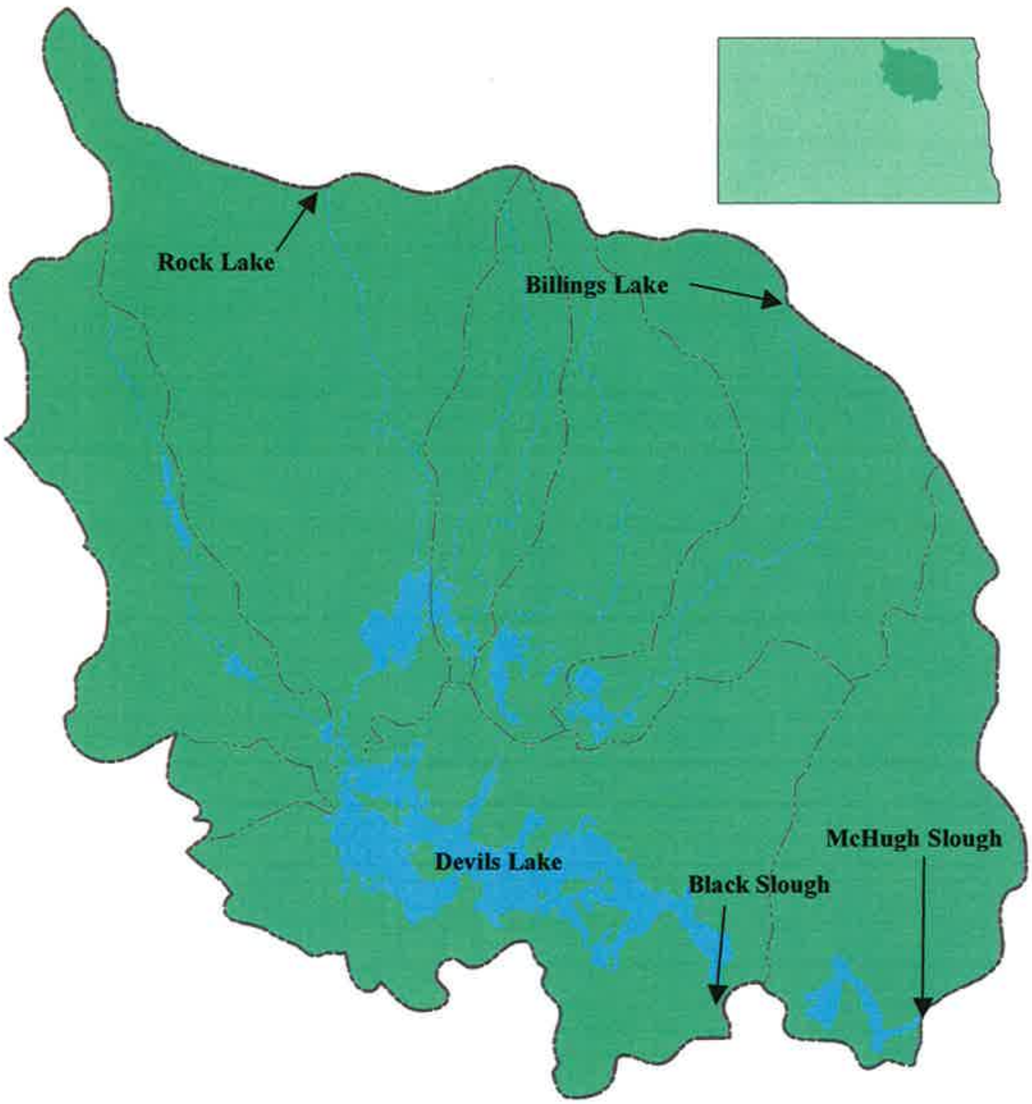
Billings Lake Inter-Basin Connection

SWC Project No. 1882-1



North Dakota State Water Commission
March 2007

Billings Lake Inter-Basin Connection Report

SWC Project No. 1882-1

March, 2007

North Dakota State Water Commission
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1. INTRODUCTION

In many areas in northern and eastern North Dakota, the divides between drainage basins are poorly defined. In some cases, not only are the divides relatively flat but there are wetlands on or near the divide. As a result, movement of water can be influenced by factors such as intensity of precipitation, wind and vegetation. This may allow water and aquatic organisms to move in either direction across the sub-basin boundaries under certain conditions.

In the Devils Lake basin, which is a sub-basin of the Red River basin, there is geological, anecdotal, and documented evidence of natural surface water connections between the Devils Lake sub-basin and other sub-basins of the Red River basin.

Among the areas with anecdotal accounts of connections include the Black Slough, on the south side of Devils Lake, Rock Lake on northern edge of the basin, and McHugh Slough area on the eastern side of the basin. Figure 1 shows the locations of these connections within the basin.

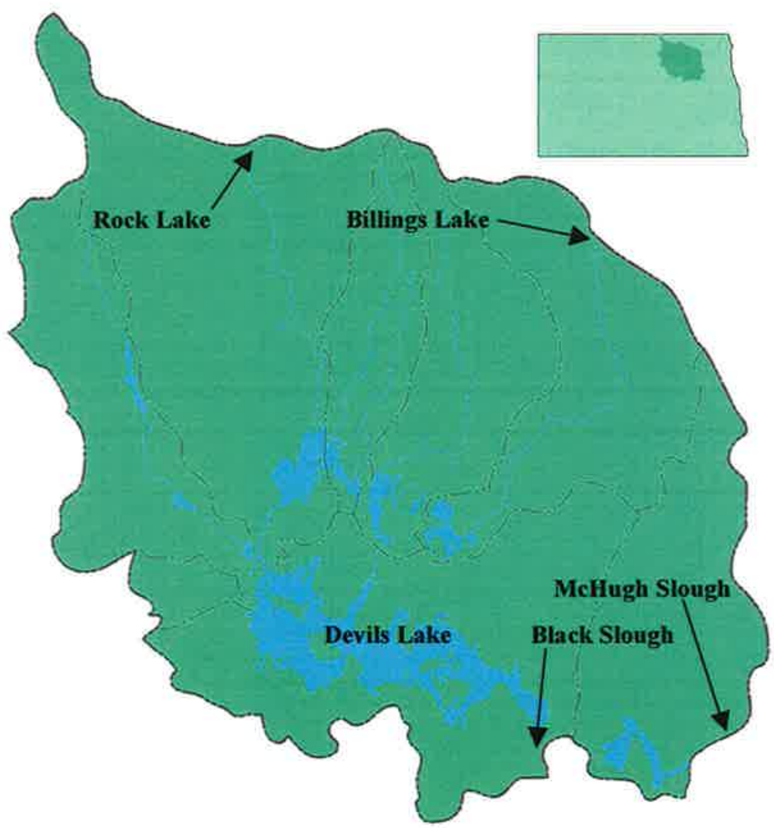

Figure 1. Natural surface water connection locations within the Devils Lake Basin.

One of the best-documented connections occurs in the northeastern portion of the Devils Lake basin, in Cavalier County, near Billings Lake (Figure 1). This connection has occurred for some portion of the last four years, at the southeast corner of Section 26, Township 160 North, Range 61. This location is approximately four miles west, and four miles north of Nekoma, ND.

The first documentation of a connection is aerial color infrared photos taken in 1997 of water flowing north over the township road located on the south side of Section 26. Another documented event occurred in the spring of 2003 when pictures were taken by a Cavalier County Water Resource Board member, whom observed flow to the north that persisted for at least a month. This event occurred after a relatively dry winter in the basin. During the spring of 2004 when the North Dakota State Water Commission (SWC) survey crew documented water flowing north, from the Devils Lake basin, across the basin divide, into the Pembina River basin. The survey crew measured the velocity of the water at this location. Coupling this reading with the height of the water in the culvert, engineers estimated an approximate flow of 15 cfs.

There are also anecdotal accounts of this connection having occurred numerous other times in the last 15 years near Billings Lake. In this area there is a large wetland complex at the basin divide along the township road on the south side of Section 26. Under wet conditions, water flows from this wetland complex mostly to the south, with a lesser flow to the north. To the north of the township road, water flows into Rush Lake, then Snowflake Creek, the Pembina River, the Red River, Lake Winnipeg and ultimately Hudson Bay. To the south of the township road, water flows into Billings Lake, then the Edmore Coulee, and ultimately into Devils Lake.

According to members of the Cavalier County Water Board, this connection has been common knowledge in their county for years, however the point in time of detection of this connection is not known.

This documented inter-basin connection has caused significant concern because common carp (*Cyprinus carpio*) have been found within a mile of the East-West township road and culvert near the sub-basin divide. Carp have never been reported in the Devils Lake Basin. Carp can compete with desirable game fish and introduction of carp into Devils Lake could potentially be detrimental to the fishery.

In October 2004, the Devils Lake Joint Water Resource Board (DLJB) concluded that an investigation of this matter was necessary. The DLJB formed the Billings Lake Sub-Committee consisting of representatives of the DLJB, the Ramsey and Cavalier county commissions, and local landowners to develop a solution acceptable to local interests. The SWC and North Dakota Game and Fish agreed to serve as technical advisors to this committee. In November 2004, in response to the Cavalier County Water Resource Boards' request, the SWC notified the Cavalier County Water Resource Board that the SWC would perform a hydrologic analysis of the Billings Lake inlet channel for the purpose of properly sizing the culverts passing through the railroad embankment in Section 3 of T159N, R161W. This analysis was based on existing survey data provided by Cavalier County and the U.S Bureau of Reclamation, and was conducted to set a basis for existing conditions. It was determined that the culverts existing through the railroad would not pass a 100-year or 50-year 24-hour storm event, but were sufficient to pass a 25-year 24-hour storm. The Billings Lake Sub-Committee requested the DLJB to serve as the lead agency to survey the divide area and develop models to determine the downstream effects (both to Devils Lake and the Pembina River) of closing the divide. As requested by the DLJB, an agreement was drafted in March 2005 between the SWC and the DLJB for this endeavor. Subsequently, when attempting to obtain Right of Entry for surveying the area, some local landowners refused to grant access. The DLJB secured Rights of Entry (one through court action) and raised funds through local entities to pay for the deposit required by the SWC for one-half of the field costs associated with the survey.

