeline cover is 2 metres and the maximum cover will depend on the site conditions and the design engineer's discretion. Provision for gaps in the Floodway Embankments may be made to prevent excessive overburden on the pipes and for ease of future maintenance.

The new lines will be installed prior to channel excavation and the existing lines can be removed during the channel excavation. Imperial Oil requires approximately 6 months lead-time to complete the detailed design and construction. An estimated 30 days will be required to complete the new oil line installations. The preferred time of year for construction is late summer or fall, however the work could be completed at any time of the year, provided the Floodway is not in operation. The location of the oil lines probably dictates that they be relocated within the first year of the Floodway Expansion construction.

Drawings providing detail of the new configuration for the oil lines are available in Annex D of "Preliminary Engineering Report: Appendix E Utilities Crossings Pre-Design" (Stanted/Teshmont 2004).

4.10.4 Manitoba Hydro Natural Gas

Manitoba Hydro's Natural Gas division has several natural gas lines crossing and adjacent to the Red River Floodway. The affected pipelines are as follows:

- ID1 300mm diameter crossing at CEMR Pine Falls (PTH 44).
- ID3 100mm diameter crossing at CNR Redditt Bridge (PTH 15).
- ID4 400mm diameter crossing at Dawson Road.
- ID5 100mm diameter crossing at PTH 59 south.

- ID7 300mm diameter parallel line on the west embankment at PTH 59 North.
- ID8 100mm crossing at Lockport.
- ID9 300mm diameter parallel line on the west embankment.
- ID10 300mm diameter line crossing temporary track at CNR Redditt Bridge.
- ID11 300mm diameter line crossing temporary track at CNR Sprague Bridge.

New lines will be installed by trenchless methods across the width of the expanded Floodway channel excavation. Lines running parallel to the Floodway will be relocated outside of the channel excavation area. The old lines will be abandoned in place and removed, if necessary, during channel excavation. The 300 mm ID9 line on the west embankment is not affected by excavation but will potentially have excessive cover from the finished embankment. This section of pipe will be reviewed by Manitoba Hydro Natural Gas during the detailed design phase of the project. Temporary railway tracks at the CNR Redditt and CNR Sprague Bridges will affect two sections of pipe. CNR has minimum pipe thickness and cover requirements for railway crossings and new lines will have to be installed beneath the temporary tracks.

The new lines will be installed prior to channel excavation and the existing lines can be removed during the channel excavation. Manitoba Hydro requires approximately 12 month lead time to complete their planning and design for the natural gas line crossings. Estimates indicate approximately 12 weeks will be required to complete the new pipe installations. Work would be completed during the summer and fall.

Drawings providing detail to each Hydro's Natural Gas lines crossing or parallel to the Floodway Channel Expansion are available in Annex D of "Preliminary Engineering Report: Appendix E Utilities Crossings Pre-Design".

4.10.5 MTS Telecommunications

A number of MTS cables cross and run parallel to the existing Red River Floodway. Some of these telecommunications lines run beneath the Floodway Channel, while others run through conduits within bridge crossings. A total of 12 MTS cable crossings have been indentified as being potentially affected by the Floodway expansion. Of these 12, five are on bridges, five are direct buried, and two run parallel to the Floodway channel.

The following direct buried crossings will require replacement:

- ID2 - Two fibre optic buried lines at McGregor Farm Road crossings.
- ID3 – Buried cables crossing at Springfield Road.
- ID4 – Buried cable crossing at Dawson Road.
- ID6 – Buried cables crossing south of CPR Emerson Bridge.
- ID7 – Buried cables crossing at St. Anne's Road.

The following MTS cables on bridges will also require replacement:

- ID1 – Cables crossing at PTH 59 North Bridge.

- ID5 – Cables crossing at PTH 59 South Bridge.
- ID8 – Cables crossing at St. Mary's Road Bridge.
- ID11 – PTH 15 Dugald Road Bridge.
- ID12 – PTH 44 (Lockport) Bridge.

Two MTS cables running parallel to the Floodway channel have been identified. There are no current plans to add additional embankment above these existing cables. Consequently, no impact is expected for the following cables:

- ID9 – Buried fibre optic lines parallel to Bird's Hill west and east banks.
- ID10 – Buried fibre optic lines parallel to Floodway near McGregor Farm Road.

For buried cable crossings, the proposed Floodway channel expansion involves a channel widening that conflicts will the existing cables, therefore all MTS cables crossing the Floodway will require replacement prior to Floodway expansion.

For cables within bridge crossings, a number of strategies will be employed:

- Bridges crossings at PTH 44 and PTH 59N will be raised and refurbished. During the construction, a temporary wood pole support line will be used. Upon completion of the refurbished bridges, the cables will be returned to the refurbished bridges.
- At PTH 50 S, the existing southbound bridge is being replaced. During construction, the cables will be temporarily re-routed over the northbound bridge before being returned to the rebuilt southbound bridge.
- At PTH 15 (Dugald Road), the existing crossing will be twinned. The new bridge will be built and the cables will be transferred to the new bridge. The old bridge will then be replaced.
- At PTH 200 (St. Mary's Road) Bridge, the bridge will be replaced and MTS cables will be direct buried for this location.

4.10.6 Manitoba Hydro Telecommunications

The Manitoba Hydro Communications Department has confirmed that there are three communication cable crossings and two communication lines parallel to the Red River Floodway. One fibre optic line, the –21X St. Vital to Richer line was installed in 2003 with provision for Floodway expansion as the cable was installed at a depth of approximately 15 metres below the Floodway Channel. Consequently, this line is unaffected by the proposed Floodway expansion.

The following communication lines will have to be replaced prior to Floodway expansion:

- MH-3X – two fibre optic communication lines crossing south of PTH 59 North.
- MH-14X – fibre optic communication cable at Riel Corner.

