MAPLE CREEK

(BIGSTICK LAKE) WATERSHED

JUNE, 2010 FLOOD

Prepared For: Saskatchewan Watershed Authority
Agriculture and Agri-Food Canada

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EXECUTIVE SUMMARY

Wet weather through April and May of 2010 combined with a moderately severe rainstorm to produce record flooding in the Maple Creek area on June 18 and 19. Severe flood damages in the Town of Maple Creek, the surrounding rural area and Highway 1 resulted. The Saskatchewan Watershed Authority (SWA) and Agri-Environment Services Branch (AESB) of Agriculture and Agri-Food Canada jointly commissioned this report on the flood event in order to:

- Document meteorologic, hydrologic and hydraulic conditions;
- Document operation of the AESB reservoirs;
- Review forecasting and warning;
- Describe the magnitude of damages;
- Identify potential management and structural options to mitigate future flood damages.

The flood damages were concentrated north of the relatively steep north slope of the Cypress Hills. Flood water from the hills tends to spread out onto the flat terrain just north of the hills.

Shortage of water for irrigation is a common problem in this region and water developments have concentrated on development of reservoirs to store water from wet periods for use in subsequent dry periods.

The spring runoff in 2010 was sufficient to fill all the water storage reservoirs in the area. April and May precipitation was unusually heavy resulting in very wet conditions. May was a relatively cool month so evaporation was low. A moderate rainstorm on June 16 replenished the wet conditions. Over night on June 17 to June 18, just over 100 mm of rain fell on the Maple Creek drainage basin with some locations receiving more and some less. The rainfall frequency analysis indicates that the average was close to a 1:100 year event with some areas possibly as high as 1:200 years. Combined with the very wet antecedent conditions, this event produced a flood peak that exceeded the previous highest flood by a factor of more than four.

There are no hydrometric stations in this drainage basin. During the flood, AESB monitored the level of Junction Reservoir north of Maple Creek and calculated the inflow. Runoff to Junction Reservoir originated from Gap Creek and Maple Creek upstream of the Town of Maple Creek with Gap Creek being the larger of the two. The peak inflow to Junction Reservoir occurred near midnight on June 18/19 at 408 m^3/s. Since the Maple Creek tributary is adjacent to the Town and causes the Town's flood problems, it was necessary to calculate what portion of the peak flow came from this creek. Based on a detailed hydrologic analysis, the peak flow on Maple Creek at the Town was estimated to have been 100 m^3/s, plus or minus 10 m^3/s. The corresponding Gap Creek peak was estimated to have been 315 m^3/s. Because of timing factors, the peaks do not occur at exactly the same time so the sum of the two peaks does not equal the total peak.

The hydrologic factors used to segregate the flows for 2010 were then applied to data from a similar but smaller flood in 1998 to confirm that the method used would also work for other flood events.

Based on frequency analysis of flood peak flows from past studies, the 2010 flood is estimated to have had a peak flow equal to the 1:3700 year frequency. The flood volume was not as extreme at about 1:250 years.

Although it is very unlikely to be repeated in any one year, this was not the largest flood that could possibly occur. Analysis of the probable maximum flood (PMF) potential indicates that the 2010 flood peak was 60 percent and the volume was 12 percent of the PMF.

In 2009, the Town of Maple Creek commissioned a Maple Creek Floodplain Delineation Study by Schaffer Andrews Ltd. and Sameng Inc. As part of the study, the creek was surveyed and a hydraulic model was developed based on the U.S. Army Corps of Engineers HEC-RAS computer model. Sameng Inc. provided the input file for their model to assist with the analysis of the 2010 flood.

The 1:500 year flood flow of 70.5 m^3/s was tested to confirm the Sameng results. The estimated 2010 flood peak of 100 m^3/s was found to cause flood levels 0.4 m to 0.9 m higher than the 70.5 m^3/s flow. The modelled levels were high compared to the observed flood levels in the north part of Town and very conservative for the south part of Town. It was noted that the model only included the surveyed creek and a small part of the flood plain. In a major flood, the flood plain would carry more flow than this model provides for. SWA has a topographic map of the creek and Town based on the Flood Damage Reduction Program that could be used to extend the model cross sections so the model would more completely model extreme floods.

The model calculations indicate that the water levels are strongly influenced by the bridges, and particularly the Second Avenue Bridge which can cause substantial backwater.

Model calculations demonstrated that the level of Junction Reservoir does not influence flooding in Town.

Stage-discharge data for locations through the Town were estimated but it is noted that estimates are conservative in extreme floods and should be refined before final planning is undertaken.

Various potential flood mitigation strategies were considered. Major diversions upstream of Town are not practical. Upstream storage reservoirs would have to be very large and are not practical. Channel improvements must be very large to significantly change flood levels. A better crossing at Second Avenue could reduce flood levels significantly in the local area upstream of the bridge. Dyking is the most practical structural option to protect the Town. If future developments are allowed in the flood hazard area, they should be flood proofed. Zoning controls should be established to prevent further expansion to the flood damages.

A review of the operation of the AESB control works upstream of Maple Creek indicates that these works cannot prevent floods. The operation in 2010 did as much as is practical but only lowered flood levels by an estimated 0.1 m. These works were designed for water supply and are located off the main creeks so they cannot provide significant flood control. Junction Reservoir is on the main creek downstream of Town and it does significantly reduce the flood potential for facilities like Highway1 farther north.

It was not possible to complete a detailed review of damages because much repair work remains to be completed. It was noted that the flood caused widespread damage to public and private infrastructure. No loss of life or serious injury occurred. About one-third of the Town of Maple Creek was flooded with the west side flooding deep enough to cause sewer back up. Highway # 1 was washed out and closed for one week then functioned at reduced capacity for 6 months and will have to be partly closed again in 2011 to finish repairs. Total cost for this highway will be in the order of $10 million.

Forecasting of runoff from snow melt can be provided to some extent but forecasting summer rain floods is not possible based on weather data. Some small warning might be gained if real time water levels from a hydrometric station in the head waters area were available. At present there are no active hydrometric stations in this drainage basin.

RECOMMENDATIONS

1. The hydraulic model should be updated to reflect levels observed in 2010.

2. Dyking is the only practical option to protect existing developments in Maple Creek from flood damages. Very high design standards should be applied. Inspection and maintenance are essential to the long term safety of dykes. The detailed design must consider the tendency of water from the creek south of Town upstream of Highway 271 moving east into areas that would not flood directly from the creek on the west side of Town.

3. Further investigation of the possibility of reducing the backwater at Second Avenue by installing some type of low level crossing is needed.

4. Flood proofing should be required for any future developments or redevelopment of the flood plain.

5. Zoning should be established to ensure that damage potential does not expand in the future.

6. Developments should not encroach too far into the flood plain, obstructing flood flows.

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1.0 INTRODUCTION

1.1 REPORT PURPOSE

Wet weather through April and May of 2010 combined with a heavy rainfall from June 16 to 18 resulted in an extreme runoff event in Southwest Saskatchewan. Runoff on Gap Creek and Maple Creek was particularly high and severe flood damages occurred in the Town of Maple Creek, the surrounding rural area and Highway 1 in the Bigstick Lake Drainage Basin.

The Agri-Environment Services Branch (AESB) of Agriculture and Agri-Food Canada owns and operates four water supply reservoirs and a network of irrigation canals in the drainage basin. The works associated with these reservoirs were damaged by this flood. The Saskatchewan Watershed Authority (SWA) manages the provincial water responsibilities. AESB and SWA assembled various components of the background to this flood event during the summer of 2010. Water Resource Consultants Ltd. was commissioned to assemble their information and information from other sources into this report. The cost of the study was shared equally by AESB and SWA.

The Terms of Reference for the study are attached as Appendix A to this report and are summarized below:

- Document meteorologic, hydrologic and hydraulic conditions;
- Document operation of the AESB reservoirs;
- Review forecasting and warning;
- Describe the magnitude of damages;
- Identify potential management and structural options to mitigate future flood damages.

1.2 GEOGRAPHIC SETTING

As shown on Figure 1.1, Maple Creek and Gap Creek drain the portion of the north slope of the Cypress Hills south of the Town of Maple Creek. The two creeks join at Junction Reservoir north of the Town and continue north as Maple Creek. North of Junction Reservoir the creek crosses Highway and flows into Bigstick Lake. The total drainage basin tributary to Bigstick Lake is 7,600 km² but less than one quarter, 1,284 km² is effective in contributing runoff to the lake in a normal year. The portion of the drainage basin north of Highway 1 is relatively flat with large areas of depressions which capture runoff and return it to the atmosphere through evaporation in this semi-arid region. Bigstick Lake is the terminus of this closed drainage basin.

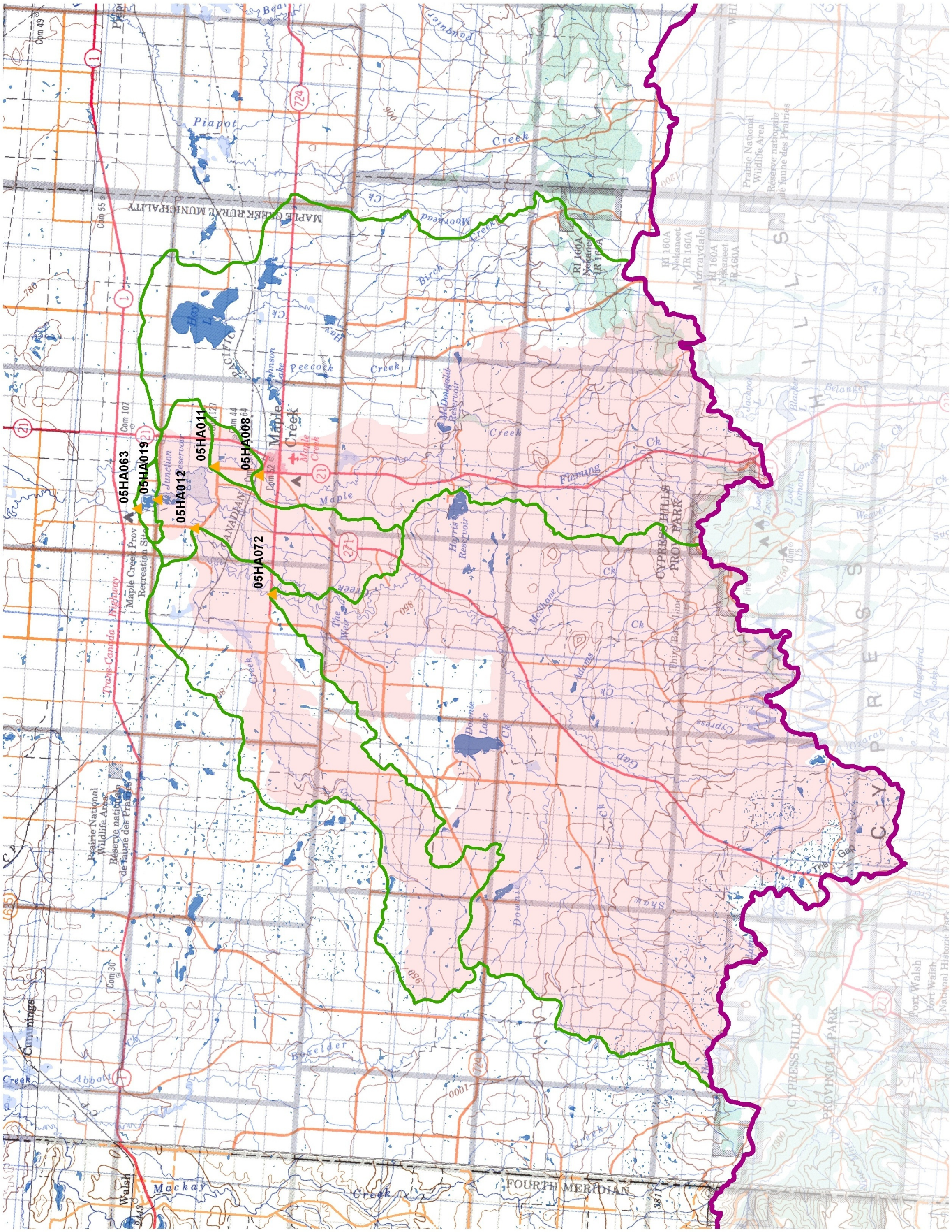

Figure 1.1: Location Plan with Drainage Areas

The majority of the runoff in the basin arises in the 1,235 km² of gross drainage area upstream of Junction Reservoir. Of this gross area, 915 km² is effective in normal runoff events. Gap Creek is the larger tributary with 729 km² of gross area and 646 km² of effective drainage area. Maple Creek drains the remaining 506 km² of gross area and 269 km² of effective drainage area. Most of the ineffective area within the Maple Creek portion of the basin is the Hay Lake drainage area which never contributes to Maple Creek, even in an extreme event like the 2010 flood. Based on the effective drainage areas Gap Creek makes up 76 percent of the runoff potential and Maple Creek has 24 percent of the runoff potential south of Junction Reservoir.