The purpose of this preliminary report is to objectively provide the Billings Lake Sub-Committee with a list of possible alternatives for reducing the chance for aquatic organisms to pass across the basin divide into the waters of the Devils Lake basin. This report recognizes that other transfer pathways, such as bait bucket transfer, and other natural surface water connections provide a means for carp to get into Devils Lake over time. However, the Billings Lake pathway seems to be the most probable surface water connection that will give carp access to Devils Lake.

1.1 Permit Requirements

Dependent upon which of the alternatives are chosen, certain permits may be required for construction. If fill is placed into a wetland, a Section 404 Permit will be required by the United States Army, Corps of Engineers. If more than 12.5 acre-feet are to be diverted, then a Water Use Permit will be needed from the State Engineer. If the project is capable of retaining, obstructing, or diverting more than 25 acre-feet of water, then a Construction Permit will be required by the State Engineer.

1.2 Additional Issues

No biota transfer control method can guarantee that carp will not be introduced into the Devils Lake sub-basin. Other transfer mechanisms, such as the bait-bucket effect, and extreme rainfall events have the potential to overwhelm any control method.

2. PROBLEM DESCRIPTION

As shown in Figure 2, the area of interest is a wetland in the northeast corner of Section 35, T.160N, R.61W. The wetland is within the incorporated city limits of Loma, ND in southwestern Cavalier County. The wetland is bounded on the north and east by township roads. There are three culverts on the north-south (N-S) township road, and one culvert, denoted as Culvert #1, on the east-west (E-W) township road.

The wetland is fed by a NW flowing coulee from Nekoma, ND. Water discharges from the wetland in either the SW direction to Edmore Coulee, or north through Culvert #1 toward Snowflake Creek. Edmore Coulee is in the Devils Lake Sub-Basin, and Snowflake Creek is in the Pembina River Sub-Basin.

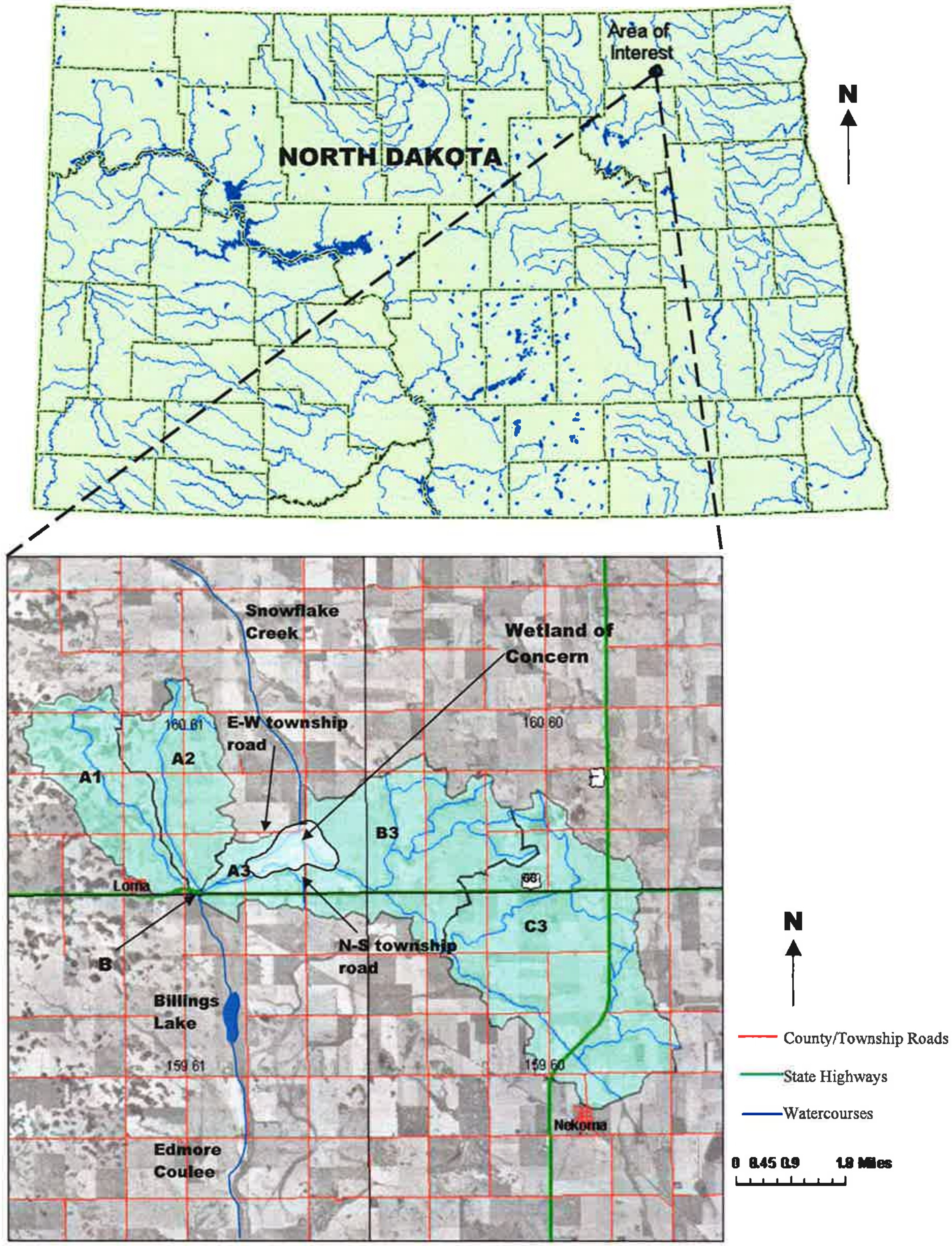

Figure 2. Site location map showing sub-basins and wetland of interest.

Carp have been reported in Snowflake Creek at a location about a mile north of the wetland. Since the wetland is on the Devils Lake/Pembina River Basin boundary, Culvert #1 provides a possible pathway for carp migration into the Devils Lake Basin. Overtopping of the E-W township road can also occur providing a possible pathway for carp migration. In light of this, it has been proposed Culvert #1 be removed and also to possibly raise the E-W township road. Removal of Culvert #1 and raising the E-W township road may increase the flow to the Edmore Coulee, and it is of particular interest to investigate and quantify these possible downstream effects.

3. ALTERNATIVES

3.1 Screen

a) Description: A series of screens inserted into steel walls, and anchored by 4" steel posts, would be installed on both the north and south ends of the culvert on the road on the southeast corner of Section 26. Gravel fill would be placed inside the structure(s) to inhibit vegetation growth. Lynn Schleuter, of North Dakota Game and Fish, provided a possible design that would reduce the chance of carp passage, and also allow for cleaning of the screens. Essentially, the design incorporates of series of increasingly finer screens at either end of the culvert, that are intended to filter out larger pieces of organic matter, such as cattails (*Typha* spp.) so that the fine screens are not plugged.

b) Estimate of Cost: A cost estimate was not developed for construction of this option. Additional costs include the amount of hours that would be required to have someone physically clean the screen.

3.2 Earthen Barrier (At Divide)

a) Description: This option would entail raising the road to 1,582 feet-msl (the low spot in the road is 1,577.5 amsl) on the south side of Section 26, and closing off the culvert. Earthwork would raise the elevation of the divide to prevent a 100-year, 24-hour rain event (4.9") from overtopping the divide under dry starting conditions. An event of this magnitude was chosen because an earthen barrier sized for this event provides a relatively low, theoretically 1%, risk of being overtopped while still being reasonably priced. It should be noted that the flood risk is based upon pre-1993 precipitation records, so the chance of overtopping is somewhat greater than 1%. Hydraulic analysis based on the survey information for 25 year, 24-hour (3.8") to 100 year, 24-hour (4.9") rain events indicated minimal impact to the south, with only an additional 1.0 (25 year) to 2.7 (100 year) acres of additional inundated land in the 3.3 miles of stream between the divide area and Billings Lake. The additional inundated acreage was derived by calculating the increase in top width of the cross sections, compared to the top width of the cross section given existing conditions, within the area. Details and results of the hydrologic and hydraulic models are given in Sections 4, 5 and 6.

b) Estimate of Cost: Raising the township road near the divide to an elevation of 1582 feet-msl should provide three feet of freeboard for any 100-year event. For roughly 1500 feet of road to be raised, approximately 4,800 cubic yards of fill and gravel would be required. Many of the costs associated with this will be variable but a conservative estimate for the raising of the road is approximately $55,400. Preliminary cost estimate details are shown in Table 1. These estimates include excavation of the existing township road, which may not be necessary. Hydraulic analysis of the divide area showed an increase in runoff to Billings Lake of less than ten percent for snowmelt events and less than five percent for rainfall events. A hydraulic model extending from the slough at the divide to just upstream of Billings Lake revealed an increase in inundated land of 1.0 to 2.7 acres for the 3.3-mile reach in question. Effects further downstream would be negligible as Billings Lake will serve to attenuate the hydrograph and additional inflows downstream would have already crested from runoff from adjoining land by the time the water from the divide arrives.