The MH-3X and MH-14X cables will be replaced, and would likely be updated to the current design standard. New lines will be installed by trenchless methods where the Floodway Channel is to be widened. A concern regarding heavy equipment used in excavation was raised. If the soil is soft, such equipment can cause ruts 1.5 metres deep, which matches the typical burial depth for communications cables. Once excavation begins, it was suggested that the communications cables would have to be identified and entrances to the Floodway excavation would need to be established.

4.10.7 Municipal Utilities

Neighbouring municipalities of Ritchot, Springfield, East St. Paul and St. Clements were contacted to confirm the location of existing or planned utilities crossing or adjacent to the Red River Floodway. All contacted municipalities responded and one crossing location was identified. The affected utility is a crossing consisting of 2-250mm diameter water distribution lines belonging to the RM of East St. Paul which crosses the Floodway at station 34+290. The waterlines were installed with minimum cover and any channel excavation will require replacement of these two lines. The new lines will be installed by trenchless methods where the Floodway channel is to be widened. The old lines beneath the channel base will remain in service where cover permits. The portion of pipe affected by widening of the Floodway channel will be abandoned in place and removed during channel excavation. A gap will be let in the Floodway embankments to prevent excessive overburden on the pipes and for ease of future maintenance.

The new lines will be installed prior to channel excavation and the existing lines can be removed during the channel excavation. Estimates indicated that completion of this work would require approximately 30 days. The location of the watermains likely means that this work would be conducted in the third year of construction.

A complete summary of all utility crossings conceptual scheduling is provided in Table 4.10-3.

Table 4.10-3
Utilities Crossings – Overall Proposed Schedule

Utility Description	Estimated Year of Completion			
	1	2	3	4
Manitoba Hydro Communications, Distribution and Transmission				
Manitoba Hydro Utilities – Crossing Floodway				
MH-17X 66kV Lines 10 & 14, crossing at PTH 59 South	✓			
MH-16 115kV VJ5O/VT63 crossing at PTH 59 South	✓			
MH-14X Fibre optic communication cable at Riel Corner		✓		
MH-21X Fibre optic communication cable at Deacon Crossing		✓		
MH-11X 66kV Line 31, Crossing at St. Boniface Road		✓		
MH-10X 115kV ST5/ST6 crossing at East Transcona			✓	
MH-9X 115kV WT34/WT34 crossing at East Transcona			✓	
MH-8X 115kV GT1/ST2 crossing at East Transcona			✓	
MH-4X 230kV R33V/R49R crossing south of PTH 59N			✓	
MH-3X Two fibre optic communication cabbies crossing south of PTH 59N			✓	

Utility Description	Estimated Year of Completion			
	1	2	3	4
MH-1X 12kV RY250/RY256 crossings at Garven Road			✓	
Manitoba Hydro Utilities – Parallel to Floodway				
MH-9P 12.47 kV lines along Chrypko Road	✓			
MH-10P 12.47 kV lines at end of Navin Road		✓		
MH-8P 12.47 kV lines north of PTH 59 north, along PR202			✓	
MH-1P Two fibre optic cables crossing south of PTH59 North			✓	
Manitoba Hydro Gas Pipelines				
ID3 Crossing at CNR Redditt Bridge (PTH 15)	✓			
ID10 CNR Redditt Bridge	✓			
ID11 CNR Sprague Bridge	✓			
ID5 Crossing at PTH 59 South	✓			
ID9 West Embankment (15+220 17+070)		✓		
ID4 Crossing at Dawson Road		✓		
ID7 West Bridge Embankment at PTH 59 North (34+400 to 35+200)			✓	
ID1 Crossing at CNR Pine Falls (PTH 44)				✓
ID8 Lockport				✓
MT				
ID5 Cables crossing at PR200 St. Mary's Road Bridge (Parcel 2)	✓			
ID7 Buried cables crossing at St. Anne's Road	✓			
ID6 Buried cables crossing south of CPR Emerson Bridge	✓			
ID5 Cables crossing at PTH 59 South Bridge (Parcel 2)	✓			
ID11 Cables crossing at PTH 15 Dugald Road Bridge (Parcel 2)	✓			
ID12 Cables crossing at PTH 44 (Lockport) Bridge (Parcel 2)	✓			
ID1 Cables crossing at PTH 59 North Bridge (Parcel 2)	✓			
ID4 Buried cables crossing at Dawson Road		✓		
ID3 Buried cables crossing at Springfield Road			✓	
ID2 Two fibre optic buried lines at McGregor Farm Road			✓	
ID9 Buried fibre optic lines parallel to Birds Hill west & east bank (Parcel 3)			✓	
ID10 Buried fibre optics parallel to FW near McGregor Farm Rd. (Parcel 3)			✓	
Pipelines (Oil and Water)				
Oil Pipelines – Imperial Oil				
ID1 Oi – 250 mm Dia. Pipeline Crossing at PTH 59 North		✓		
ID2 Oi – 200 mm Dia. Pipeline Crossing at PTH 59 North		✓		
Water Pipelines				
ID1 Water – RM East St. Paul 250 mm water distribution line			✓	
ID2 Water – RM East St. Paul 250 mm water distribution line			✓	

4.11 WEST DYKE ENHANCEMENTS

Details pertaining to West Dyke Enhancements are derived from the most current design information as of June 2004. This information is contain in the document "Preliminary Engineering Report: Appendix F West Dyke Surveys, Field Investigations and Engineering" (Acres/UMA 2004).

The West Dyke is an integral part of the existing flood protection system for Winnipeg. On the west side of the Red River, the existing West Dyke extends approximately 60 km (44 miles) in length. The dyke is a compacted-earthen embankment with grassed sideslopes typically at 1v:5h.

The West Dyke was recently upgraded along portions of Highway PR 305 and a 11 km (7 miles) long reach running east of PR 330.