The north slope of the Cypress Hills is the portion of the drainage basin with the highest runoff potential. The land slopes from elevations over 1,200 m at the top of the hills to below 800 m in the Junction Reservoir area in a distance of 35 km. North of Junction Reservoir the terrain is much flatter with a drop of only 25 m in a similar distance from Junction Reservoir to Bigstick Lake.

The town of Maple Creek is located on the east side of Maple Creek about 3 km upstream of Junction Reservoir in the area where the topography transitions from the slope of the Cypress Hills to the flat plain to the north.

1.3 BASIN WATER DEVELOPMENTS

This region has a semi-arid climate with water supply as a major limitation on agricultural production. As farming developed in the late 1800's and early 1900's, the potential to enhance production through irrigation, using the relatively reliable flows from the Cypress Hills, was recognized. Some of the earliest irrigation developments in Saskatchewan occurred in the Bigstick Lake Drainage Basin.

The early projects were generally based on the flow of the creek within each year and production was extremely variable as the flows varied. In order to stabilize the irrigation water supply, Canada developed four reservoirs in the drainage basin.

The largest is Junction Reservoir which was created by a dam on Maple Creek about 8 km downstream of the Town. This reservoir is about 11 m deep; has an area of 494.2 ha; and a volume of 12,933 dam³ at its Full Supply Level (FSL) of 757.28 m. This reservoir provides water for agricultural uses.

The second largest reservoir is Downie Lake which is located in the Gap Creek drainage basin. Downie Lake receives runoff from a small local drainage area of 25 km² but its main source of supply is a diversion channel from Downie Creek and Gap Creek. The reservoir is about 7 m deep; 260 ha in area; and has a volume of 12,223 dam³ at its FSL of 878.89 m. An outlet canal permits releases to Downie Creek which allows delivery of water to Gap Creek and irrigated areas near the Town.

The third reservoir is Harris Reservoir which is located beside Fleming Creek. This reservoir has a local drainage area of 14 km^2 but its main supply is from a diversion channel from Fleming Creek. The reservoir is about 9.5 m deep; 131 ha in area and has a volume of 6,047 dam^3 at its FSL of 857.36 m. Water can be released through two canals. One leads west to Gap Creek and the other east to Fleming Creek which leads to Maple Creek. The canals permit delivery of water to the irrigated area near Town from either Gap Creek or Maple Creek.

The fourth reservoir is McDougald Reservoir which is located beside Maple Creek. This reservoir has a local drainage area of about 2 km^2 but its main supply is from Maple Creek by a diversion canal. The reservoir is about 4 m deep; 21 ha in area; and has a volume of 931 dam^3 at FSL of 844.92 m. Water can be released through a canal to Maple Creek to supply irrigation areas near Town.

Downie Lake, Harris Reservoir and McDougald Reservoir are operated along with a network of irrigation supply canals and drains to serve the Maple Creek Irrigation Project west of the Town. The location of the reservoirs is shown on Figure 1.1. Area and capacity curves for the reservoirs and a general plan of the irrigation project are attached in Appendix B.

These reservoirs are owned and operated by AESB.

In addition to this system of reservoirs and irrigation canals there are many small individual irrigation, stock water and domestic water projects.

2.0 METEOROLOGY AND HYDROLOGY

2.1 METEOROLOGY

A major storm system dropped up to 140 mm of precipitation over parts of the drainage basin between June 16 and June 18, 2010. Most of the precipitation fell between 18:00 June 17 and 17:50 June 18. The rainfall, which averaged 107.7 mm over the effective area of Maple Creek above Junction Reservoir, resulted in unprecedented inflow into the reservoir. The runoff from the rainstorm caused severe damage to transportation and irrigation infrastructure within the basin and produced record flooding at the town of Maple Creek.

The flood peak at Junction Reservoir, located downstream of the town of Maple Creek, was more than four times the largest previous flood on record. The previous record summer flood occurred in June 1998 following a heavy rainstorm on June 27th. The difference in magnitude of the 2010 and 1998 flood peaks is more than can be explained by differences in rainfall. Weather conditions leading up these events were quite different.

Figure 2.1 shows that the 1998 and 2010 precipitation events are of a similar order of magnitude. It could be expected that they would produce similar runoff.

This section describes the rainstorm event that produced the 2010 flood, compares the 2010 and 1998 precipitation, and examines the antecedent conditions that existed prior to the rainstorm. These were prepared to provide an overall understanding of the storm and the conditions that produced the June 2010 flood.

2.1.1 Data Sources

Information for the June 2010 storm was obtained from:

1) Cypress Hills Park, Cypress Hills, Maple Creek North and Maple Creek meteorological stations operated by the Meteorological Services Branch of Environment Canada;

2) precipitation estimates from an analysis of radar images by Ron Hopkinson, Custom Climate Services, using the radar station at Schuler, Alberta; and

3) analyses of rainfall data by AESB.

Data for the analysis of historic precipitation and temperatures was obtained from the Environment Canada stations listed in Table 2.1. The Maple Creek and Cypress Hills Park data were the primary data sets. Both stations have numerous gaps in the record. The location of each station has changed at least once during the period of record as shown by the two entries for each station in Table 2.1. Although rainfall can vary with location it was

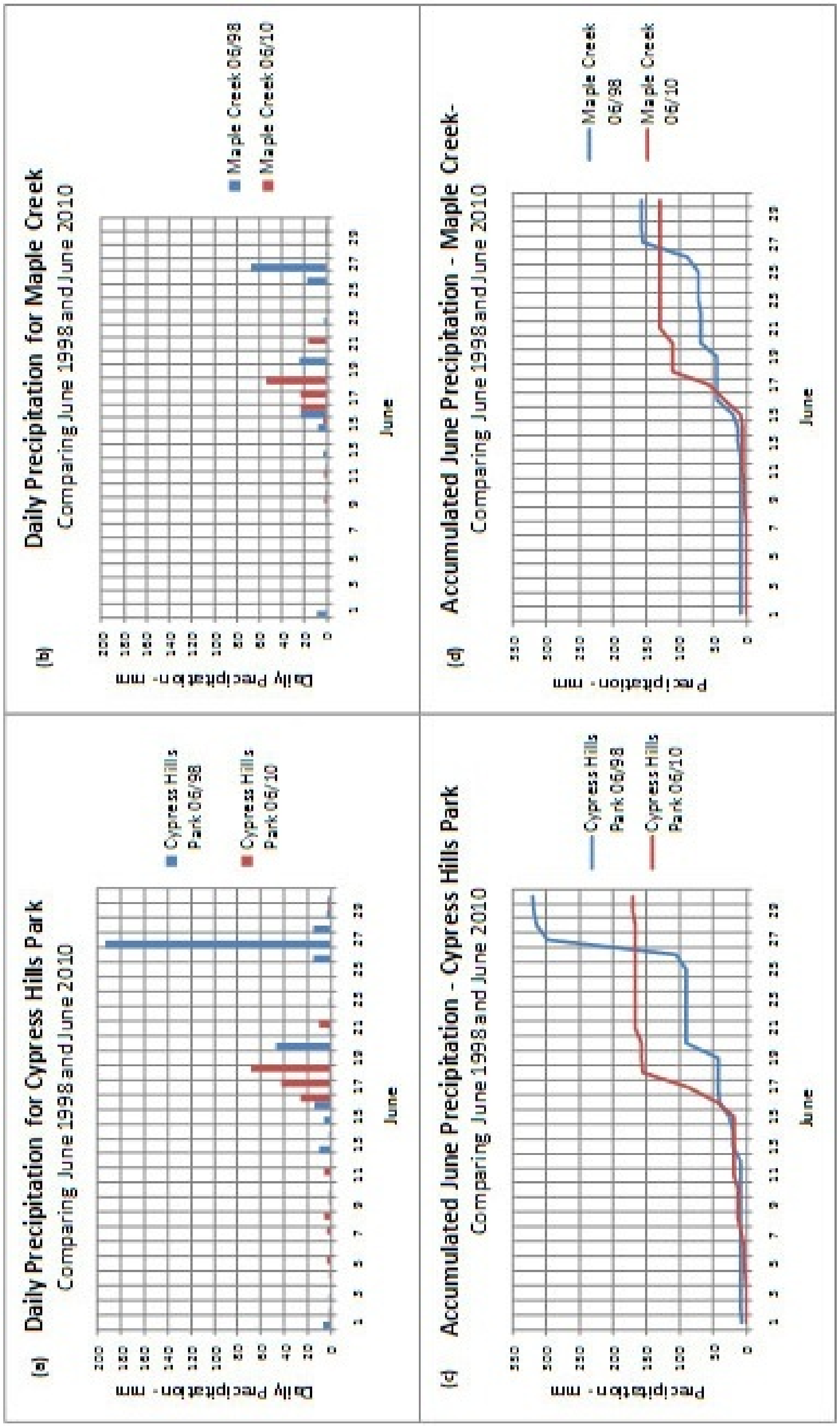

Figure 2.1: Comparison of June 2010 and 1998 Storm Precipitation

assumed for the purposes of this study, that the records for both locations could be viewed as a single record.

TABLE 2.1 ENVIRONMENT CANADA METEOROLOGICAL STATIONS IN THE VICINITY OF THE BIGSTICK LAKE BASIN						
Station Name	Latitude	Longitude	Climate ID	WMO ID	Elevation	Period of Record
Maple Creek	49° 54' 9.05" N	109° 27' 58.06" W	4024919	71453	766.7 m	1994-2010
Maple Creek	49° 54' 0.00" N	109° 29' 00.0" W	4024920		763.5 m	1884-1982
Maple Creek North	50° 00' 0.0" N	109° 28' 00.0" W	4024936		764.1 m	1951-2008
Cypress Hills Park	49° 38' 26.009" N	109° 30' 52.004" W	4031999	71139	1270.6 m	1992-2010
Cypress Hills Park	49° 40' 0.00" N	109° 29' 0.00" W	4032000		1371.6 m	1918-1972
Cypress Hills	49° 40' 0.00" N	109° 28' 0.00" N	4021996		1196.0 m	1981-2010

The Cypress Hills and Maple Creek North stations provided information on climate normals that was not available for the primary stations. Data for these stations were obtained from the National Climate Data Archive managed by Meteorological Services Canada.

Ron Hopkinson, Custom Climate Services completed a detailed assessment of the regional and local meteorology associated with the storm. This is documented in the report "Analysis of June 15-18 Cypress Hills Storm over Southwest Saskatchewan" September 20, 2010. The information in this report was used extensively as a reference in the current study to describe the storm. An earlier study entitled "An overview of 2010 early summer severe weather events in Saskatchewan", Ron Hopkinson, Custom Climate Services Inc., July 30, 2010, and an e-mail report by Hopkinson to Doug Johnson, Saskatchewan Watershed Authority, August 19, 2010, also provided information that was used in the initial stages of this study.

2.1.2 Description of June 17-18, 2010 Storm

After a moderate rain on June 16, the major rainstorm began at 18:00 June 17 and lasted approximately 24 hours until 17:50 June 18. The storm covered the entire basin for this duration as illustrated by the radar maps for Schuler, Alberta shown on Figure 2.2. This Figure shows that the Maple Creek storm was part of a larger storm system that extended from Medicine Hat to Swift Current. The storm covered both the north and south slopes of the Cypress Hills. Hopkinson describes the storm as "a synoptic scale event associated with a well developed low pressure system". According to Hopkinson, the rainstorm that fell on the Bigstick Lake Basin was "highest along the north slope of the Cypress Hills where upslope conditions associated with strong northerly winds enhanced the rainfall." The heaviest rain fell during the early evening of June 17 and the early morning hours of June 18. The maximum intensity in the upper basin was 10 mm/hr at the Cypress Hills Park station while a spike of 18 mm/hr occurred at Maple Creek in the early morning of June 18.

The National Agroclimate Information Services (NAIS) of AESB provides another regional view of the storm on Figure 2.3 showing precipitation accumulations over southwestern Saskatchewan and southern Alberta for June 17-18, 2010 based on data from meteorologic stations.

The analysis of radar by Hopkinson indicated that accumulated precipitation of up to 140 mm occurred in the basin. Figure 2.4 shows the detailed analysis of radar imagery for June 18. Rainfall accumulations for various locations over the region affected by the storm including Cypress Hills and Maple Creek are shown on Figure 2.5. The analysis shows that about 100 mm fell during the 24 hour period. This was preceded by nearly 40 mm on the two previous days.

Additional assessments of the rainfall were made by Rick Rickwood and Trevor Hadwen of AESB. Their information indicated that the basin average rainfall was 93.4 mm for Maple Creek at the Town of Maple Creek, and 102.6 mm for the area contributing to Junction Reservoir. According to Hadwen the basin rainfall varied from 75.4 to 135.5 mm. This is consistent with Hopkinson's findings.