Table 1. Cost Estimate for Raising the Township Road.

	Quantity	Unit Cost	Total Cost
Mobilization/Demob	1	$3,000.00	$3,000.00
Excavation	2000 yds	$3.00	$6,000.00
Gravel	800 yds	$8.00	$6,400.00
Fill	4000 yds	$4.00	$16,000.00
Equipment and Labor	120 hours	$200.00	$24,000.00
			$55,400.00

3.3 Pump Water over Divide

a) Description: This option would be implemented with raising the township road, and would prevent any additional flows into Devils Lake that would be diverted south by the barrier. Pumping water from the south side of the road to the north side would require a detention basin on the south side and an outfall structure on the north side.

b) Estimate of Cost: Use of a single Gator Pump will move 25 cfs, or nearly 50 acre-feet/day if pumping continuously. However, this pump would require frequent supervision, as it is not capable of being set up for automation. Use of another pump with a similar pumping rate as one Gator Pump could possibly cost $100,000 or more. Excavation of a detention basin on the south side of the road would be necessary for collecting a sufficient volume of water to avoid excessive pump cycling. The detention basin will need to be at least 7 feet deep and have a minimum total volume of roughly 0.5 acre-feet. To minimize erosion from discharge flows, a riser pipe surrounded by riprap would be installed at the end of the discharge pipe which would stabilize the outfall. Operations and maintenance costs would include manpower, fuel, and repair of any erosion if necessary. Total cost of this option would be roughly $15,000, which does not include cost of the pump or O/M costs. If this option is implemented, the excavated material (about 850 yards) could be used for fill for the road raise if the material meets specs, thus reducing the road raise cost.

3.4 Electrical Barrier

a) Description: This option would run an electrical charge through the water channel to restrict the movement of fish from one basin to the other. Water would flow unimpeded to the north through the culvert in the southeast corner of Section 26, and then through the channel as it grows more defined to the north. A much larger version (average annual flows of 3,213 cfs) of this treatment process has been used for the Chicago Sanitary and Shipping Canal in the City of Chicago, IL, and a smaller version has been used for over a decade at the J. Clark Salyer National Wildlife Refuge in North Dakota. An electrical barrier option such as is available from Smith-Root, Inc. can be placed in a concrete box culvert. The culvert would be placed in the existing road crossing between Sections 14 and 23 of T160N, R61W (two miles north of the township road which approximates the divide.) A fine mesh trap gate which automatically closes upon loss of power is available as a backup measure and a standby generator can be included to serve as redundant power supply. Trash racks should be included on both sides as a safety precaution since current would be introduced to the water. Systems also exist which include a pump and jets to induce a downstream current in stagnant waters to insure shocked fish (which float to the surface) don't carry on upstream although a stiff north wind may reduce their effectiveness. The effects of partial ice conditions on the barriers effectiveness are also unknown.

b) Estimate of Cost: A 6 foot by 6 foot box culvert would be sufficient at this site, which from Cretex would cost approximately $65,000 installed. The electrical barrier package, which includes pulsators, building, backup power, and power supply, would cost roughly $50,000. Total cost for the barrier would be about $115,000. Power usage should be significantly less than the Salyer system's average of approximately $2500 per year.

4. HYDROLOGIC MODEL

In order to develop a hydraulic model of the wetland a hydrologic model must first be developed to define the discharge hydrographs. The first step in developing a hydrologic model is to estimate the contributing sub-basins at the area of interest. The next step is to determine the response of the sub-basins to rainfall/runoff events, for example a 24-hour, 10-year event. The Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) computer program is applied to determine event hydrographs (sub-basin responses). These hydrographs will serve as inflows for the hydraulic model.

4.1. Sub-basins

The defined sub-basins in the study area are shown in Figure 2. Sub-basin *B3* and *C3* are identified as the contributing sub-basins to the wetland inflow. Sub-basin *A3* is identified as the sub-basin containing the wetland at the border between the Devils Lake and Pembina River Basins. The downstream impacts can be addressed by investigating the hydrographs at point *B* in Figure 2 for the existing conditions, the condition with Culvert #1 removed, and the condition with Culvert #1 removed and the E-W township road raised. A hydrograph is a measure of the time dependent changes in flow or stage at a point in a sub-basin. Typically this point is taken at the outlet of the sub-basin. The existing hydrograph at point *B* would include summing the hydrographs of sub-basins *A1*, *A2*, *A3*, *C3* routed through *B3* and *A3*, and *B3* routed through *A3*. By routing the *C3* and *B3* hydrographs, changes in the hydrographs that occur while flowing through the sub-basins to point *B* can be accounted for.

4.2. Sub-basin Properties

Once the sub-basin areas have been defined, each of the sub-basins must be hydrologically characterized. A summary of the hydrologic properties of the sub-basins is given in Table 2. The curve number, *CN*, (Soil Conservation Service, 1972) characterizes the water losses in the sub-basin, and is based on the coverage associated with each of the sub-basins. The number can range from 0 to 100, where 0 represents 100% water loss, and 100 represents 0% water loss. The lag time is the time between the center of mass of rainfall excess and the peak of the unit hydrograph. The lag time is estimated to be 60% of the time of concentration, t_c, which represents the time it takes a drop of water at the greatest distance from the outlet of the sub-basin to reach the outlet.

Table 2. Hydrologic properties of sub-basins.

Sub-basin	Area, acres	Area, sq. mi.	*CN*	*lag*, hrs	t_c, hrs
A1	2388	3.73	78.1	7.61	12.68
A2	2646	4.13	76.6	7.35	12.24
A3	1134	1.77	78.0	2.91	4.85
B3	4660	7.28	79.1	7.96	11.83
C3	7168	11.20	77.5	17.14	28.57

The data in Table 2 serves as the input parameters into HEC-HMS. The results are event hydrographs for each of the sub-basins. For the spring runoff events the sub-basin imperviousness was set to 95%. Note that the *arbitrary* simulation time chosen was the first week in June 2005 for the 24 hour storm events, and the first two weeks in June 2005 for the ten day events. Therefore the results will be reported over this time period.

5. HYDRAULIC MODEL

To construct a hydraulic model of the wetland, accurate evaluation of the influence of topographic features is needed. This is due to the complex flow conditions that can exist in sloughs or wetlands. These can include for example; indistinguishable flow lines and depression storage due to ambiguous topography. To address these issues, a detailed survey of the area was conducted along with a field visit to identify coverage and hydraulic structures.

5.1. Survey Data

A topographic survey of the divide area was conducted, with the resulting contour map shown in Figure 3. This area includes Sections 25, 26, 35, and 36 in North Loma Township ((T160N, R61W) of Cavalier County. The overall trend is a southwestern sloping topography that is relatively flat around the E-W township road. Notable topographic features include two rises near the center of Section 35, a rise in the SW 1/4 of Section 35, and a ridge in Section 36 extending SE from the junction of the E-W and N-S township roads.

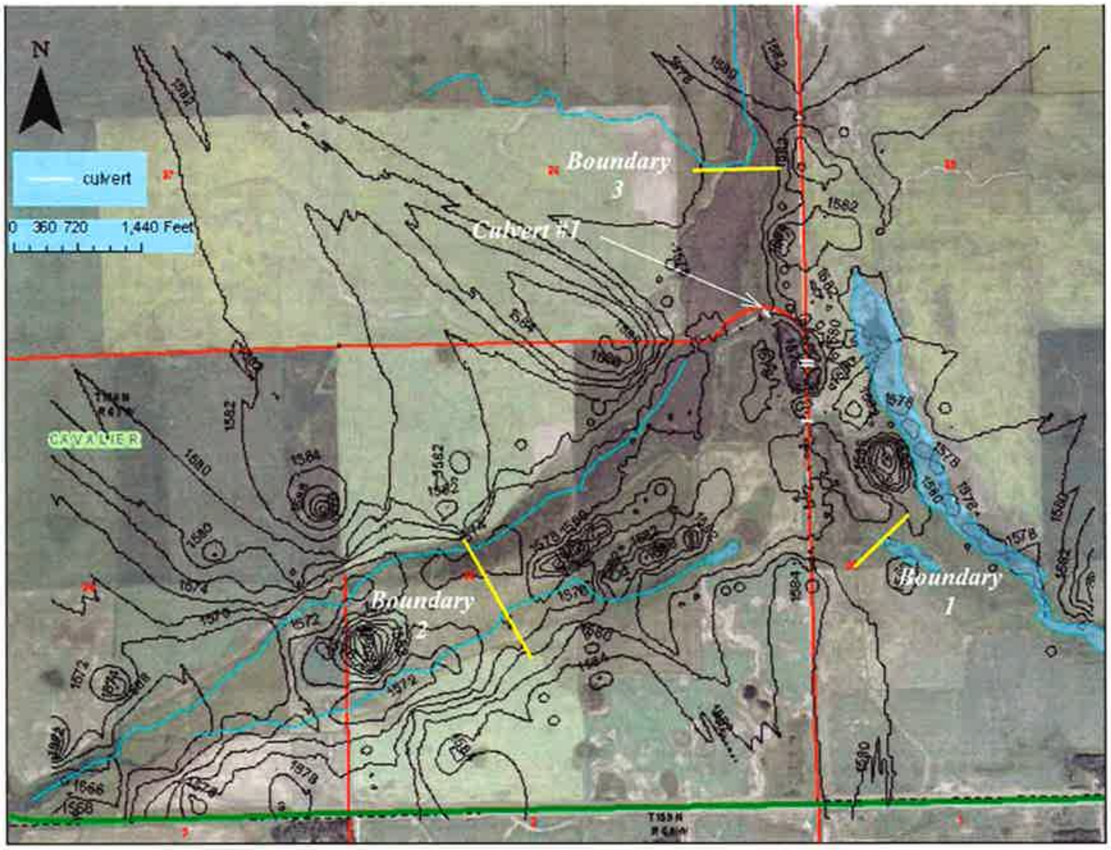

Figure 3. Contour Map (Counter Intervals = 2 ft.) based off survey data. Culverts locations are also given on the township roads.