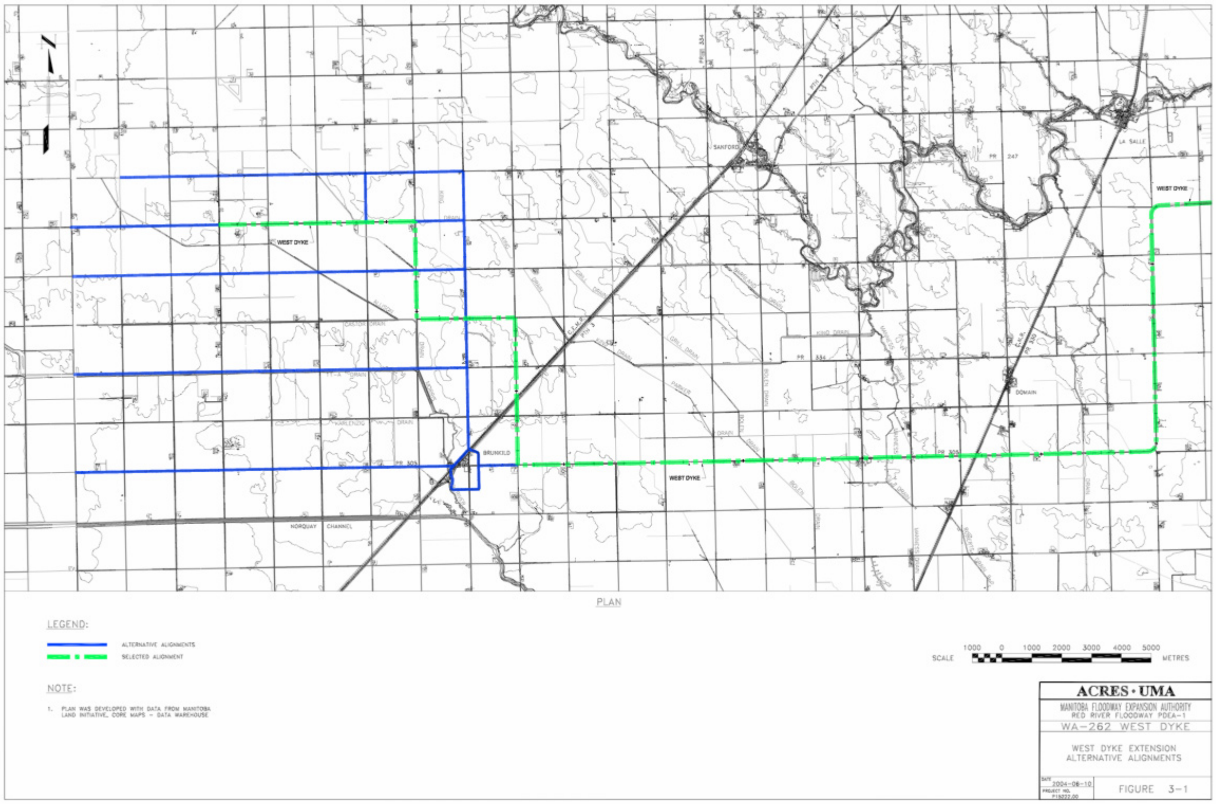
The current elevation of the Dyke is anticipated to be capable of providing protection against water levels of approximately 237.1 metres ASL (778 ft). The proposed Project will raise the height of the existing dyke to accommodate a greater allowance for freeboard (1.7 meters) due to wind effects, and extends the western limit of the West Dyke into higher ground.

4.11.1 West Dyke Alignment

A field program was conducted to assess the current state of existing structures and adjacent land uses, As the field program progressed, it became apparent that a number of physical features including major drains and impacts on residences on the upstream (wet) side of the proposed dyke could be avoided by shifting the last leg of the dyke, which runs west from PR 332, further to the south (Figure 4.11-1). A review of various alternative alignments for the extension to the West dyke, west of Brunkild, was therefore undertaken. This initial review involved a site **reconnaissance** to obtain a qualitative assessment of the potential to re-align the dyke and ascertain if there are alignments west of Brunkild that offer a cost advantage. The potential alignments were reviewed based on the following criteria:

- Topography (height of land).
- Length of dyke.
- Right-of-way width.
- Surface water drainage.
- Dyke foundation conditions.

A new alignment was ultimately selected along the watershed boundary and along existing dirt roads, where feasible. This alignment runs north off of PR 305 along the first dirt mile road east of Brunkild, as shown on Figure 4.11-1. By shifting the alignment to the watershed boundary, the number of drains that have to be crossed is reduced to one and reconstruction of existing gravel roads is minimized. In most cases, it is expected that the dyke crest elevation along the existing gravel roads can be achieved by adding gravel to the existing road surface.



Source: Acres/UMA 2004)

Figure 4.11-1

Considered alternative West Dyke alignments (blue) & selected design alignment (green)

General criteria for the pre-design of the West Dyke are intended as a guide to uniformly safe design. The basis of the design document (Preliminary Engineering Report Appendix F: West Dyke Surveys, Field Investigations and Engineering [Acres/UMA 2004]) evolved throughout the pre-design process as more extensive design criteria were established. The basis of design, used during the pre-design process, is summarized as follows:

- Raise the West Dyke Crest to accommodate a maximum water level of 1:700 year flood event.
- Design wind event was initially the 1:10 year wind event, then increased to the 1:100 year wind event.
- Freeboard requirements shall be designed based on the total wind effect. Minimum freeboard shall be the Wind Setup plus Wave Runup.
- Erosion protection shall be placed on the upstream side of the West Dyke to prevent erosion of the embankment from wave action. The extent of riprap for final design shall be based on hydraulic model testing and three-dimensional numerical modeling.
- Minimize land acquisition.
- Minimize utility relocation.