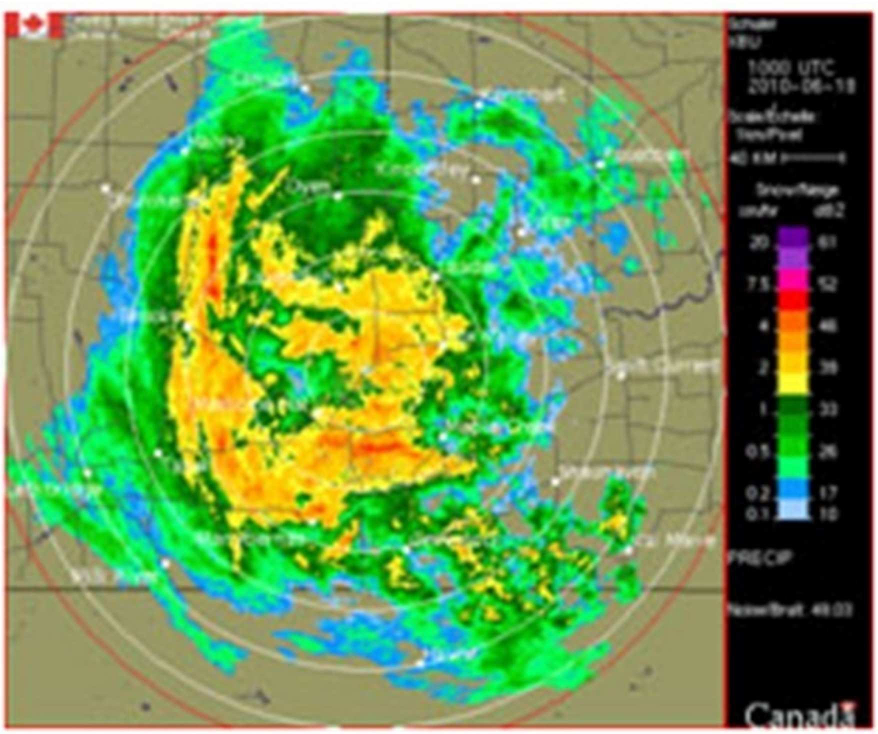

Figure 2.2: Radar Image of June 2010 Rainstorm, Hopkinson

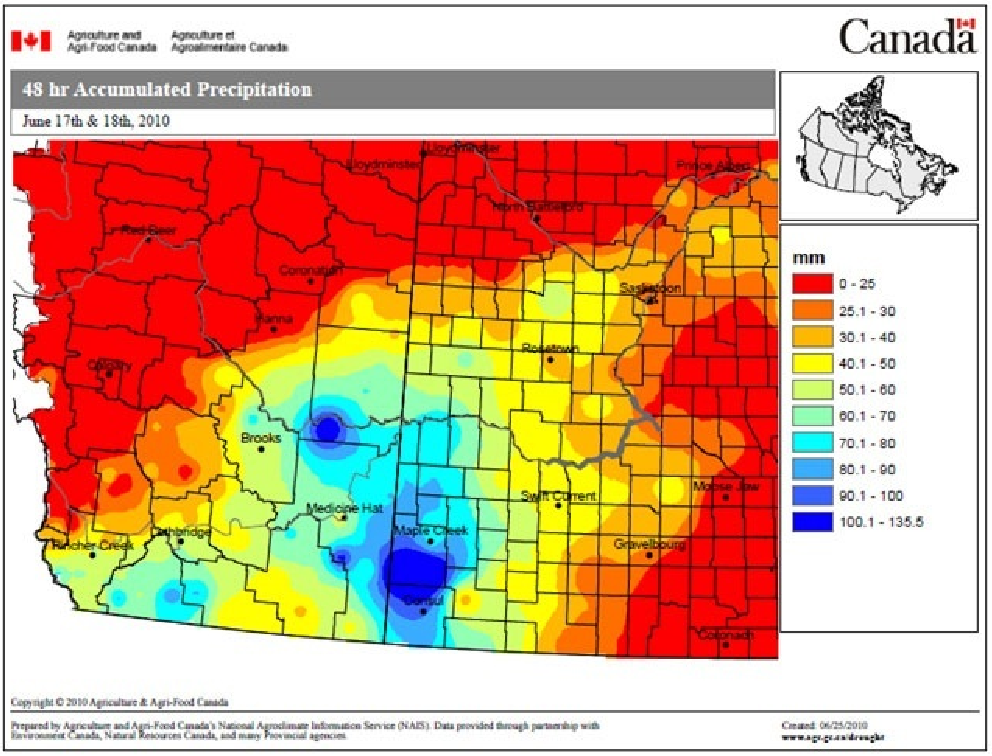

Figure 2.3: 48-Hour Accumulated Rainfall, AESB

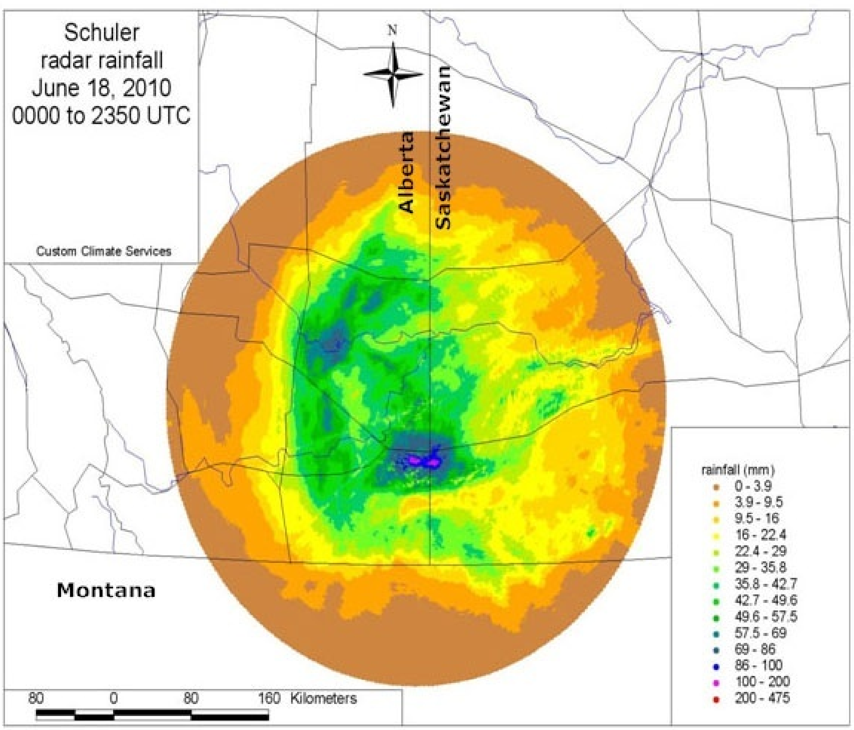

Figure 2.4: Precipitation from Radar June 18 0:00 to 23:00 MDT, Hopkinson

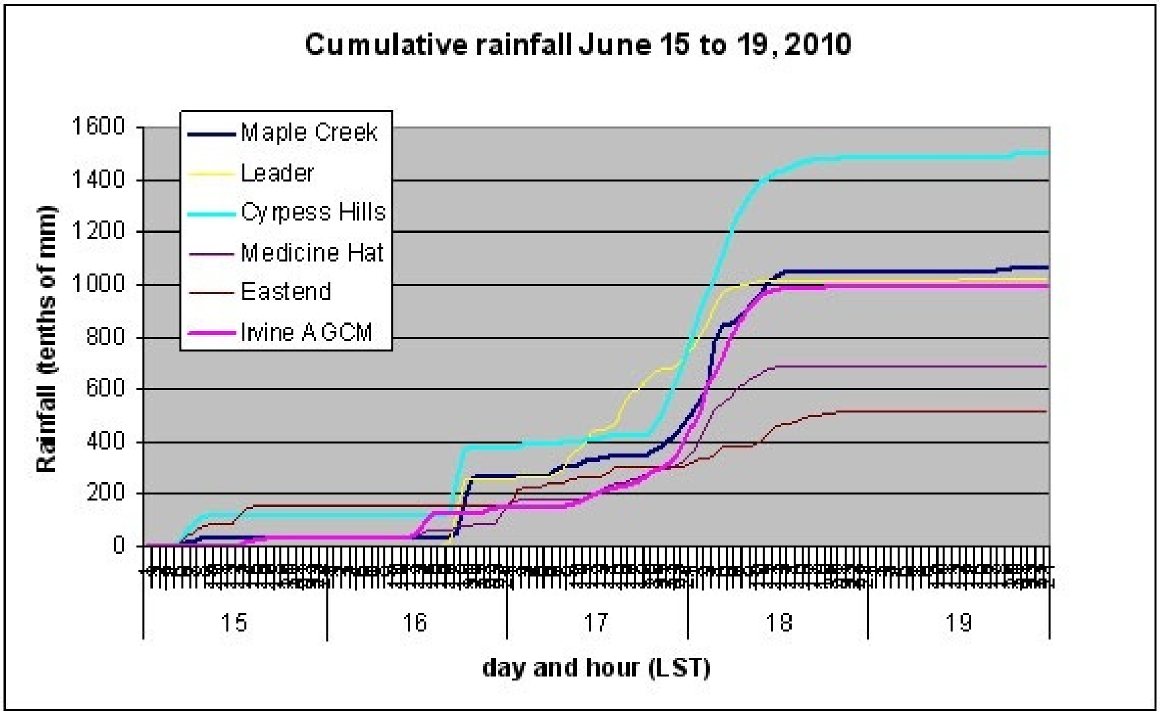

Figure 2.5: Accumulated Rainfall June 15-19, 2010, Hopkinson

Plots of hourly precipitation for Maple Creek and Cypress Hills Park meteorological stations prepared by AESB are shown on Figures 2.6 and 2.7. Figure 2.6 shows that rain fell continuously over the 23 hour period from 18:00 June 17 to 17:00 June 18 at the Cypress Hills Park station in the upper reaches of the basin reaching the maximum intensity of 10 mm/hr at about 01:00 June 18. It is worth noting that at the height of the storm the rainfall intensity exceeded 6 mm/hr for 9 hours within a 10 hour period. The rainfall at Maple Creek shown on Figure 2.7 was of similar duration but less intense with the exception of a short intense rainfall of 18mm/hr at 03:00 June 18. Environment Canada's daily rainfall totals for Cypress Hills Park, Cypress Hills and Maple Creek are shown in Table 2.2. These data were obtained from Canada's National Climate Archive.

The Rainfall Frequency Atlas for Canada (Environment Canada, 1985) provides a method for estimating the rainfall for various return periods. A 1:100 year 24-hour storm would range between 91 and 97 mm in this region. The 2010 peak 24-hour precipitation of 105 mm for Cypress Hills Park exceeded the 1:100 year value and based on relationships provided in the Rainfall Frequency Atlas was close to the 1:200 year return period. The Maple Creek 24 hour precipitation of 70 mm was less extreme at about the 1:20 year return period. The Rainfall Frequency Atlas does not fully reflect the increased rainfall potential due the topographic relief of the Cypress Hills. Therefore the true frequency of this storm was likely less than the 1:200 year value calculated.

TABLE 2.2 DAILY PRECIPITATION AT SELECTED LOCATIONS IN THE MAPLE CREEK BASIN			
	Daily Precipitation - mm		
	Cypress Hills Park	Maple Creek	Cypress Hills
June 16	28.2	23.5	25.5
June 17	82.2	54.2	42.3
June 18	16.6	23.7	68.6
3 day total	127.0	101.4	136.4

The maximum return period for the June 2010 24-hour precipitation is estimated to be less than 200 years. This is more than an order of magnitude less than the return period for the flood peak discussed later. Also, the rainfall in other parts of the basin were not as extreme, indicating that factors other than this rain storm influenced the flood.