5.2. Field Visit

On October 19th, 2005 the site was visited. Discharge to the north through Culvert #1 was noted. There was no noticeable flow from the east side of the N-S township road to the west side. Three different types of coverage were identified:

1) *Coverage 1*-Thick growth of cattails and other dense stands of wetland plants. This was observed in the areas of ephemeral flow and standing water.
2) *Coverage 2*-Open channel with intermittent spots of *Coverage 1*. Consisted of areas of pooling with a smooth silt-sand bed.
3) *Coverage 3*-Thick prairie grass. Areas typically not submerged except in times of high water.

5.3. Conceptual Model

Based upon the field visit, the interpretation of survey data, and the HEC-HMS results of the area, a conceptual model can be developed in which to numerically model the fluid flow.

5.3.1. *Boundaries*

Developing constraints for the model boundaries is important before developing an actual numerical model. Model boundaries are typically chosen as hydraulically stable (free from hydraulic structures and abrupt flow area changes) regions away from the primary area of modeling interest. For our purposes, the primary area of modeling interest is the zone near the culverts of the N-S and E-W township roads (see Figure 3). Therefore, three hydraulically stable boundaries a sufficient distance from this area were chosen. These boundaries are shown in Figure 3. *Boundary 1* represents where the inflow is coming into the wetland, *Boundary 2* represents a hydraulically stable reach of outflow to the Edmore Coulee, and *Boundary 3* represents a hydraulically stable reach of outflow to Snowflake Creek. However, flow patterns between these boundaries are unknown. This problem will be clarified in *Sec. 5.3.3*.

5.3.2. *Hydraulic Properties/Structures*

The following hydraulic structures to be modeled in the area of interest are:

1) 3 culverts on the N-S township road
2) Culvert #1 on E-W township road
3) 2 weirs, i.e. the N-S and E-W township gravel roads

The culverts are all approximately 32 foot in length, 3 ft. in diameter, and are constructed of corrugated metal. All the culverts are slightly projecting from the embankment. The roads are assumed to be 25 feet wide.

Based on the descriptions given in *Sec. 5.2*, the following roughness coefficients are assigned:

a. *Coverage 1-* $n = 0.115\text{-}0.195$
b. *Coverage 2-* $n = 0.085\text{-}0.115$
c. *Coverage 3-* $n = 0.05\text{-}0.085$

Roughness coefficients describe the resistance the ground surface has in the conveyance of flow. The n values reported above are high. However, these values are typical for marsh/wetland type environments, which tend to store water or resist water movement.

5.3.3. *One Dimensional and Two Dimensional Surface Flow*

Fluid flow in natural channels is typically modeled as one dimensional flow where it is assumed the velocity is constant at any given cross section with respect to depth and width. Two dimensional surface flow assumes that the fluid velocity is constant with respect to depth, but can vary across the channel width. Figure 4 illustrates one dimensional and two dimensional fluid flows. Note that the one dimensional velocity profile is the average velocity of the two dimensional velocity profile. Both have their benefits and drawbacks. One dimensional flow is easier to apply and model. However, the assumption of an unvaried velocity over a cross section is not always applicable. Two dimensional flow allows for better definition of the flow pattern. However, in areas characterized by extensive wetting/drying and little relief (like a wetland), a two dimensional model cannot always give reasonable results.

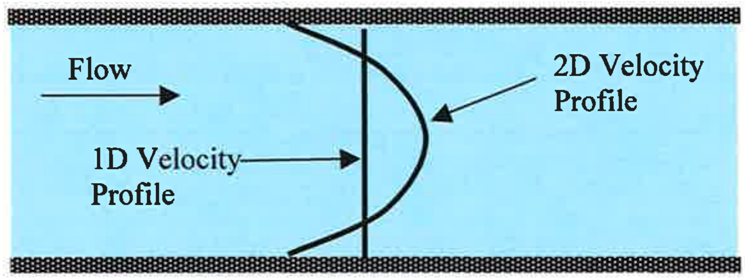

Figure 4. One dimensional and two dimensional flow in a channel (plane view).

In light of the fact there is diverging flow in the wetland of interest, a steady state (flow constant with respect to time) two dimensional model to identify the flow pattern was applied. Once the flow pattern is defined this will allow for cross sections to be cut perpendicular to the flow regime allowing for an accurate one dimensional transient (flow varying with respect to time) model of the area to be constructed. This one dimensional model can then be applied to the 24-hour and spring runoff events.

5.3.4. Two Dimensional Model

A two dimensional model is developed using the finite element method (FEM), which breaks up the solution domain (wetland for our case) into a finite number of connected triangular or rectangular elements that form a "mesh". Each finite element contains nodes, which are points located at the corners and midpoints of the finite element sides. Each of these nodes is assigned an elevation based on the survey data. Once boundary conditions are assigned to the boundaries as denoted in Figure 3, the fluid velocity and stage is calculated in each of the elements. Figure 5 shows the finite element mesh used for the study area, and the resulting steady state flow pattern in the wetland. Each flow arrow represents an element. Note that the arrows are at a greater density near the culverts, which is a result of the model being more discretized in this area. A diverging flow pattern on the west side of the N-S township road is evident. Also noted is the weakly defined flow on the north side of the E-W township road. This implies that the inflow from Culvert #1 is having little impact on the flow conditions to Snowflake Creek.

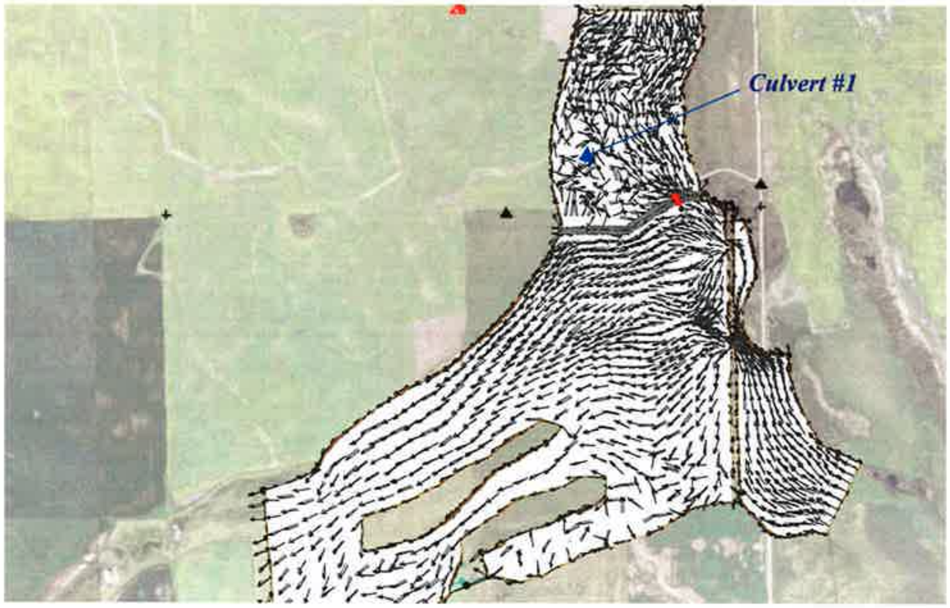

Figure 5. Flow pattern showing divergent flow.

5.3.5. <u>One Dimensional Model</u>

Since the flow pattern has been identified by the two dimensional finite element model, cross sections can now be cut perpendicular through the flow pattern and applied to a one dimensional model. The HEC River Analysis System (HEC-RAS) will be applied.

5.3.5.1. Cross Sections and Hydraulic Structures

The cross sections used for the one dimensional model are shown in Figure 6, along with the survey points. Comparison of Figure 5 and 6 illustrates that *cross section 7.5* required a "dog leg" to allow for the one dimensional model to handle the diverging flow pattern west of the N-S township road.

The N-S township road is modeled as a weir with three culverts contained in it. This assumes that when the road is overtopped, the water flows over the road as a weir. The E-W township road is modeled as a lateral structure with a culvert, because according to the two dimensional model the majority of the flow is parallel to the E-W township road.

5.3.5.2. Boundary Conditions and Simplifying Assumptions

Boundary conditions are applied to *cross sections 11* and *-6*. The upstream boundary conditions applied to *11* are the combined hydrographs of sub-basin *B3* and *C3* from the HMS hydrologic model. These hydrographs were found by routing the *C3* outflow hydrographs through sub-basin *B3* by application of the Muskingham routing method (e.g. see Chow et al., 1988; pg. 257). The lag time of sub-basin *B3* was used, which was 7.96 hours. A weighting factor of $X = 0$ was applied (assumes linear sub-basin). The routed hydrographs

Figure 6. Map illustrating cross sections, culverts, and roads modeled as weirs for one dimensional model. Survey points are also shown.

from sub-basin *C3* were then added to the hydrographs of sub-basin *B3*. This yielded combined inflow hydrographs into the area of interest. These are plotted in Figure 7.