Hydraulic Design Criteria

The hydraulic design criteria adopted for the pre-design study:

- Still Water Level – equal to a 1:700 year flood event, the design flood for the Expanded Floodway.
- Wave Runup – equal to the maximum runup produced by a 100 year wind event.
- Wind Setup– equal to the maximum wind setup produced by a 100 year 24 hour wind event.

These criteria will be reviewed in the detailed design stage.

These components combine to produce the design water level as follows:

- Design Water Level = Still Water Level + Wave Runup + Wind Setup.

Based on the analysis of erosion protection options, portions of the West Dyke which will experience significant wave heights greater than 0.7 metres (2 ft) during the design condition will be armoured with riprap. The remaining dyke will be armoured using vegetative soil reinforcement.

4.11.2 West Dyke Pre-Design

The pre-design of the West Dyke is based on topographic surveys, field investigations, laboratory tests results, available data and the design criteria, including the following considerations:

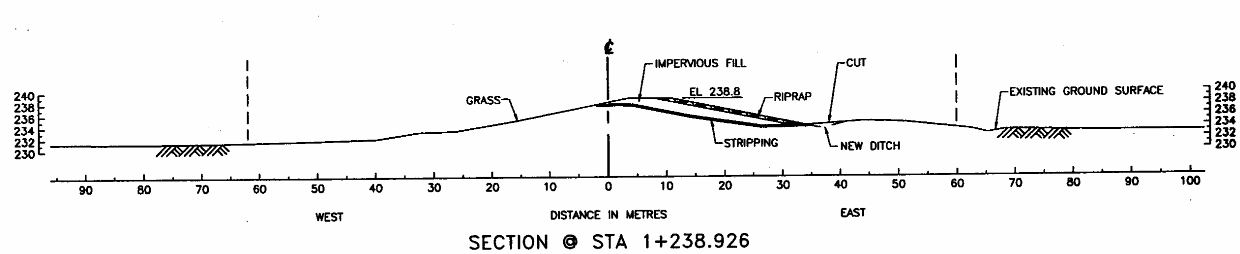
- Constraints associated with existing right-of-way limitations.
- Constraints associated with existing utility corridors and crossings.
- Modifications required to existing parallel and through-dyke drainage and associated control structures (e.g. Domain, Manness Control Structures and Bolen Drain).
- Identify parallel and through-dyke corridors and crossings which may require relocation.
- Incorporation of new through-dyke drainage culvert/gate control structure located at the NE corner of Section 15-8-2-E (e.g. Gleanlea Control Structure).
- Culvert extension for both field and farm access.
- Roadways crossing the West Dyke shall be widened and regraded to match the new dyke elevations.
- 20 km of ditch grading to improve drainage.
- Erosion protection grass slopes for waves <0.7 metres (2 ft) and riprap for waves >0.7 m (2 ft).
- Additional land acquisition needs.
- Stability analysis for the end of construction, long term and flood conditions.
- Evaluation of riprap requirements.
- Wave protection options.

The dyke crest elevation will be increased by varying amounts with the corresponding increased width of the dyke cross section extending into or beyond the existing right-of-way limits. The proposed western extension of the dyke, between Stations 45+540 to 63+601 will be located primarily along the existing road allowances and/or drainage rights-of-way.

In general, the West Dyke will be an earthfill embankment consisting of silty clay compacted to 95% Standard Proctor Density. The crest width for all sections of the dyke shall be set at 7.4 m (24 ft). A layer of road topping material will be placed on the crest of the dyke. All municipal access roads and ramps onto the West Dyke will be widened and regraded to match the new dyke elevations. Grassed slopes shall be used for waves <0.7 metres (2 ft) and riprap underlain with geotextile filter fabric will be used for sections exposed to waves exposed to >0.7 m (2 ft).

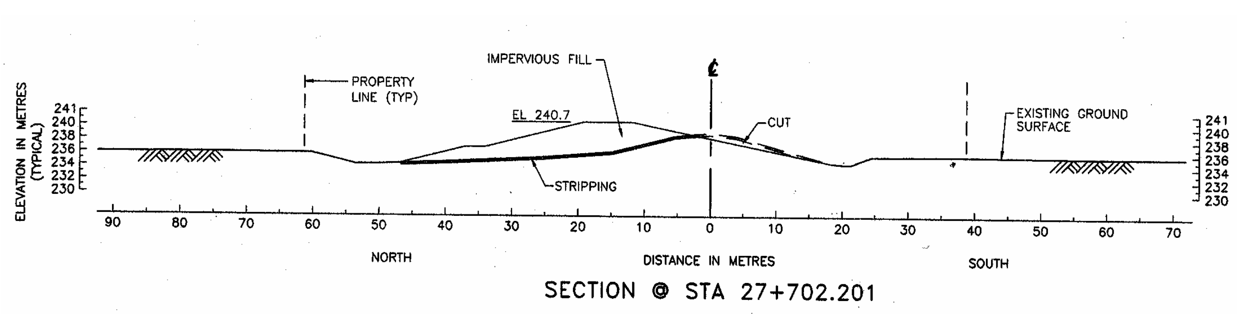
Approximately 44.8 km (27.8 miles) of the West dyke will be raised and widened between Stations 0+280 to 0+600 (on the east side of highway 75) and from Stations 0+744 to 45+540. The West Dyke will be extended approximately 18 km (11.2 miles) beyond its current location at Station 45+540 up to Station 63+601 (See Figure 1-1). The dyke extension will consist of raising existing Municipal roads.

The upstream and downstream slopes of West Dyke extension will be set at 1v:4h. Typical cross sections of the West Dyke are shown in Figures 4.11-2 and 4.11-3.