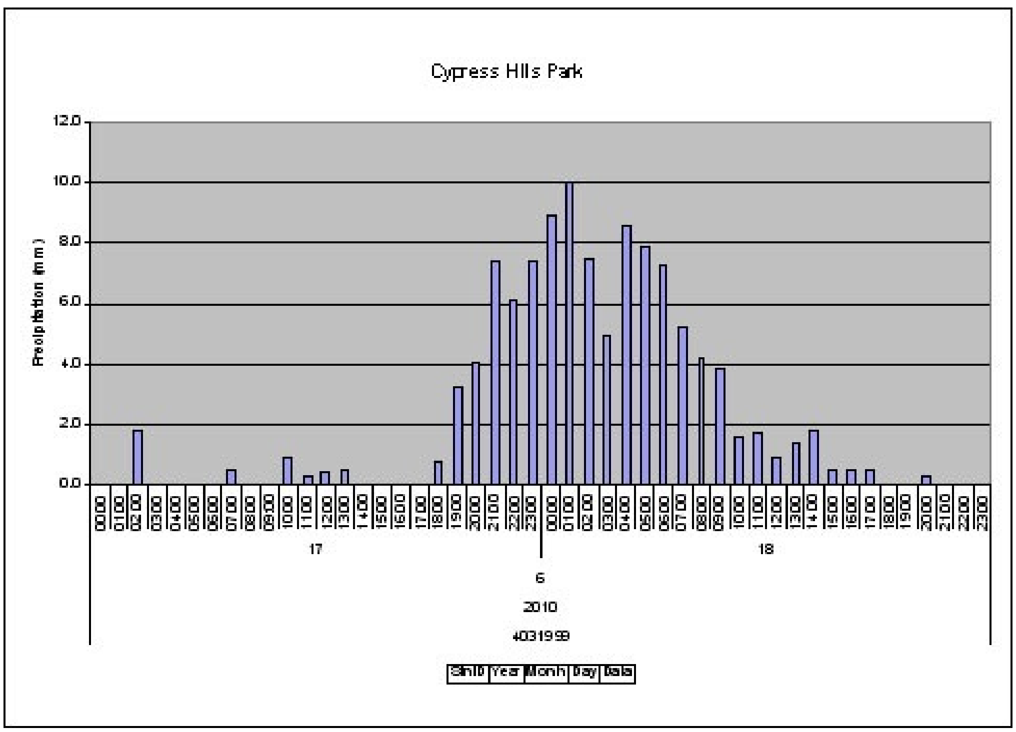

Figure 2.6: Hourly Rainfall Cypress Hills Park June 17-18, 2010, AESB

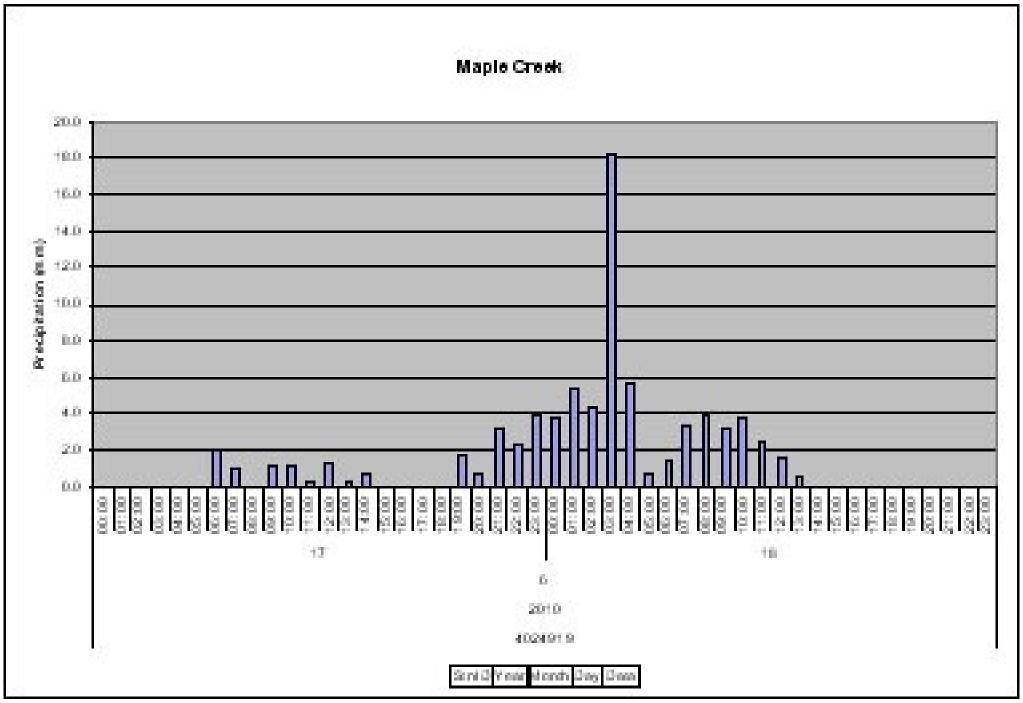

Figure 2.7: Hourly Rainfall Maple Creek June 17-18, 2010, AESB

Ron Hopkinson of Environment Canada, in a report dated February 23, 1983, conducted a probable maximum precipitation (PMP) study for Junction Dam by maximizing and transposing historic storms to the basin. The basin average PMP was estimated to be 444 mm. This is a little more than four times the basin average for the June, 2010 storm (104 mm) as estimated by AESB. Therefore it can be noted that the June, 2010 storm was not the upper limit of potential rainfall in this region.

From this it can be concluded that the 2010 rainfall was a rare event for the Cypress Hills, and well above normal for Maple Creek but not the worst that can occur.

2.1.3 Antecedent Conditions

Figures 2.8 and 2.9 indicate that the fall precipitation in 2009 was a little above normal. Figures 2.10 and 2.11 suggest that winter precipitation in 2009-2010 was below normal. However, it has been determined by AESB that these gauges substantially under captured the snowfall and are not a reliable indication of the snow pack in 2010. A better indication is provided by Figure 2.12 which shows that the Cypress Hills area had about double the normal winter snow pack on March 1.