The downstream boundary condition applied at *cross section -6* is a normal depth condition. This specifies what the slope of the water surface is at *cross section -6*. In hydraulically stable regions the slope of the water surface is often the slope of the ground, which in this case is 0.0015.

Since there is a road embankment, ineffective flow areas must be defined. These are the areas around either side of the N-S township road which do not convey flow. This is a result of the water only being allowed to flow through the culverts, thus greatly reducing the total conveyance of the cross sections on either side of the road embankment. Generally for culverts in a roadway the ineffective flow area on each side of a culvert decreases by a 1 to 1 ratio upstream and a 1 to 1.5 ratio downstream. These ratios were applied to this model. After the road is overtopped the downstream ineffective flow areas are turned off, because the effective flow area has increased.

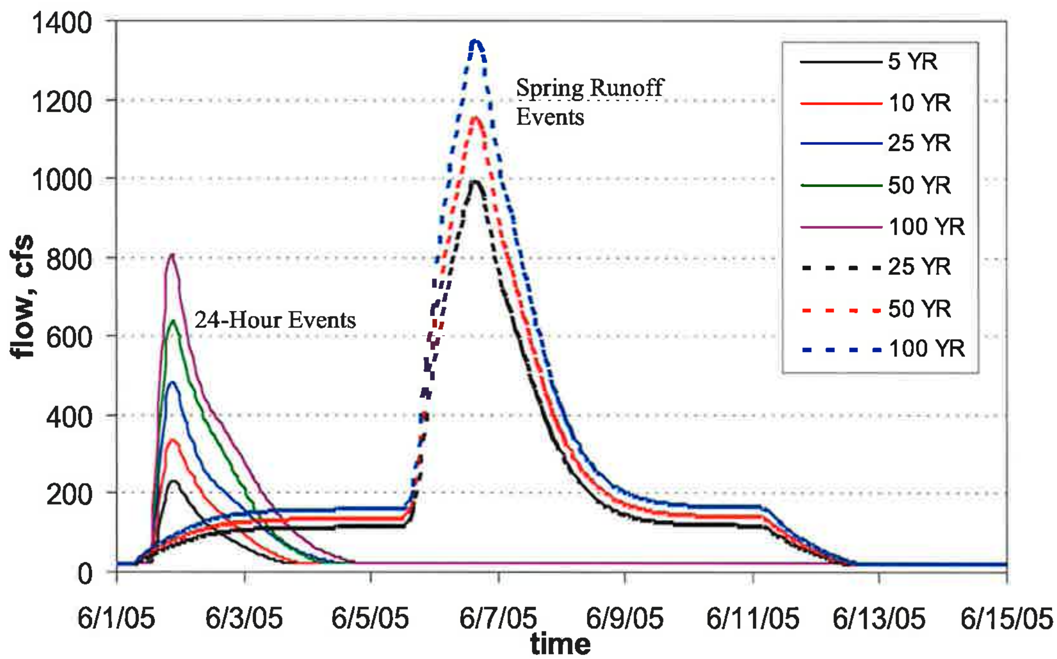

Figure 7. Inflow hydrographs for hydraulic model at *cross section 11*.

6. RESULTS

Three HEC-RAS models of the area of interest were created. The first modeled the existing conditions, the second modeled the conditions with Culvert #1 removed, and the third modeled the conditions of Culvert #1 removed and the E-W township road raise. For each of the models a 5-year, 10-year, 25-year, 50-year, and 100-year 24-hour storm event was simulated. Also, ten day spring runoff events for a 25-year, 50-year, and 100-year event were simulated. The model results reflect the new culverts (7' CMP) that were recently installed under the railroad bridge.

6.1. Existing Conditions and Results of Culvert #1 Removal

6.1.1. Effect of Discharge

In Figure 8 the outflow hydrographs at *cross section -6* for the existing conditions are plotted. In Figure 9 the relative percentage increase caused by removing Culvert #1 is reported. There were no differences in the 5-year and 10-year events. The initial spike in the 24-hour event curves in Figure 8 represents the time in which the water level is rising, but has not yet overtopped the E-W township road. The saddle shape in the spring runoff curves illustrate that the relative percent increase drops due to the high flow rates in the center of the flow hydrographs given in Figure 7. When the E-W township road is overtopped, the weir-type overflow conveys the majority of the flow to Snowflake Creek (elaborated on in *Sec 6.1.5*).

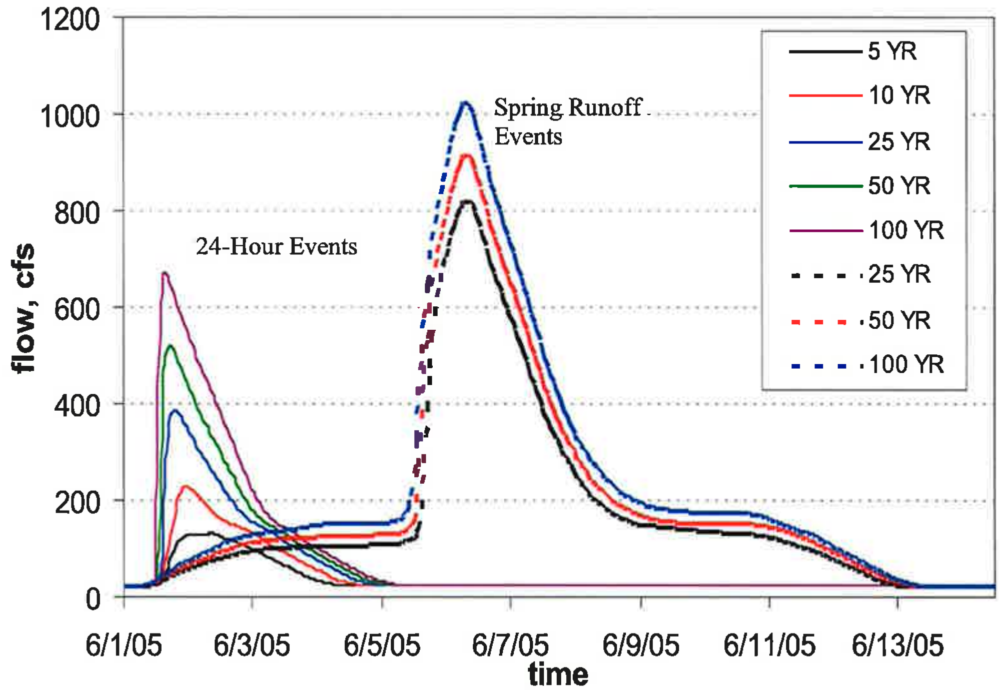

Figure 8. Outflow hydrograph at *cross section -6* for hydraulic model with existing conditions.

6.1.2. *<u>Effect on Stage</u>*

In Figure 10 the stage hydrographs at *cross section -6* for the existing conditions is plotted. When Culvert #1 is removed the resulting increase in stage is reported in Figure 11, which shows that the increase is extremely small. The trend and shape of Figure 11 is similar to Figure 9. The increase is under a quarter inch. This implies that the area of inundation downstream if Culvert #1 is removed will not be significantly affected.

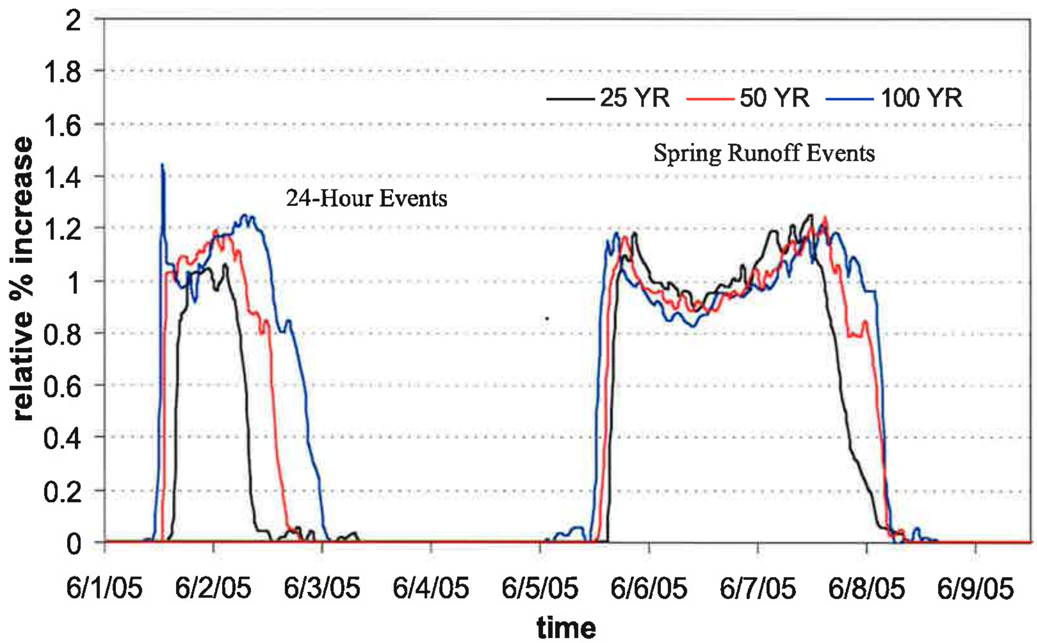

Figure 9. Relative percent difference in outflow caused by removal of Culvert #1.