Source: Acres/UMA 2004

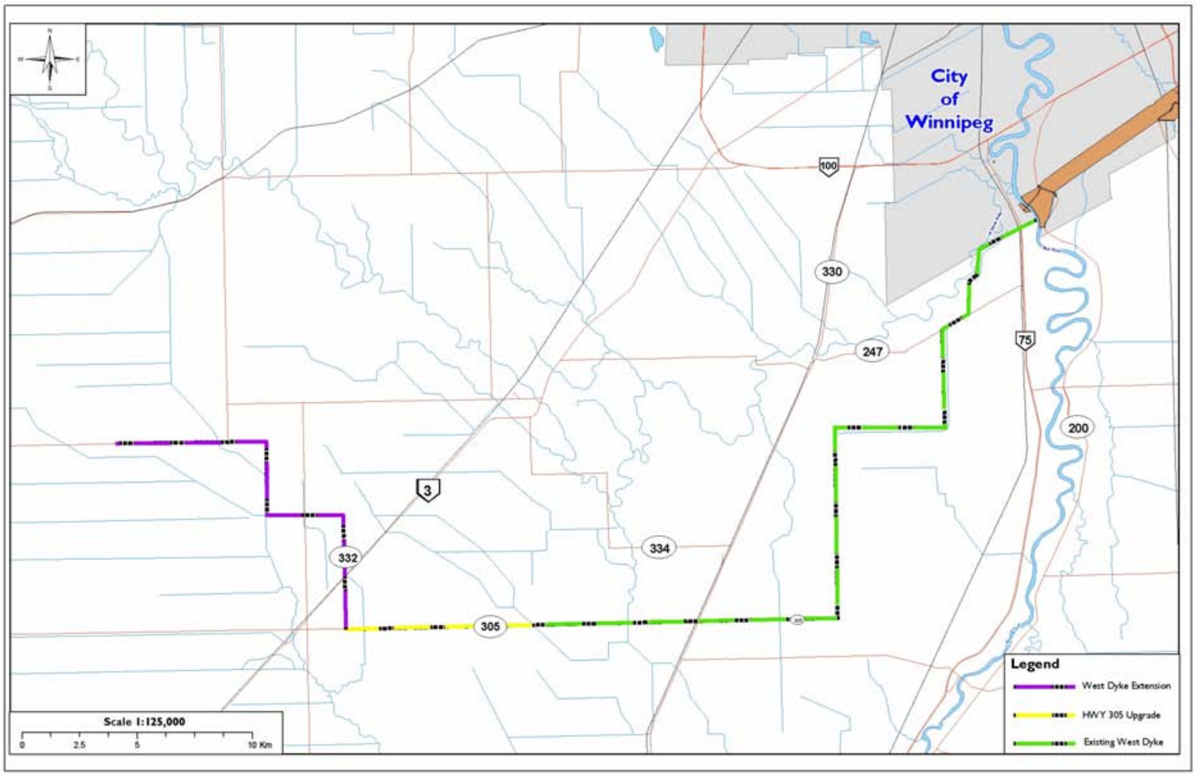
Figure 4.11-2
Typical cross sections of the West Dyke (East)



Source: Acres/UMA 2004

Figure 4.11-3
Typical cross sections of the West Dyke (West)

The crest elevation of the West Dyke will be governed by the still water conditions during a 1:700 year flood event in combinations with the 1:100 year wind event and upstream slope geometry. The design crest elevation varies throughout the length of the West Dyke. Details of the crest elevations for various zones are described in "Preliminary Engineering Report: Appendix F West Dyke Surveys, Field Investigations and Engineering" (Acres/UMA 2004). The crest elevation will be set at el 238.8 metres (783.3 ft), for Zone 1, el 240.7 for Zones 2 and 3, and el 239.6 metres (785.9 ft), for Zone 4. The zone locations are shown in Figure 4.11-4. A summary of the pre-design crest elevation and embankment geometry is provided in Table 4.11-4.



Acres/UMA 2004

Figure 4.11-4
Location of Zones for West Dyke

The upgraded West Dyke will have a maximum height of 6 metres (19.7 ft) above prairie and a maximum height of 10 metres (33 ft) above ditch invert (Station 2+100).

The upgraded West Dyke is anticipated to be capable of providing protection against water levels of approximately 237.1 metres (778 ft) plus freeboard at the Floodway Inlet.

Table 4.11-1
West Dyke Pre-Design Summary

Dyke Zone	Station		Design Crest el (m)	Freeboard (m)	Side of Dyke to Upgrade	Side Slopes		Erosion	
	From	To				D/S	U/S	D/S	U/S
1	0+000	0+600	NC	>1.5	N/A	E	E	G	R
1	0+744	15+800	238.8	1.5	South/East	E	1v:5h	G	R
1	15+800	20+740	238.8	1.5	East	E	1v:5h	G	G
1	20+740	23+660	238.8	1.5	East	E	1v:5h	G	G
1	23+660	23+780	Varies	>1.5	East	E	1v:5h	G	G
1	23+780	23+950	240.7	>1.5	North	1v:4h	1v:5h	G	G
1	23+950	24+244	240.7	>1.5	North	1v:4h	1v:5h	G	R
2	24+244	24+450	240.7	>1.5	North	1v:4h	1v:5h	G	R
2	24+450	29+844	240.7	1.5	North	1v:4h	1v:5h	G	G
3	30+200	33+740	240.7	1.7	North	1v:4h	1v:5h	G	R
3	33+740	45+540	240.7	1.7	North	1v:4h	1v:5h	G	G
4	45+540	47+600	239.6	0.6	East	1v:3h	1v:4h	G	G
4	47+800	50+465	239.6	0.6	West	1v:3h	1v:4h	G	G
4	50+465	53+760	239.6	0.6	North	1v:3h	1v:4h	G	G
4	53+760	54+000	239.6	0.6	Centered	1v:3h	1v:4h	G	G
4	54+000	55+440	239.6	>0.6	N/A	E	E	G	G
4	55+440	56+060	239.6	0.6	Centered	1v:3h	1v:4h	G	G
4	56+060	58+100	239.6	>0.6	N/A	E	E	G	G
4	58+100	58+660	239.6	0.6	Centered	1v:3h	1v:4h	G	G
4	58+660	63+601	239.6	>0.6	N/A	E	E	G	G

Source: Acres/UMA 2004
R – Riprap
G – Grass cover
E – Existing
NC – No change
N/A – No upgrade required, only road topping shall be placed.