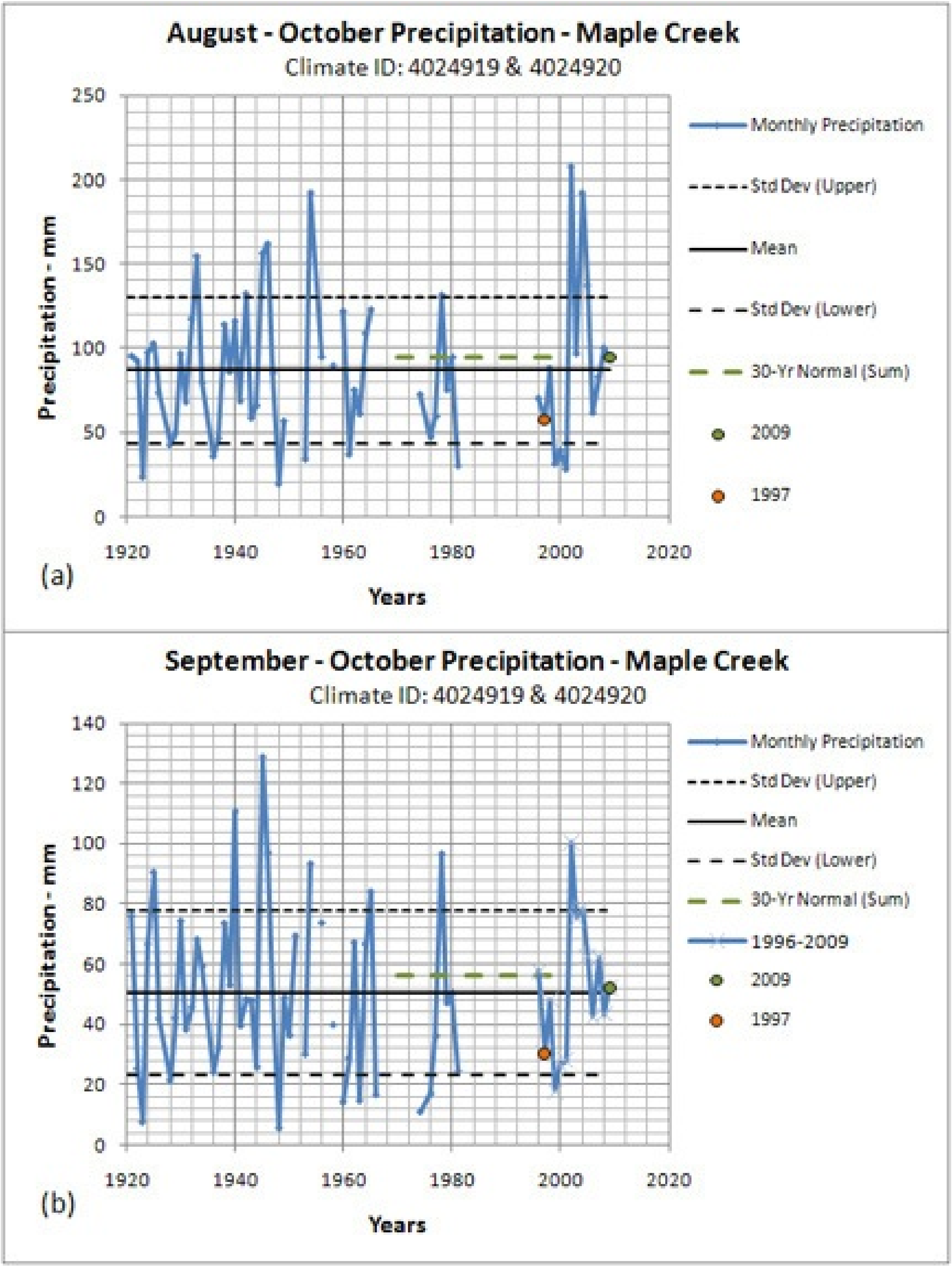

Figure 2.8: Fall Precipitation for Maple Creek

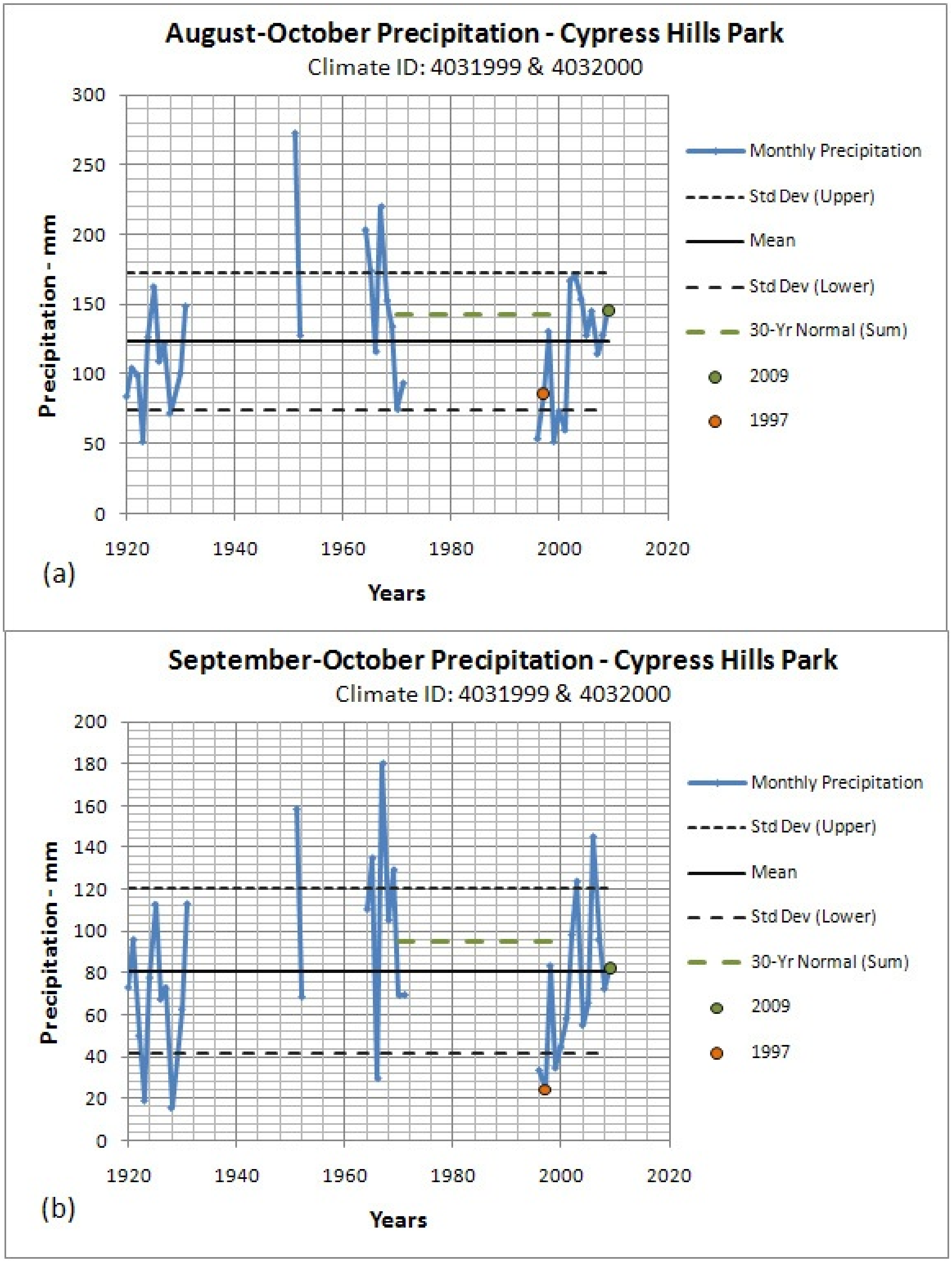

Figure 2.9: Fall Precipitation for Cypress Hills Park

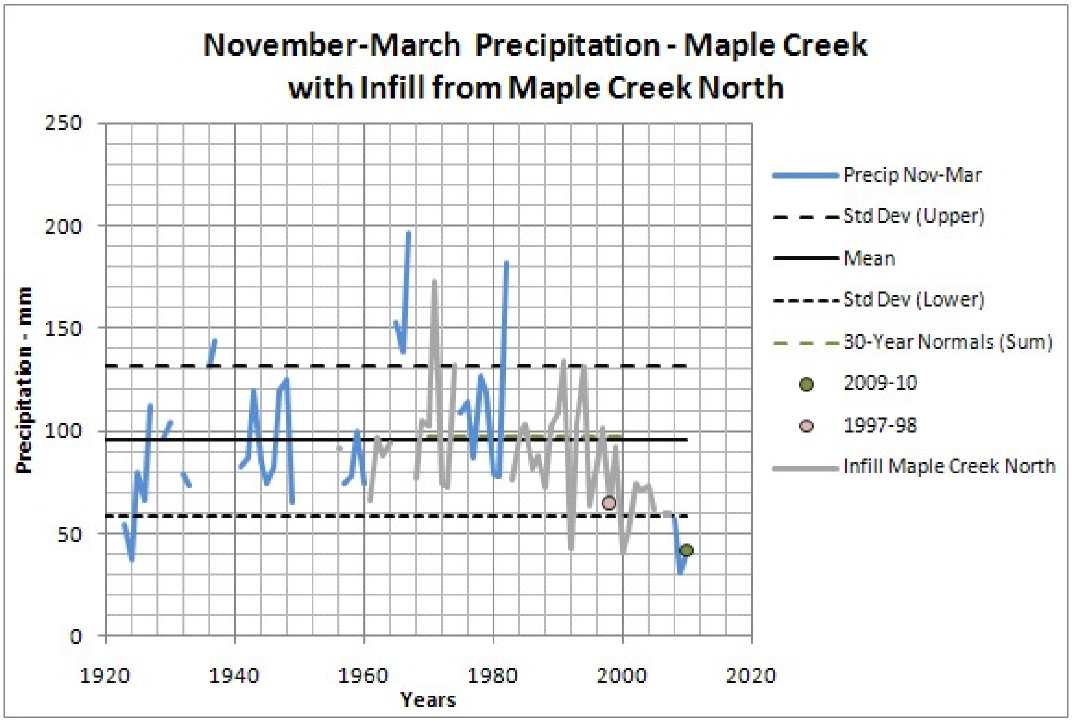

Figure 2.10: Winter Precipitation for Maple Creek

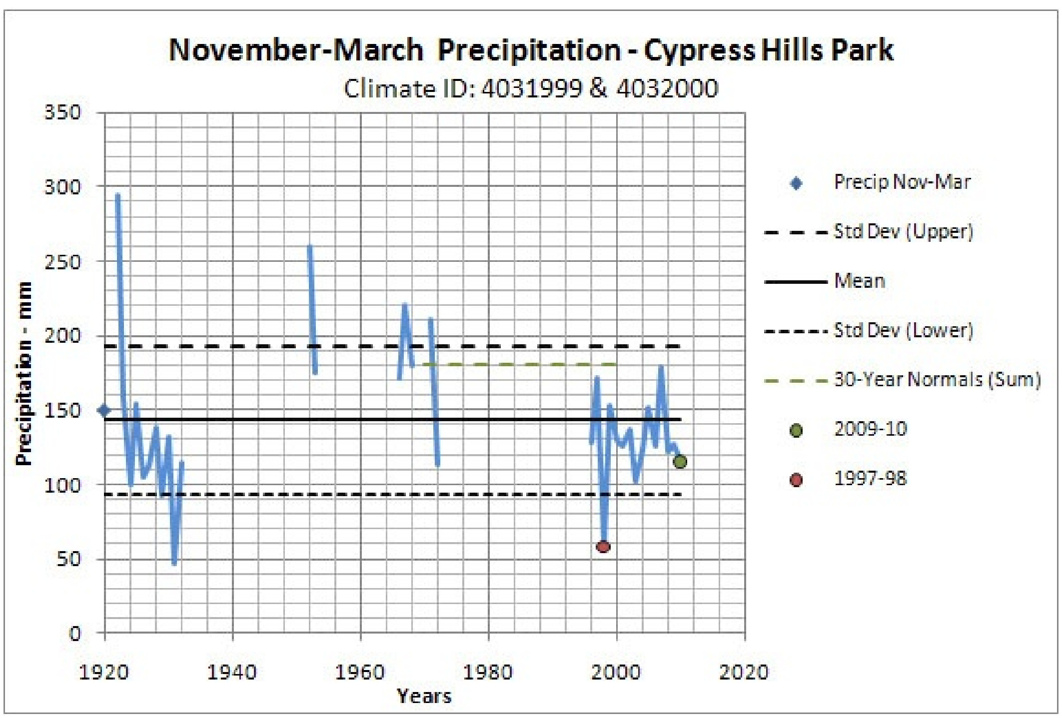

Figure 2.11: Winter Precipitation for Cypress Hills Park

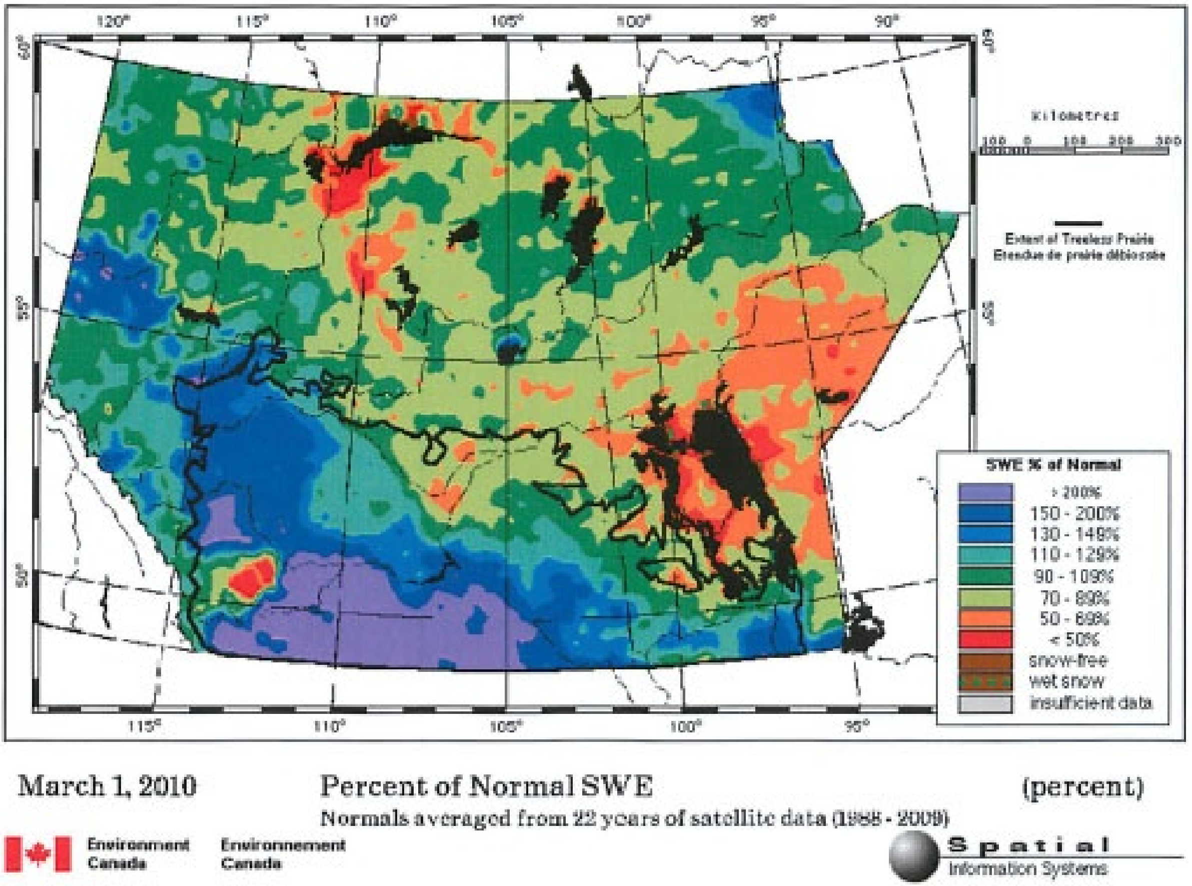

Figure 2.12: Comparison of the 2009-10 Winter Precipitation to the Normal

The spring runoff in 2010 was sufficient to fill all of the storage reservoirs and prime the drainage basin.

The rainfall for the months immediately preceding the June 17-18 storm was above normal as determined from Environment Canada records for precipitation stations at Maple Creek and Cypress Hills Park. A comparison of monthly precipitation totals for April and May with the 30-year normal precipitation is shown in Table 2.3.

The spring precipitation, represented by April-May and May total precipitation, were respectively the second and third highest in the 60 years of record as illustrated in Figures 2.13 and 2.14. This would indicate that the basin would be in a very wet condition at the time of the June storm with saturated soil and depression storage filled. These data are also tabulated in Appendix C.

TABLE 2.3 TOTAL 2010 MONTHLY PRECIPITATION COMPARED TO THE 1971 TO 2000 NORMALS						
	April			May		
Location	Normal (mm)	2010 (mm)	Precent of Normal (%)	Normal (mm)	2010 (mm)	Percent of Normal (%)
Cypress Hills Park	40.5	64.7	160	76.8	139.6	182
Maple Creek	28.37	52.1	182	48.1	104.7	218
Note: Normals are for Maple Creek North and Cypress Hills Stations						