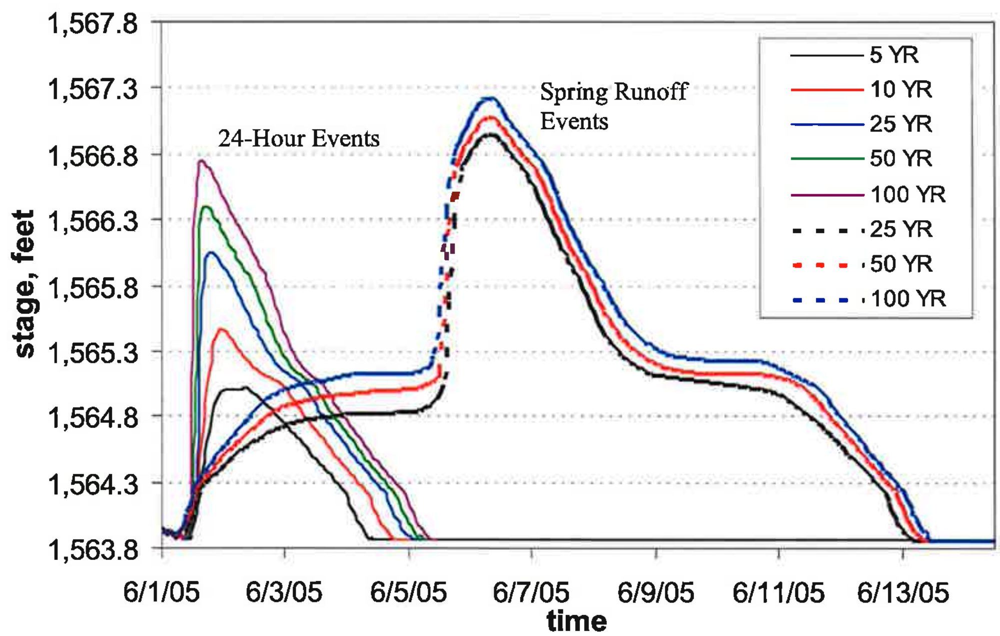

Figure 10. Stage hydrograph at *cross section -6* for existing conditions.

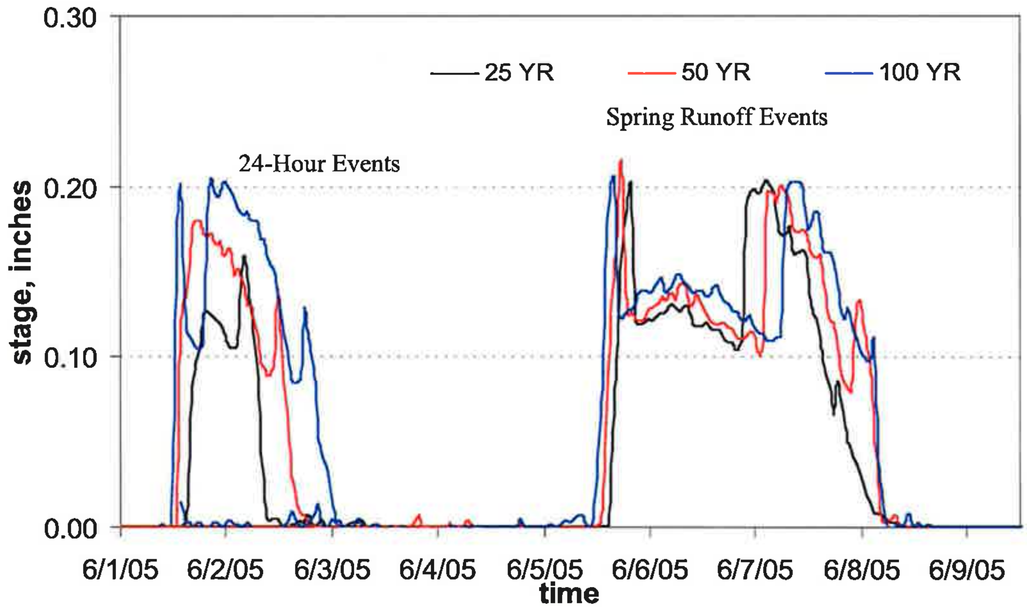

Figure 11. Stage difference between the case of Culvert #1 removed and existing conditions.

6.1.3. Effect on Volume Discharged

To find the total volume of water discharged over a specific time the discharge is multiplied by the total time of discharge. This in essence is calculating the area under the flow hydrographs. The total amount of volume discharged is plotted in Figure 12, and the resulting increase in volume discharged from removal of Culvert #1 is plotted in Figure 13. There is no difference in the 5 and 10-year events. Figure 13 represents the amount of water being diverted from the Pembina River to Devils Lake. The increase in total volume discharged as a result of removing Culvert #1 is summarized in Table 3. The percent increase in volume discharged is less than a percent of the total volume discharged under existing conditions.

Table 3. Increase in volume discharged at *cross section -6* with Culvert #1 removed.

Event	Increase in Volume Discharged (acre-ft)	% increase
24 hour 25 year	4.22	0.39
24 hour 50 year	8.36	0.60
24 hour 100 year	13.49	0.76
10 day 25 year	25.8	0.56
10 day 50 year	30.2	0.56
10 day 100 year	34.7	0.57

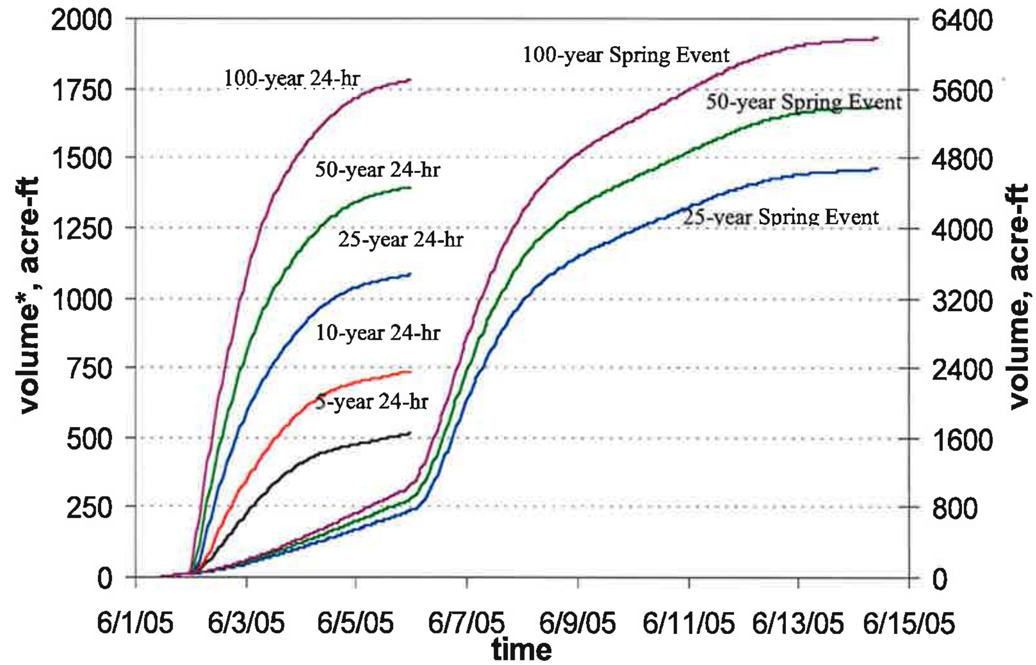

Figure 12. Volume discharged from hydraulic model at *cross section -6* for existing conditions. Axis with * corresponds 24 hour storm events.

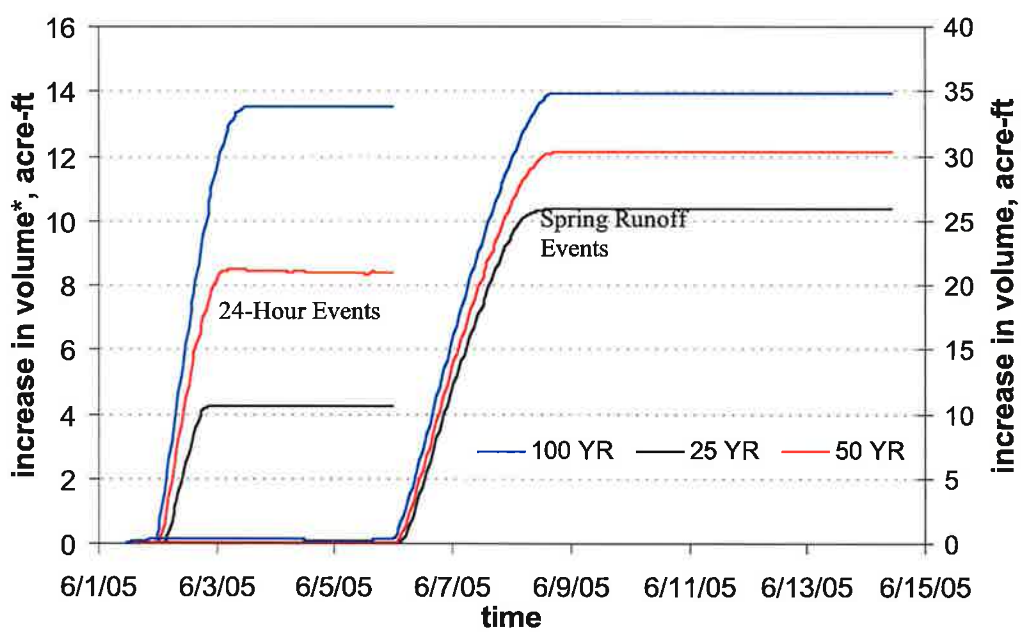

Figure 13. Increase in volume discharged as a result of removing the Culvert #1. Axis with * corresponds with 24 hour storm events.