Slope Protection

Based on the erosion protection evaluation, the slope protection options selected for the West Dyke are a combination of Riprap and Vegetative cover. Riprap erosion protection will be placed on the upstream slopes of the dyke (water side) in selected areas. Riprap will also be installed at Avonlea corner. All riprap installations will be underlain with a geotextile filter fabric. Vegetative cover will be used in all other areas of the dyke as erosion protection.

Seeding

The preparation of seed bed shall be accomplished by placing the stripped topsoil onto the embankment slopes and harrowing the ground. The grass seed shall not be sown at a depth greater than 25 mm (8 inches). The grass seed mixture will not be broadcast. The proposed grass seed will be Canada No. 1 Grade seed mix as follows:

- Alfalfa 12%.
- Brome 16%.
- Meadow fescue 25%.
- Creeping red 25%.
- Russian wild rye 16%.
- Alsike clover 6%.

Drainage Ditches

The pre-design reviewed the possibility of improving the ditch drainage system parallel to the West Dyke. It is estimated that approximately 20 km (12 miles) of ditch grading is required to remove overgrowth and re-establish the invert ditch grades. Some ditch grade adjustments have been proposed in the pre-design. The pre-design attempts to maintain a minimum ditch gradient of 0.03% and a minimum ditch cross-fall of 0.03%.

The pre-design assumed 0.3 metres (1 ft) of stripping shall be required at all borrow locations. Also it is deemed that additional surveying is required to firm up the Borrow Sources to optimize its usage and further minimize land acquisition.

Land Acquisition

The pre-design considerations have minimized the proposed land acquisition requirements (involving farm land). It is expected that approximately 160 ha of land acquisition may be required along the existing West Dyke right-of-ways. No land acquisition is deemed required for the western extension of the West Dyke between Stations 53+765 to 63+601.

Pre-Design Gleanlea Control Structure

The proposed Gleanlea drain is located at NE 1/4 15-8-2E. The drain is located along the west side of the municipal road, east of Sections 22, 27, 34-8-2E to the La Salle River. The drain has a 0.02% slope from the West Dyke to the La Salle River. The drain is to have 1v:4.5h slopes along the edge of the road and 1v:4h slopes along the prairie embankments.

Natural resources drawing P12-5-1816 indicates that the required flow for the 4 metres base drain is 5 cms. Based on the limiting criteria it was found that two 1.5 metres (5 ft) diameter pipes are required to pass the 5 cms (175 cfs) flow through the West Dyke.
To prevent the release of water through the structure during a flood event a control structure with sluice gates is required. The concrete control structure is located on the upstream side of the drainage pipes approximately 17 metres (56 ft) from the centerline of the West Dyke.

The two 1.5 metres diameter pipes are cast into the headwall and the sluice gates are mounted to the vertical upstream face of the headwall. The headwall is 5.0 metres wide and extends to el 237.60 metres to contain the dyke and provide a barrier for vehicular access to the sluice gates.

The wingwalls extend parallel from the headwall to retain the soil from the dyke and are 5.6 metres in length. The top elevation of the wall varies over the length of the wall from 236.8 metres at the headwall to 235.0 metres at the end of the wall.

A key extends below the base slab of the headwall and wingwalls to improve the stability of the components. The headwall and wingwalls were all sized to meet the design criteria.

The two 1.5 metres (5 ft) diameter corrugated metal pipes extend 46.2 metres (152 ft) from the control structure through the dyke to the outlet. The outlet channel is lined with riprap immediately downstream of the pipes to prevent erosion from the pipe discharge. Limits of riprap shall be determined in final design.

<u>Existing Control Structures</u>

The following two concrete drainage control structures were identified along the existing West Dyke alignment:

- Domain Drainage Control Structure.
- Manness Drainage Control Structure.

The pre-design assumes no changes are required to the concrete structures themselves. At these locations the pre-design proposes that the dyke upgrade take place on the north side of the West Dyke. The drainage culverts shall be extended on the north side and an access ramp complete with road topping material shall ramp down off the dyke crest to a granular parking pad approximately 6 metres deep.

The existing erosion protection on the north side of the structures shall be removed. The drainage culverts shall be extended and the embankment rebuilt with 1v:5h slopes. The outlet of the drain shall be ripraped and underlain with a geotextile filter fabric. The riprap will rise up the embankment to the top of the culverts.

There are culverts adjacent to the drainage control structures, which drain the West Dyke internal ditches into the drainage channels. These culverts and field access ramps may require relocation. Other through dyke culverts were identified along the West Dyke alignment. It has been deemed that these culverts can simply be extended as required.

<u>Utilities</u>

The following utilities were identified within and adjacent to the existing West Dyke right-of-way:

- Manitoba Hydro lines.
- Central Gas lines.
- Trans-Canada Pipelines.
- Municipal Water Lines.
- MTS Lines.

The location of all the identical utilities are shown in the Plan and Profile Drawings FE-PDEA-1254G-03.C Sheets 001 to 026. Three major electrical transmission power lines were identified crossing the West Dyke. At station 34+420, the Manitoba Hydro Line Laverendrye to Letellier (230 kV Y51L transmission line – Gulfport located on 36m (120 foot) wide easement may require the relocation of two Gulfports (towers) to increase the ground clearance over the dyke crest. Alternatively, the dyke could be constructed at a lower elevation.