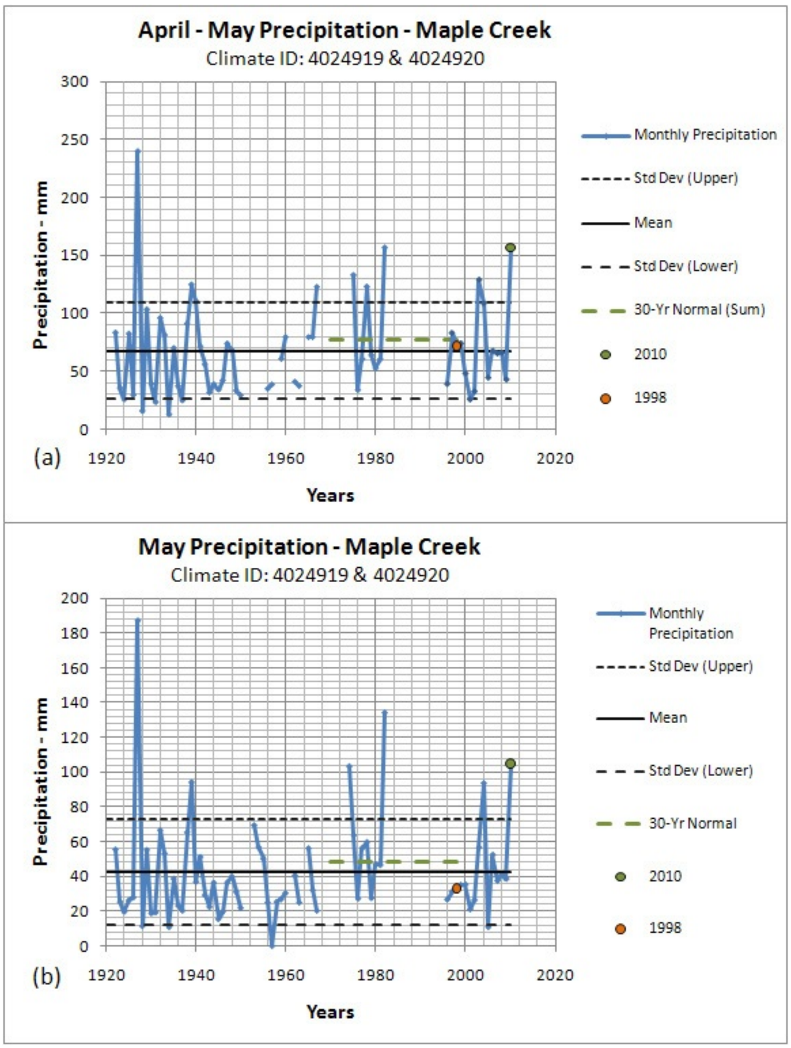

Figure 2.13: Spring Precipitation for Maple Creek

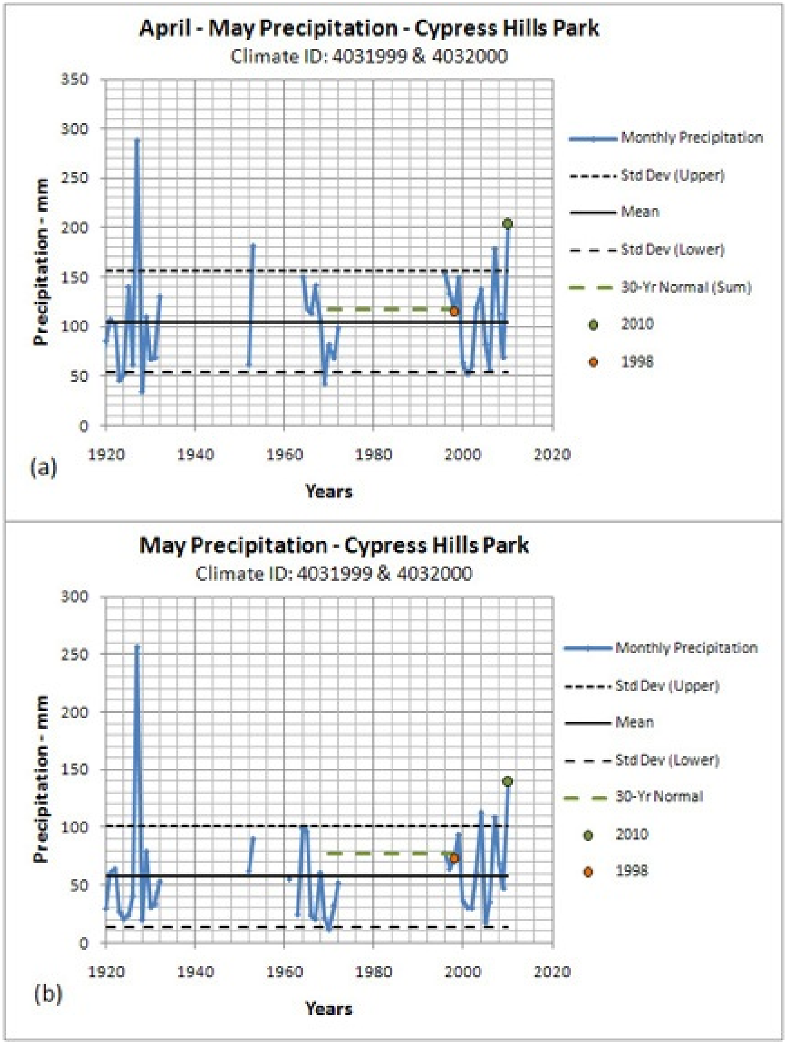

Figure 2.14: Spring Precipitation for Cypress Hills Park

The AESB Drought Watch website indicates that the accumulated April 1-May 31 precipitation "record wet". As well the drought watch website reported that on farm surface water supplies for the area north of the Cypress Hills were full at the end of May, 2010. These two maps are shown on Figures 2.15 and 2.16.

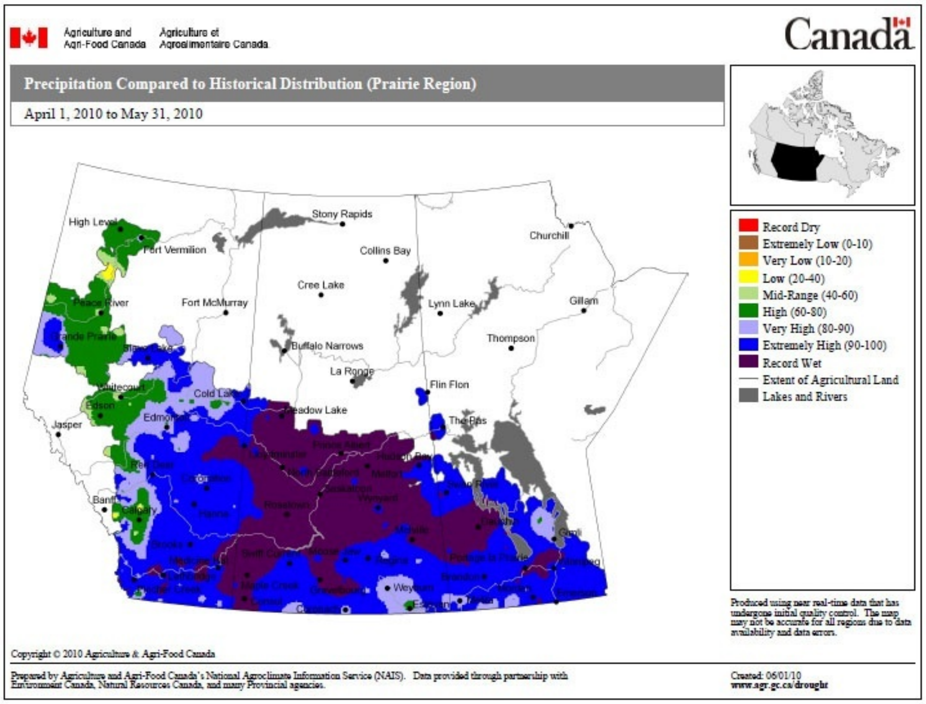

Figure 2.15: Spring Precipitation Compared to Historic - AESB Drought Watch

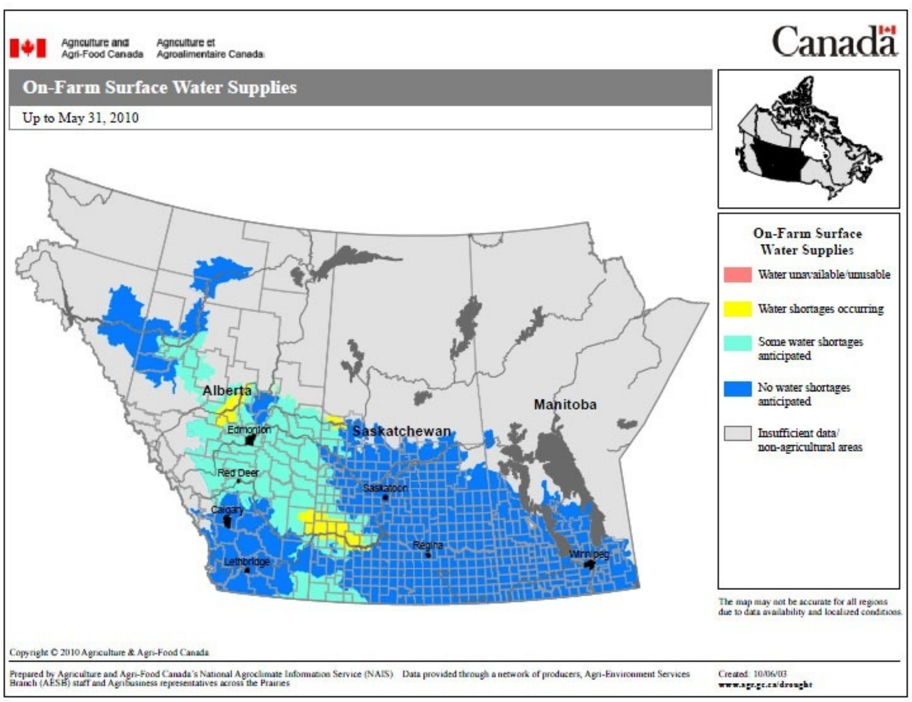

Figure 2.16: May 31 On Farm Surface Water Supplies - AESB Drought Watch

Further to this, May was a very cool month as shown on Figures 2.17 and 2.18. The cool temperatures limited evaporation from the water surfaces and evapotranspiration was low, leaving the soil moist.

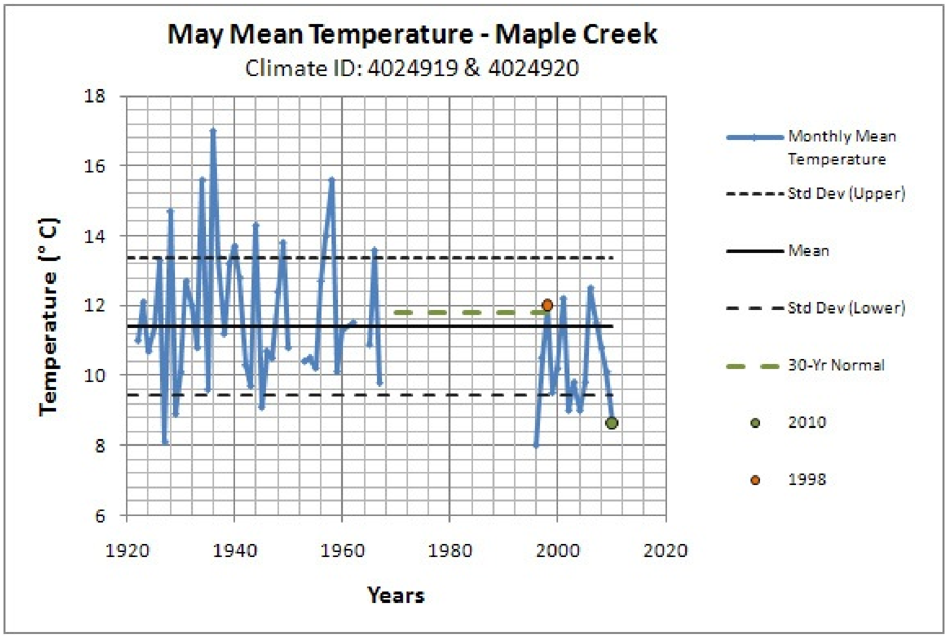

Figure 2.17: May Monthly Mean Temperature for Maple Creek

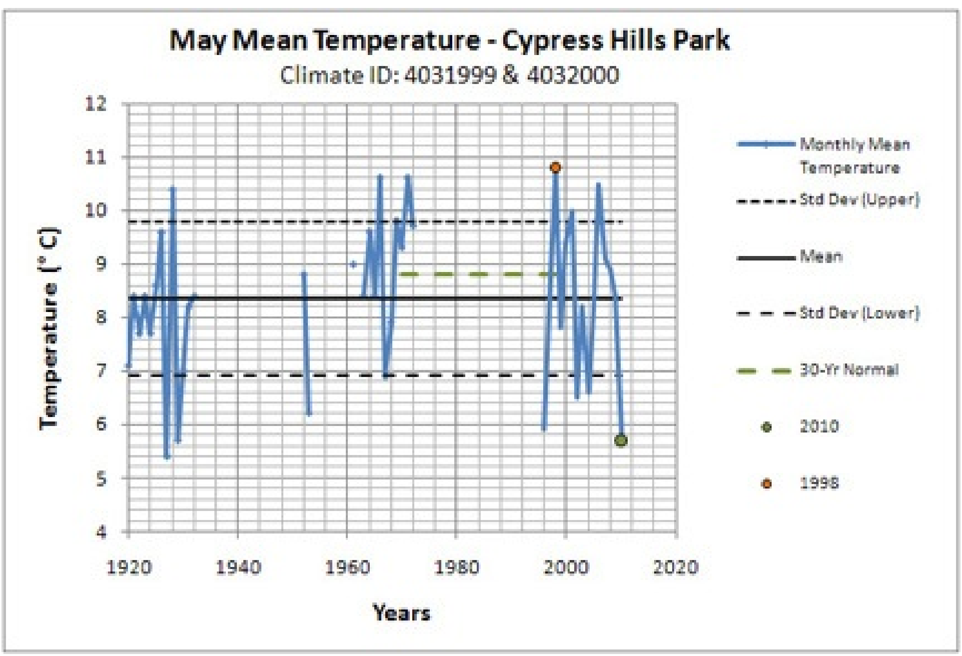

Figure 2.18: May Monthly Temperature for Cypress Hills Park

The combination of high precipitation and below normal temperatures resulted in very wet conditions in the basin prior to the June 17-18 2010 storm.

Antecedent conditions also played a role in reducing the runoff from the June 29, 1998 rainstorm. Spring precipitation was near normal (Figures 2.13 and 2.14) while the mean temperature for the month was slightly above normal for Maple Creek and well above normal for Cypress Hills Park (Figures 2.17 and 2.18). These conditions would lead to a relatively dry watershed allowing greater infiltration resulting in less surface runoff. The antecedent precipitation and temperatures for 1998 and 2010 are highlighted on the figures to emphasize the difference between these events.

How did a rainfall that averaged less than a 1:200 year event produce the 1:3700 year flood probability estimated in the hydrology section of this report? High rainfall in the two months prior to the event, and low temperatures combined to create a saturated basin. Depression storage was full, and temperatures were very low limiting evapotranspiration so the soil was saturated. All of this, combined with the moderately heavy rainstorm to generate an extreme flood event. The combined probability of two or more independent events is the product of the individual probabilities. The rainstorm ranged from under 1:100 years to 1:200 years in various parts of the basin with an average of about 1:100 years. It is difficult to assign a specific probability to the antecedent conditions represented by high monthly precipitation and low temperatures for May but even if it was equivalent to a 1:37 year event which would not be very extreme, the combined probability with a 1:100 year storm comes to 1:3700 years as indicated by the hydrology.

2.2 HYDROLOGY

2.2.1 Estimated Flows of Maple Creek and Gap Creek

Water levels observed by AESB at Junction Reservoir provided the best direct measure of the magnitude of the flood in the Maple Creek drainage basin. The reservoir levels provide a reliable basis for calculating the outflow from the reservoir based on the spillway rating curve. The recorded levels combined with the reservoir surface area curve from the original reservoir development surveys provide a measure of the changes in storage that occurred during the flood. AESB estimated the inflow to Junction Reservoir by calculating the balance of outflow and storage. Figure 2.19 shows the graphical calculation of the inflow and the data is tabulated in Appendix D. For comparison to the largest previous floods, Figure 2.19 also shows the floods of 1998 and 1955.