6.1.4. Discharge from Culvert #1

The discharge through Culvert #1 to Snowflake Creek for the 24-hour and spring runoff events is plotted in Figure 14. Maximum flow through the culvert is about 10 cfs for the 24-hour events and 16 cfs for the spring runoff events.

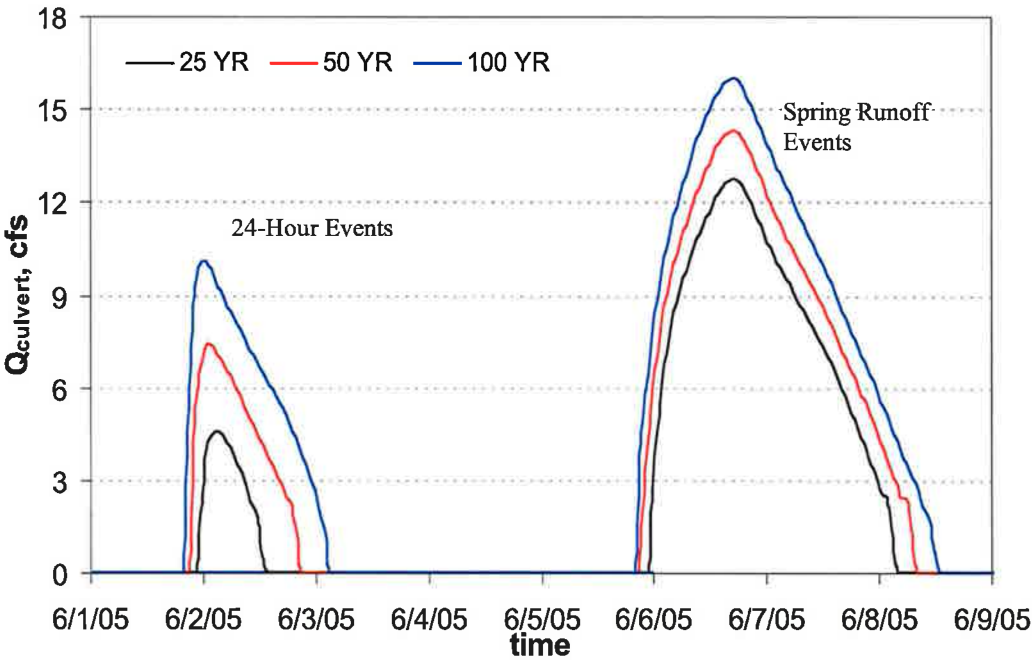

Figure 14. Discharge of Culvert #1 to Snowflake Creek.

6.1.5. Discharge over E-W Township Road

In Figure 15 the weir flow over the E-W township road has been plotted. In comparing these discharges to those of Culvert #1 it appears the majority of water during the 100 year events is discharged toward Snowflake Creek via weir flow over the E-W township road. The increase in weir-type flow over the E-W township road caused by removing Culvert #1 is plotted in Figure 16. Although the weir flow has increased, comparison with Figure 14 shows it does not account for all of the discharge of Culvert #1. This is supported by the small increases observed in stage and discharge at *cross section -6*.

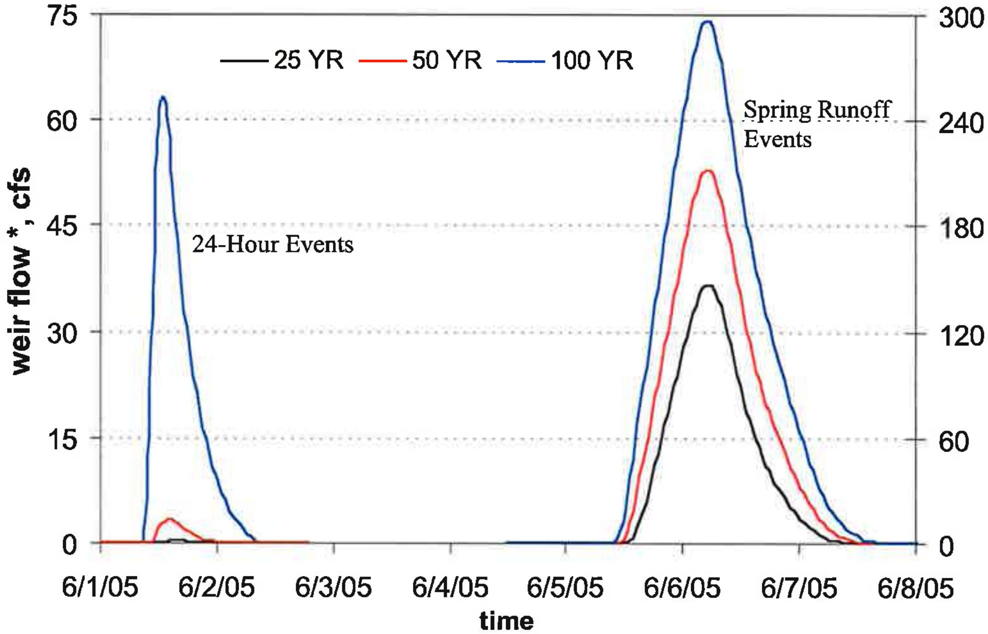

Figure 15. Total weir flow over E-W township road. Axis with * corresponds with 24 hour storm events.

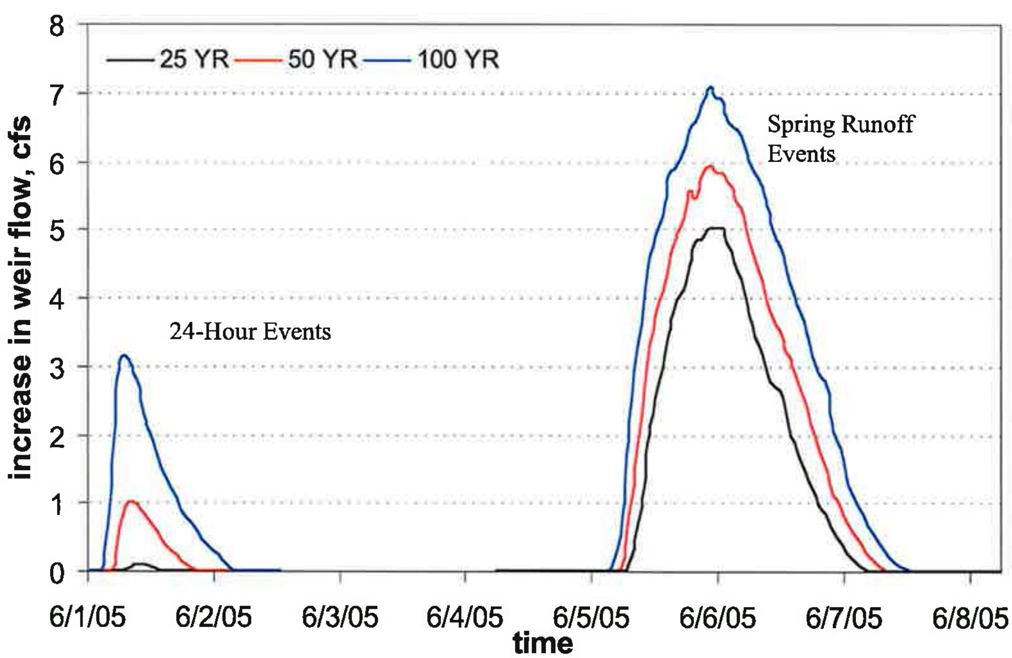

Figure 16. Increase in discharge to Snowflake creek by weir flow over the E-W township road caused by removing Culvert #1.

6.2. Effects of Raising E-W Township Road

As shown in *Sec. 6.1*, overtopping of the E-W township road is predicted for 25-year, 50-year, and 100-year 24-hour and spring runoff events. In these events, it is plausible for carp migration over the submerged E-W township road. Therefore it is of interest to investigate the downstream effects caused by raising the E-W township road and completely stopping flow into Snowflake Creek. In Table 4 the peak discharge and total volume discharged at *cross section -6* for the existing conditions and condition with the E-W township road raised. The relative effects on discharge, stage, top width, and volume discharged are discussed in the proceeding sections.

Table 4. Peak discharge and total volume discharged for events under existing conditions and with the E-W township road raised.

		Existing Conditions		E-W Township Road Raised	
Event		**Volume Discharged, acre-ft**	**Peak Discharge, cfs**	**Volume Discharged, acre-ft**	**Peak Discharge, cfs**
24-hour	25 year	1080	384.82	1084	388.85
	50 year	1389	517.60	1404	532.76
	100 year	1774	668.46	1831	722.14
10-day	25 year	4635	819.00	4887	971.40
	50 year	5347	912.54	5743	1130.61
	100 year	6124	1020.38	6721	1326.79

6.2.1. Effect on Discharge

The relative percent increase in discharge at *cross section -6* resulting from raising the E-W township road and removing Culvert #1 is illustrated in Figure 17. The 24-hour storm events yield an increase in discharge of less than 10% for the 25-year, 50-year and 100-year events. However during the spring runoff events the discharge has increased by 17%-30% for the 25-year, 50-year, and 100-year events. These increases may not necessarily be negligible, and may have effects on the area of inundation downstream.

6.2.2. Effect on Stage

The time dependent increase in stage for the 24-hour and spring runoff events is given in Figure 18. By comparison with Figure 11, raising the road results in an increase in stage at *cross section -6*. For the 24-hour storm events the greatest increase in stage is one inch and for the spring runoff events the greatest increase is almost 4.5 inches. At peak flows, the increase in top width of flow area is summarized in Table 5 along with the percent increase, which shows the differences may not be negligible in large events. The largest increase in top width is the 100 year 24-hour event. This large increase is essentially a result of the two separate stream channels shown in Figure 3 overflowing and becoming one channel. Also noted is that these increases in stage are short lived as can be inferred from Figure 18.