The other two transmission lines are located parallel to PTH 3. They cross the West Dyke between Stations 47+600 and 47+820. The pre-design of the West Dyke between Stations 47+600 to 47+850 will be a dyke closure point. Therefore, it is deemed that the transmission lines do not require relocation.

4.11.3 Construction

The construction sequence for the West Dyke will be performed over two years, depending on the number of contractors working on the project. The construction season will be between the months of May and November. The West Dyke schedule is relatively independent from the other flood activities. The only potential delay in the start of construction could be the relocation of the utilities as the utility companies will be busy on the relocations associated with the Floodway Channel. Land acquisition delays could also affect the West Dyke Schedule.

The work will have to be scheduled in such a manner as to allow ingress and egress to all residential and business properties, with minimal disruption. In addition, between Stations 0+744 to 7+000 the construction activities will be scheduled for the autumn in an effort to minimize the impact to Parkland Mews' Peregrine Falcon Breeding Program.

4.12 ANCILLARY PROJECT COMPONENTS

4.12.1 City of Winnipeg Flood Improvements

The Floodway Expansion Project contemplates that the City of Winnipeg will implement improvements to its flood protection system, including upgrading of its primary dykes. The capacity of the older combined sewer systems to deal with spring/early summer runoff events and improved isolations of storm drainage and sanitary sewers is expected to be implemented. These upgrades are expected to occur in the latter stages of the Project construction.

4.12.2 Recreational Facilities

The Manitoba Floodway Expansion Authority initiated a process in 2004 to solicit expressions of interest from recreational groups for proposed recreational facility development on lands within the expanded Floodway right-of-way. Specifically, MFEA expressed interested in receiving conceptual plans for recreational developments such as cycling trails, Nordic skiing trails, and other sport/outdoors developments. The potential recreational opportunities that will be considered are expressly stipulated as not interfering with the fundamental operation of the Floodway (Existing or Proposed). As such, no water-based recreation such as rowing channels, white-water rafting, etc. Will be considered.

4.13 OVERALL CONSTRUCTON SCHEDULE

The attached Figure 4.14-1 presents an overall construction schedule for the project (KGS/Acres/UMA, 2004). It incorporates the construction schedule considerations discussed earlier for each major project comment. The schedule has assumed that regulatory approval will be obtained by mid-July, 2005.

The schedule is divided into three phases. Phase 1 covers the project definition and environmental assessment phase of the project, starting with the PDEA1 studies which commenced in February 2003, and concludes with the receipt of regulatory approvals.

Phase 2 covers the final design, tendering and award of contracts, and all necessary preparations for construction, including surveys, land acquisition and the establishment of environmental monitoring programs. Finally, Phase 3 covers all of the construction activities for the project, as well as construction management and contract administration.

In its present form, the construction schedule does not reflect any constraints on the availability of funding. The four and a half year duration of the construction phase of the project is perceived to be a reasonable balance between the utilization of local resources, minimizing the disruption of traffic flows over bridges that require modifications, and completing the work in an acceptable timeframe. Each year of construction will achieve an incremental increase in the capacity of the Floodway to handle larger floods. Any delays in the regulatory review process will impact on the construction phase of the project, as currently scheduled.

The schedule is based on the following key assumptions:

- Final engineering for the project will start in September 2004. This work will include the completion of all remaining surveys and field investigations, the finalization of all design concepts and the development of detailed designs. Construction drawings and tender documents will be prepared in time for the first contracts to be tendered and awarded in the summer of 2005.

- No construction work requiring access to the Channel will be undertaken during April and May, as those are months in which the Floodway has historically been in service, 2 out of every 3 years.
- Excavation of the Main Channel will start at the upstream end of the Floodway and will progress downstream to the outlet of the Channel. The schedule shows that this is the order in which work on the Floodway Expansion Project will generally be undertaken, for the bridges and other structures. Exceptions to this convention may arise, as the detailed design and construction logistics for the project are refined.

4.13.1 Basis of Schedule Development

The points below summarize the basis on which the schedule has been developed.

- Channel: Construction of the Low Flow Channel is scheduled for the winter months, as the channel will have the least flow during that period. Excavation and erosion protection of the remainder of the Channel will be undertaken starting in June and finishing each season at the end of November. Work on the Channel is shown as spanning five construction seasons, with work in the final season being revegetation. However, if necessary, excavation activities may be completed within a much shorter timeframe. The channel excavation durations currently shown in the schedule may change once final designs are completed and a contracting strategy has been established.

- Rail Bridges: The order in which work will proceed on the rail bridges is based on the reuse of bridge girders from the CPR Emerson Bridge, for the construction of temporary detour bridges around the Sprague, Redditt and Keewatin Bridges while they are under construction. This logic requires that the bridge work starts with the Emerson Bridge and the construction of the other three bridges follows, one after the other. The CNR Sprague Bridge is scheduled to follow the Emerson Bridge, to coincide with the timing of work on the Trans Canada Highway (TCH) Bridges .

  The durations for work on the rail bridges range from 6 months for the CEMR Pine Falls Bridge, to 15 months for the CNR Sprague, Redditt and Keewatin Bridges. The schedules for the rail bridges reflect work continuing through the winter and during April and May when the Floodway Channel is inaccessible. Work in the winter will typically be undertaken on the substructures and superstructures. Work in April and May will only be undertaken on elements of the project that do not require access to the Main Channel. The construction work on the rail bridges has been spread over a three year period.

- Highway Bridges: The durations for work on the highway bridges range from 18 months, for the St Mary's Road Bridge, to 29 months for the TCH No.1 East Bridge and PTH 15 Bridge. Work on the highway construction bridges extends over a four year period, and, similar to the rail bridges, will proceed during the winter, and also in April or May on components of the project which do not require access to the Main Channel.

Work on PTH 15 Bridge is shown as starting in 2006. This start date could be adjusted to coincide with the Province's proposed project to extend Perimeter Highway 101.