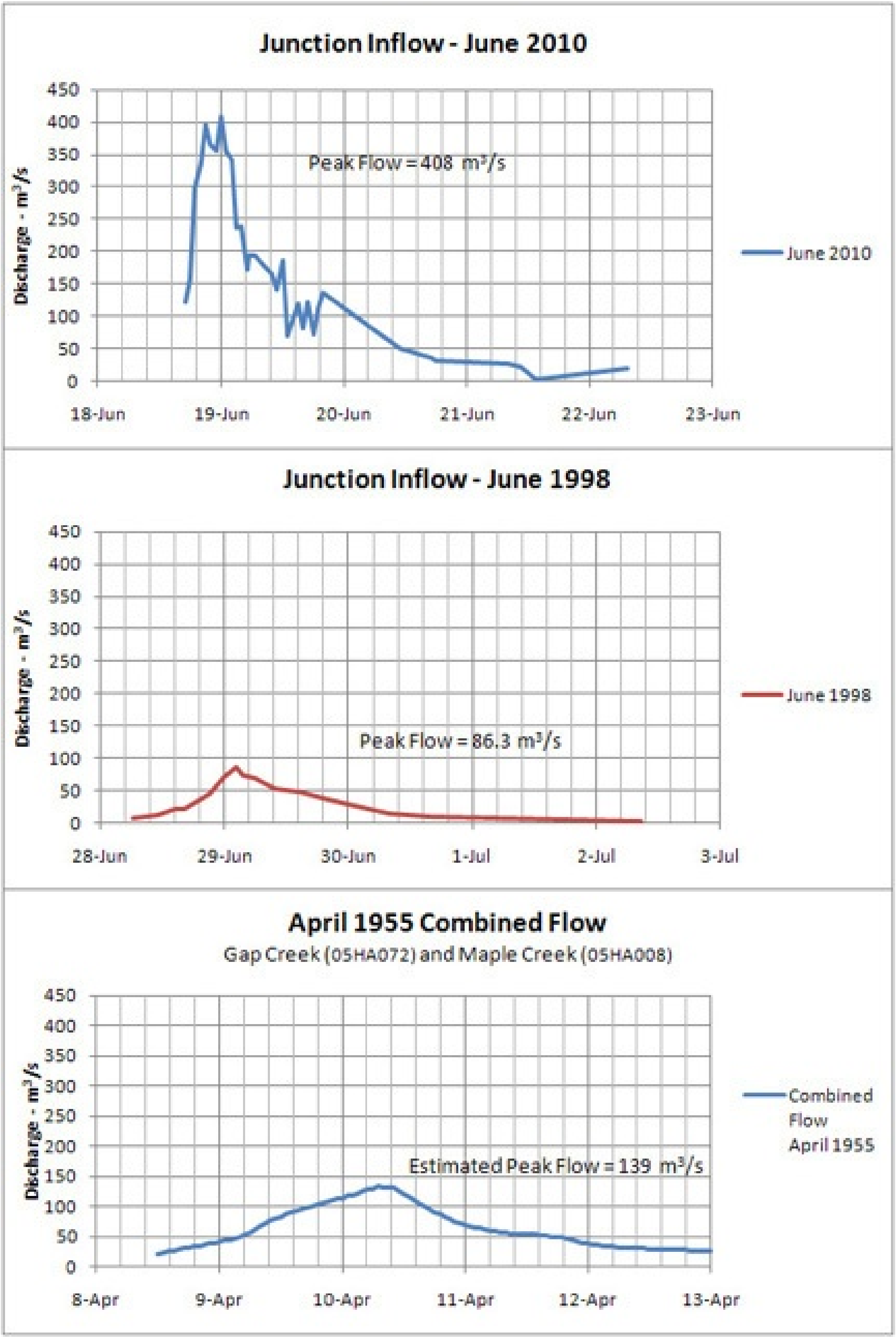

Figure 2.19: Inflow to Junction Reservoir - June, 2010; June, 1998 and April, 1955

The inflow to Junction Reservoir is made up of flow from the two main tributaries to Junction Reservoir: Maple and Gap Creeks. In order to analyse the hydraulics of Maple Creek through the Town of Maple Creek, it was necessary to estimate the proportions of flow that occurred in each of these creeks. There are hydrologic simulation computer models available which could be utilized to simulate the runoff from rainfall however there was not sufficient time nor information to do an effective calibration of these complex models. The simulation would have to include the two months of wet antecedent conditions as well as the storm event. Instead basic hydrologic principles were used to estimate the division of flow between the two sub basins. The objective is to estimate the flows in Maple and Gap Creeks so that when the hydrographs are combined the sum of the flows will be as close as possible to the calculated inflow to Junction Reservoir.

The process for determining the flood peaks is as follows:

1) Estimate the peak flows for Maple and Gap Creeks from the estimated inflow to Junction Reservoir using two methods:

a) Drainage area ratios - $Q_1=Q_{2*}(A_1/A_2)^a$ where Q_1 = unknown peak; A_1 = drainage area at location of unknown peak; A_2 = drainage area at location of known peak; Q_2 = known peak; and a = exponent applied to drainage area ratio. Past studies by SWA of data for the entire province of Saskatchewan have suggested that flood peaks in a watershed can be estimated from the peak for a second similar watershed using the ratio of drainage areas to the power 0.7. This exponent was tested first.

b) Unit runoff - $Q_1 = q_{2*} A_1$ where Q_1 = unknown peak; A_1 = drainage area at location of unknown peak; q_2 = the peak flow per unit area for the known peak. $q_2 = Q_2/A_2$. This method assumes flood peaks arise equally from all parts of the drainage basin and is similar to the first method with the exponent set to 1.

AESB estimates of gross and effective drainage areas are available for Maple Creek and Gap Creek in the vicinity of the Town and the reservoir. Table 2.4 lists the drainage areas. Hydrometric records are only available for periods prior to 1973 as listed in Table 2.5. The locations of the hydrometric stations are shown on Figure 1.1.

2) Select the appropriate drainage areas to use for this study. The choices were between using effective drainage areas (EDA) which is the area that is expected to contribute in a normal year and the gross drainage area (GDA) which would be expected to contribute during a very extreme flood event.

TABLE 2.4 DRAINAGE AREAS FOR SELECTED LOCATIONS IN THE MAPLE CREEK BASIN				
Station	Description	GDA	EDA	
05HA011	Maple Creek near Maple Creek	443.7	209.0	
05HA008	Maple Creek above Junction Reservoir	437.4	202.8	GDA includes Hay Creek (233 km²) which is not expected to contribute even in extreme floods
05HA012	Gap Creek near Maple Creek	728.8	646.2	
05HA072	Gap Creek above Junction Reservoir	578.3	529.7	
05HA019	Maple Creek at Menely's Farm	1230.8	910.6	

TABLE 2.5 HYDROMETRIC RECORDS		
Station	Description	Period of Record
05HA011	Maple Creek near Maple Creek	1910 to 1919
05HA008	Maple Creek above Junction Reservoir	1908 to 1973
05HA012	Gap Creek near Maple Creek	1910 to 1916
05HA072	Gap Creek above Junction Reservoir	1944 to 1973
05HA019	Maple Creek at Meneley's Farm	1915 to 1939

3) Determine the shape of the recession curves for Maple and Gap Creeks based on recorded hydrographs prior to 1973. Use the recession curve with the estimated flood peak to approximate the hydrograph for each of the sub basins. The analysis of recession curves is shown in Appendix D.

4) Combine the Maple and Gap Creek hydrographs in a spreadsheet and compare the combined hydrograph to the estimated flow into Junction Reservoir. Adjust the hydrographs using lag times to obtain the best fit using the flow records as a guide. The 2010 calculated inflow to Junction had two peaks three hours apart. The recorded 1955 flood peak for Gap Creek above Junction Reservoir lagged the Maple Creek peak by three hours. A visual inspection of the drainage network in Figure 1.1 suggests that distances are longer on Gap Creek and it would be reasonable to assume that the Gap Creek peak would lag the Maple Creek peak. It was concluded that the Gap Creek flow would lag Maple Creek flow by three hours.

5) Verification - Check the methodology by simulating the flood hydrograph for 1998 using the parameters developed for 2010 and the estimated flood hydrograph for 1998.

Three scenarios of contributing drainage area were considered in this investigation. The first scenario was based on effective drainage areas while scenarios 2 and 3 considered contribution from the gross area. The station at Menely's Farm was used to approximate the area contributing to inflow to the reservoir. The areas contributing to the Junction Reservoir inflow and the contributing sub-basins are described below. All areas are in square kilometers.

Scenario 1 -	Junction Reservoir	05HA019	EDA = 910.6
	Gap Creek	05HA012	EDA = 646.2
	Maple Creek	05HA008	EDA = 203.0

The EDA for local inflow is 55.4 km². It was assumed that this area would not contribute to the flood peak at Junction as runoff from the area would have peaked well in advance of the peaks in the larger and distant sub basins.

Scenario 2 -	Junction Reservoir	05HA019	GDA = 997.8
	Gap Creek	05HA012	GDA = 728.8
	Maple Creek	05HA008	EDA = 203.0

The local inflow EDA is 60 km². It was assumed that this area would not contribute to the flood peak at Junction as runoff from the area would have peaked well in advance of the peaks in the larger sub-basins.

Scenario 3 -	Junction Reservoir	05HA019	GDA= 939.5
	Gap Creek	05HA012	GDA= 728.8
	Maple Creek	05HA008	EDA = 203.0

In this scenario it was assumed that the 58 km² of gross drainage area between Junction Reservoir and the two upstream gauging stations did not contribute to the flood.

Flood peaks for each scenario were initially estimated using the drainage area ratio to the power 0.7 and the unit peak flow method. The "observed" flow for June 2010 as calculated by AESB from observations of reservoir levels is 408 m³/s. The results of the simulation are shown in Table 2.6 and 2.7.

All of the peaks calculated from the unit peak runoff were lower than the observed peak while all of the peaks calculated from the drainage area ratios to the exponent 0.7 were higher than the observed inflow. This suggests that the appropriate exponent lies somewhere between 0.7 and 1.0.

TABLE 2.6 ESTIMATED PEAKS USING DRAINAGE AREA RATIO TO THE POWER 0.7						
Scenario	Instantaneous Flood Peaks for June 2010			Peak Inflow to Junction Reservoir m^3/s	Difference	
	Maple Creek near Maple Creek m^3/s	Gap Creek near Maple Creek m^3/s	Estimated inflow to Junction Reservoir m^3/s		m^3/s	%
1	142.7	320.9	438.3	408	30.3	7.4
2	133.8	327.5	437.6	408	29.6	7.3
3	139.6	341.6	456.4	408	48.4	11.9

TABLE 2.7 ESTIMATED PEAKS USING UNIT PEAK FOR JUNCTION RESERVOIR							
Scenario	Instantaneous Flood Peaks for June 2010			Unit Peak for Inflow to Junction Reservoir $m^3/s/km^2$	Observed Inflow to Junction Reservoir m^3/s	Difference	
	Maple Creek near Maple Creek m^3/s	Gap Creek near Maple Creek m^3/s	Estimated Inflow to Junction Reservoir m^3/s			m^3/s	%
1	91.0	289.5	364.4	0.448	408	-43.6	-10.7
2	83.0	198.0	366.3	0.409	408	-41.7	-10.2
3	88.2	316.5	389.0	0.434	408	-19.0	-4.6

Since the exponent 0.7 consistently over estimates the peak and the exponent 1.0 under estimates the peak, an investigation was conducted to determine what value of exponent would be required to minimize the difference in simulated and “observed” flows at Junction Reservoir. The exponents for each scenario were plotted against the percentage difference in flow on Figure 2.20. The resulting curves were used to estimate the most likely exponent that would produce the minimum error. It was found that an exponent of 0.82 would produce the best results for Scenario 1 and Scenario 2 and an exponent of 0.92 was selected for Scenario 3. Simulations were re-run using the revised exponents. The resulting peak flows are shown in Table 2.8 and Scenario 2 is illustrated on Figure 2.21.

The best results were obtained using Scenario 2 with an exponent of 0.82. The peak flow of Maple Creek is estimated to have been 110 m³/s. A potential error range of 10 percent is suggested. The estimated hydrographs are tabulated in Appendix D.