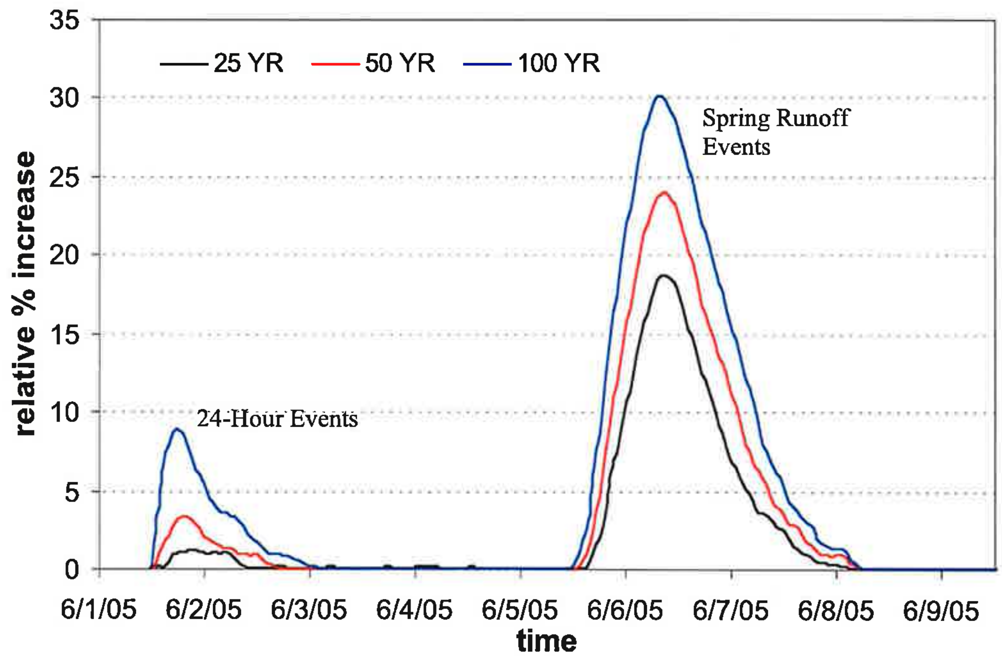

Figure 17. Relative percent increase in discharge resulting from removal of Culvert #1 and raising the E-W township road.

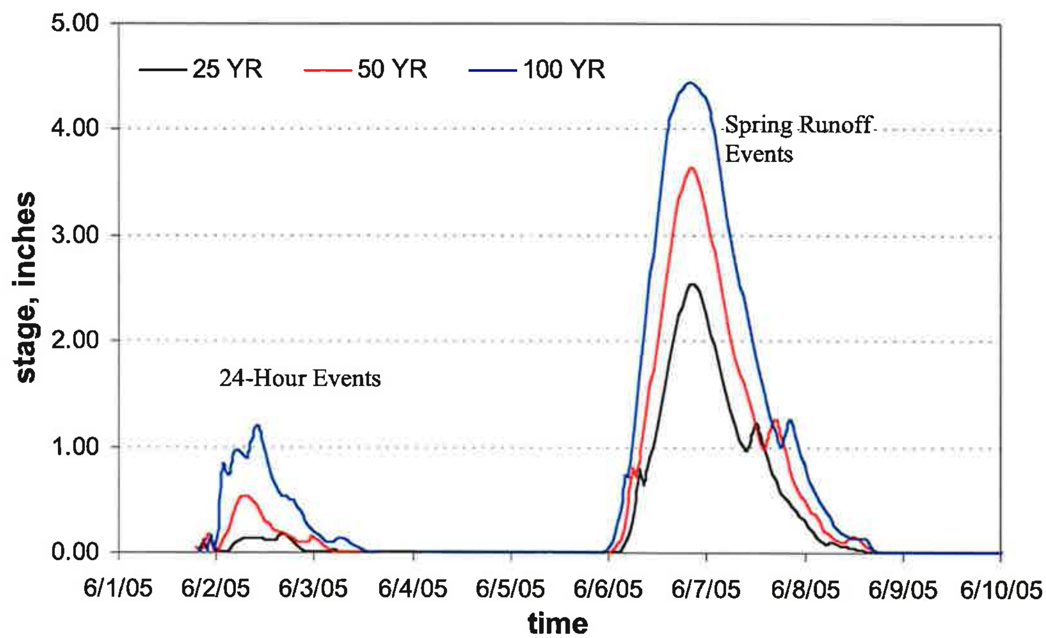

Figure 18. Increase in stage as a result of removing Culvert #1 and raising the E-W township road.

Table 5. Increase in top width at *cross section -6* with E-W township road raised and Culvert #1 removed.

Event	Increase in Top Width (ft)	% increase
24 hour 25 year	1.11	0.40
24 hour 50 year	4.04	1.29
24 hour 100 year	38.85	8.81
10 day 25 year	6.31	1.10
10 day 50 year	9.01	1.56
10 day 100 year	11.04	1.89

6.2.3. Effect on Total Volume Discharged

The increase in the total volume discharged and percent increase in total volume discharged for the various events are summarized in Table 6. The increase in volume discharge is under 4% for the 24-hour events. However ranges from 5-10% for the spring runoff events.

Table 6. Increase in volume discharged at *cross section -6* with E-W township road raised and Culvert #1 removed.

Event	Increase in Volume Discharged (acre-ft)	% increase
24 hour 25 year	4.36	0.40
24 hour 50 year	15.45	1.11
24 hour 100 year	57.57	3.25
10 day 25 year	252.83	5.46
10 day 50 year	397.00	7.43
10 day 100 year	597.73	9.76

6.3. Effects on Pembina River and Devils Lake

As mentioned before, the immediate effects on the area of inundation caused by the removal of Culvert #1 will not be significant in the case of 24-hour storms and spring runoff events (see *Sec. 6.1* and *6.2*). From *Sec. 6.2*, the results of raising the E-W township road showed more pronounced effects on the area of inundation, but were relatively short lived. However, long term effects caused from removing Culvert #1 and/or raising the E-W township road should also be addressed. Therefore let us consider a 100-year span. During this 100-year span, the study area in Figure 2 is subject to the following 24-hour storm events:

- Twenty 5-year events
- Ten 10-year events
- Four 25-year events
- Two 50-year events
- One 100-year event

Also the area is subject to the following ten day spring runoff events:

- ∞ Four 25-year events
- ∞ Two 50-year events
- ∞ One 100- year event

From *Sec. 6.1.3*, the 25-year, 50-year, and 100-year events need only be considered. Utilizing the values reported in *Sec. 6.1.3*, the total volume of water diverted from the Pembina River to Devils Lake 100 years after removing Culvert #1 is 245.39 acre-ft. From *Sec. 6.2.3* the total amount of water by raising the E-W township road and removing Culvert #1 is 2508.96 acre-ft. Assuming 100 years after the removal of Culvert #1 the Devils Lake level is 1390.1 ft amsl (10 ft below the record low). The lake level would have increased by 0.2 ft over the 100 year period as a result of the removal of Culvert #1. At the current lake level or approximately 1448 ft amsl the increase would be 0.0019 ft. Utilizing *Sec. 6.3* by raising the E-W township road and removing Culvert #1 Devils Lake would have increased by 1.9 ft from a level of 1390.1 ft amsl, and would increase by 0.019 ft at the current level of 1448 ft amsl.

As for the Pembina River, assuming a 100-year span from 1904-2004, the total volume discharged by the river without any changes utilizing mean annual discharges is 15,258,054 acre-ft. The impact of removing 245.39 acre-ft due to storm events and spring runoff events results in a relative reduction of 0.0016%. As for raising the E-W township road and removing Culvert #1 the relative reduction in volume discharged by removing 2508.96 acre-ft is 0.016%.

7. CONCLUSIONS AND RECOMMENDATIONS

To reduce the risk of introducing common carp to Devils Lake, it is recommended that Culvert #1 be removed and the E-W township road be raised to an elevation of 1582 feet-msl to form an earthen barrier (See *Sec. 3-2*). This appears to be the most reliable and economically feasible option, and requires less maintenance than an electrical barrier or screens. Implementing a Gator Pump with the barrier would be equally effective, but would increase construction costs and require significant operations and maintenance costs. In addition, construction of a detention basin and potential erosion from the outfall would alter the landscape more than by simply raising the township road. The cost of this alternative is $55,400.

Based on the results of the numerical transient one dimensional model, it appears that the removal of Culvert #1 will have minor impacts on the downstream conditions. In the case of 24-hour 5- and 10-year storm events, removing Culvert #1 yields no difference in the downstream conditions. In the case of 25-year, 50-year, and 100-year events, the diverted flow resulting from removing Culvert #1 caused small impacts on the Devils Lake and Pembina River Basins. More significant impacts resulted from raising the E-W township road. The total volume of water diverted, and the resulting effects on Devils Lake and Pembina River over a 100-year span increased by approximately 10 times from raising the E-W township road (an average of 25 acre-ft/year diverted to Devils Lake). The probability of the road being overtopped is slightly greater than 1%.

It must be noted that this model is based on available data, and that due to lack of flow records in the area studied, it is not possible to calibrate the model. Availability of data from the area in Figure 2 may help further improve the model.

8. REFERENCES

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Soil Conservation Service. (1972). National *Engineering Handbook, section 4, Hydrology*. U.S. Dept. of Agriculture, available from U.S. Government Printing Office, Washington, D.C.