- Utilities/Transmission Lines**:** Ideally, work on buried electrical cables, communication lines and pipelines, and all overhead transmission line relocations should be undertaken as early in the project as practical. However, the timing of this work, as depicted in the construction schedule, spans four construction seasons, and generally reflects the latest dates by which the crossings and line relocations must be completed to avoid interference with work associated with the Main Channel.

- Outlet Structure: As presently scheduled, this work will take two years to complete. Modifications to the Outlet Structure will be undertaken in three phases. During the first year a new east wall and a new east rollway section will be constructed while the existing outlet structure remains in service. In the following summer season, the existing west structure will be isolated and modifications will be carried out on it, while the new east structure remains operational. The final phase of the work will be undertaken during the second year, after completing the second phase. This will take place in late to mid-fall when minimal flows are expected, and will include the excavation of earth and rock, and the subsequent construction of the remaining rollways.

- West Dyke: Construction of the West Dyke is relatively independent of all other construction activities on the project. One potential delay in the start of construction could be the relocation of the utilities, as the utility companies will be busy on the relocations associated with the Floodway Channel. Land acquisition delays could also affect the West Dyke schedule. Construction will take two years, with each construction season spanning the months of May to November, inclusive.

- Aqueduct Modifications: The tie-ins for the work on the Aqueduct are scheduled for 2006 in the months of October and November when flows are at their cyclical lows and weather conditions are favourable. The Aqueduct relocation schedules must be integrated with the construction of the proposed Water Treatment Plant and other City of Winnipeg Regional Water Supply upgrading programs. Based on preliminary discussions with the City of Winnipeg, it appears feasible that the two Aqueduct reconnections could be accommodated in 2006, subject to a review on all City Regional Infrastructure projects.

- Miscellaneous Hydraulic Structures: The schedule shows work on the miscellaneous hydraulic structures as spanning the duration of the project, as construction work on other portions of the of the project dictate.

- Seine River Syphon: Work on the Seine River Syphon is scheduled to coincide with the completion of the CPR Emerson Bridge.

- Transcona Storm Sewer Outlet: Construction of this storm sewer outlet is scheduled to span from November to mid-March of 2006/2007.

- Floodway Inlet Control Structure: Work in the Inlet Control Structure is comprised of two major activities; erosion protection and modifications to the inlet control gates. The erosion protection work will be undertaken in early fall and work on the inlet control gates will be undertaken in late fall, or early winter, when the risk of a flood is minimal.

The schedule will be refined as the project design is finalized and interdependencies between the various construction tasks and activities are identified. The schedule developed in the final design phase will focus on the efficient use of construction labor and equipment, and on minimizing risk associated with interdependent construction activities.

4.14 OPERATION AND MAINTENANCE

Manitoba Stewardship will continue to be responsible for operation of the Floodway as it forms part of the overall Assiniboine and Red River Flood Protection System. The operation of the Expanded Floodway will not change from the operation of the Existing Floodway. The Operating Rules for this operation are discussed in detail in Chapter 5 (Physical Environment), as the rules relate to water level regimes.

The maintenance of the project will be the responsibility of MFEA, as owners of the Floodway assets.

Manitoba Water Stewardship's mandate, until proclamation of the Floodway Authority Act, includes responsibility for operation and maintenance of the floodway infrastructure pursuant to the Water Resources Administration Act. Under the Floodway Authority Act, the Floodway Authority's mandate has been developed to transfer the physical assets of the floodway and West Dyke from Water Stewardship to the Floodway Authority. Responsibility for on-going maintenance of the Floodway and West Dyke is to be similarly transferred to the Floodway Authority, while Water Stewardship retains responsibility for the coordination operation of these facilities and the Provincial flood control works on the Assiniboine River.

The Floodway Authority will develop an annual maintenance program for the expanded floodway. Within the channel, these activities typically include maintenance of the channel vegetative cover, clean up and disposal of debris deposited during flood flows and maintenance/minor repairs to hydraulic structures. Outside of the channel, typical annual maintenance activities can include pick-up and disposal of illegally dumped nuisance rubbish, and vegetation management in the outside drains.

4.15 ENVIRONMENTAL PROTECTION PLAN

MFEA policy dictates that it undertakes projects in an environmentally sustainable manner. The Proposed Floodway Expansion has been designed to avoid or minimize potentially significant environmental impacts where practical and possible. Unavoidable impacts will be minimized, to the extent possible, by following environmental practices outlined in the EIS, Environmental Protection Plans and license approvals.

Project-specific Environmental Protection Plans (EPPs), will be developed and submitted to regulatory agencies for review following the receipt of an Environment Act License and before start of construction, as required by the Guidelines. There will be a number of EPP's in order to reflect the different characteristics of the main Project Components, i.e., the Floodway Channel EPP will be different from the Bridge EPPs. The EPPs will be part of the various contract documents and will include the following information:

- a brief project description including any modification to the Project as a result of contractor methods or conditions of the licensing/approvals process;
- a summary and site-specific mapping of the identified environmental sensitivities and mitigative actions;
- a listing of all federal, provincial or municipal approvals, licenses and permits, and reference to site specific conditions, that are required for the Project;
- a description of general practices and specific mitigating actions relative to project-specific activities including: site clearing, excavation, earthworks, construction, roads, heritage resources, work in water courses, wildlife, hazardous and non-hazardous material (transport, handling, storage and disposal), noise, safety and public access;
- Emergency Response Plans, training and information to address medical and fire events, hazardous material spill response and containment/clean-up; and
- Environmental/engineering monitoring plans and reporting protocols.

The EPPs will include sections that are applicable to Project construction, operation, site clean-up/ rehabilitation and decommissioning (where applicable, such as work areas).

4.16 PROJECT COST ESTIMATE

The cost estimate is $417 Million. This cost estimates includes direct costs only, not including contingencies, interest during construction, etc.