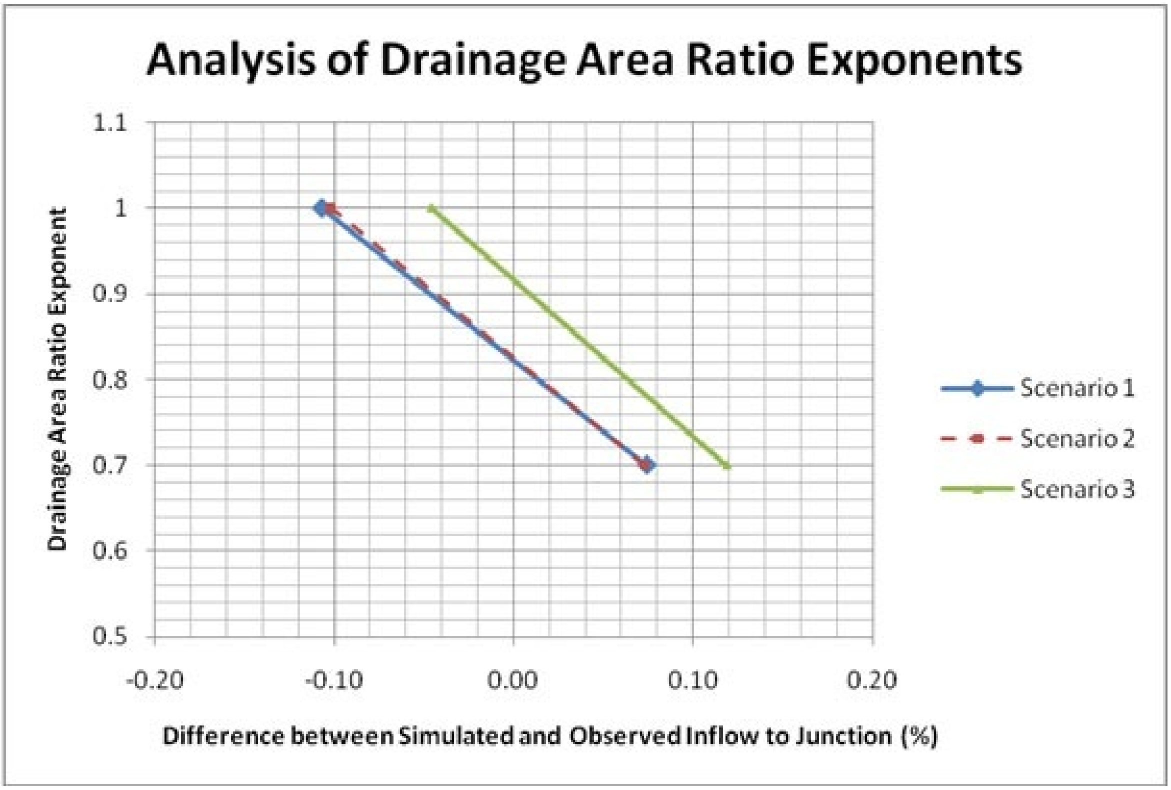

Figure 2.20: Drainage Area Ratio Exponent versus Difference between Simulated and Observed Peaks at Junction Reservoir

TABLE 2.8 ESTIMATED PEAK USING DRAINAGE AREA RATIO WITH ADJUSTED EXPONENT							
Drainage Area Scenario	Instantaneous Flood Peaks for June 2010				"Observed" Inflow to Junction Reservoir m^3/s	Difference	
	Exponent (a)	Maple Creek near Maple Creek m^3/s	Gap Creek near Maple Creek m^3/s	Estimated Inflow to Junction Reservoir m^3/s		m^3/s	%
1	0.82	119.2	308.0	406.0	408	2	-0.5
2	0.82	110.6	315.3	406.3	408	1.7	-0.4
3	0.92	99.7	323.0	405.0	408	3	-0.7

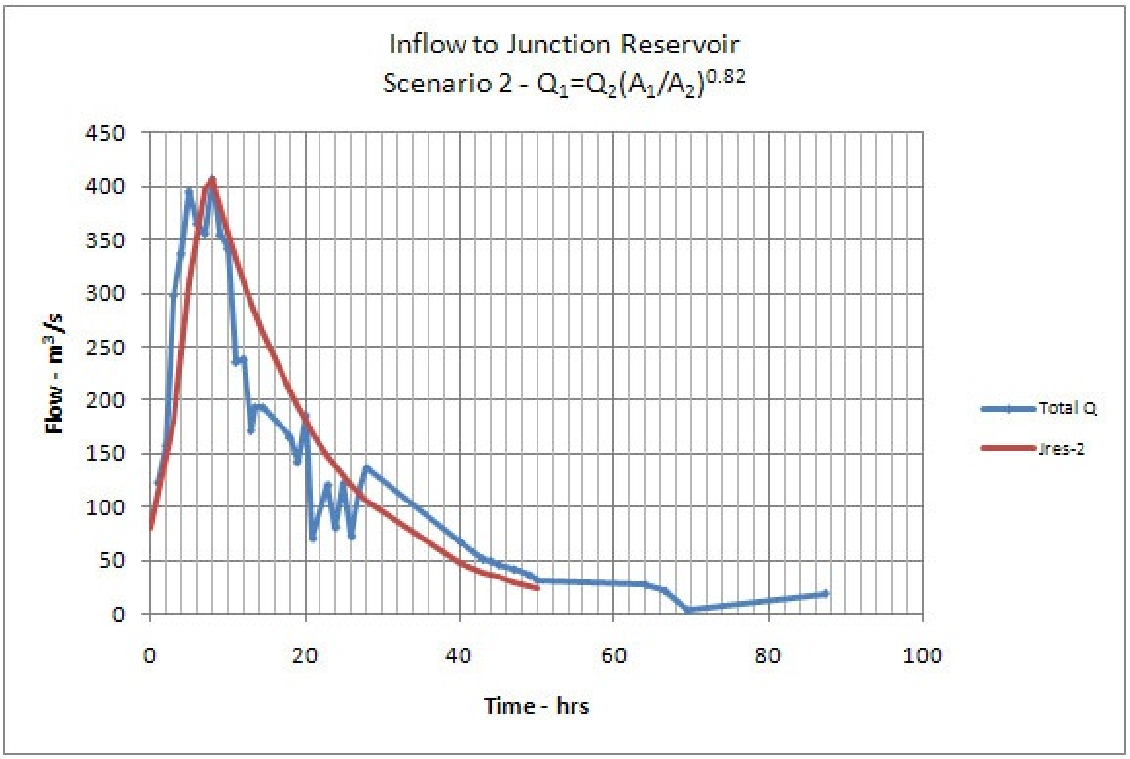

Figure 2.21: Simulated June 2010 inflow to Junction Reservoir with Adjusted Drainage Area Ratio

This calculation provided an estimate of the runoff that was generated in the Maple Creek basin upstream of Junction Reservoir. As discussed in Chapter 4, AESB operations at Harris Reservoir diverted about 10 m³/s of water from Maple Creek to Gap Creek. This diverted water reported to Junction Reservoir via Gap Creek but originated in Maple Creek and would have been part of the 110 m³/s. The flow at the Town would have been close to 100 m³/s.

2.2.2 Verification From 1998 Flood

The methods developed for the 2010 flood were applied to the 1998 flood. Using the drainage area ratio to the power 0.82 and the estimated Junction inflow peak of 86.3 m³/s yields flood peaks for Maple and Gap Creeks of 23.4 m³/s and 66.7 m³/s. These peaks were combined using the same lag and recession parameters as for the 2010 analysis. The resulting flood peak was 86 m³/s which agrees with the original estimate. The hydrograph is shown on Figure 2.22.

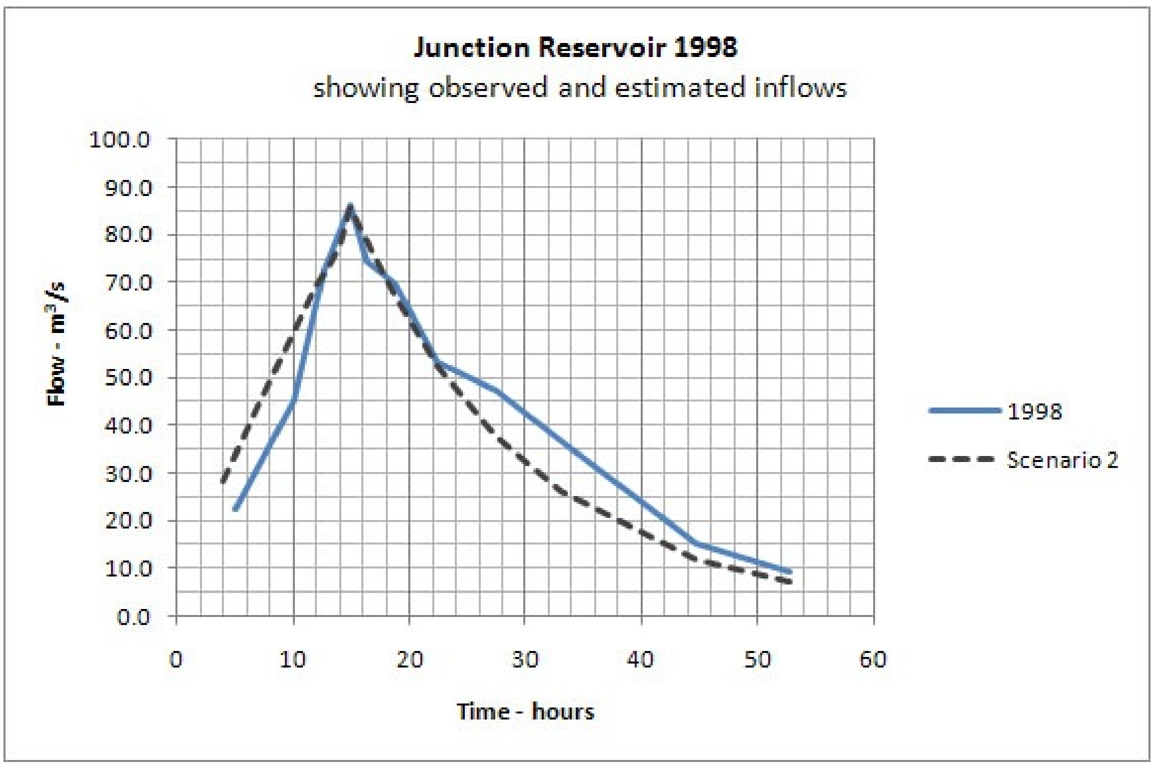

Figure 2.22: Calculated and Simulated Inflows to Junction Reservoir - June 1998

2.2.3 Comparison to Harris Reservoir Information

AESB observations at Harris Reservoir indicate that the water overtopped the Harris Diversion Dam on Fleming Creek. Based on the rating curve for that structure, that would require a flow of about 80 m³/s. The drainage area at this diversion of 71 km² is 35 percent of the area at Maple Creek which would suggest a peak flow of about 175 m^3/s might have occurred at the Town. However, if the same drainage area ratio assumption was applied to the whole basin, including Gap Creek, the estimated inflow to Junction reservoir would work out to 1025 m^3/s which is much higher than the reservoir level observations indicate. This apparent discrepancy is explained by the topography of the drainage basin. The flows at the Harris Diversion are from the steep portion of the drainage basin and are contained in a well defined creek channel. Little attenuation due to spreading into a flood plain would have occurred. As these extreme peak flows leave the steep hills and spread onto the flats south and west of town, a large storage volume becomes available to attenuate flows and the sharp peaks from the slope are attenuated. The data from Junction Reservoir provides the best indication of the flood flows after passing through the flats.

2.2.4 Hydraulic Confirmation

The hydraulic calculations detailed in Chapter 3 also confirm that the peak flow in the reach of Maple Creek by the Town was in the range of 100 m^3/s. In particular, the capacity of the CPR bridge can be calculated fairly closely and if the flow had significantly exceeded 100 m^3/s, that bridge would have been overtopped and the low segment of the rail grade east of the bridge in town would have overtopped and likely washed out.

2.2.5 Estimation of Return Period

There were four frequency curves available from previous studies by PFRA. Curves for snowmelt and rainfall were developed in 2004 for Junction Reservoir and snowmelt and rainfall frequency curves for Maple Creek above Junction Reservoir were developed in 1994. The 1994 estimates were intended as inputs to hydraulics studies for Maple Creek under the Canada-Saskatchewan Flood Damage Reduction Program (FDRP). These flood potential estimates are summarized in Table 2.9 and plotted on Figures 2.23 and 2.24.

These frequency curves are based on the log-normal distribution. These curves were used to estimate the plotting position for the 2010 and 1998 flood peaks at Junction Reservoir. It is estimated that the 2010 and 1998 flood peaks would have return periods of 3700 and 67 years respectively.

The 1994 study by PFRA indicated that the highest instantaneous spring flood peak at Maple Creek occurred on April 10, 1955 (37.4 m^3/s mean daily and 49 m^3/s instantaneous). Its frequency was estimated to be a 1:250 year event. At the time of the 1994 study the 1955 flood peak was three (3) times any recorded flood. In their 2009 study of the 1:500 flood plain delineation for Maple Creek Sameng Inc. used an instantaneous 1:500 flood of 70.5 m^3/s. This was considered to be conservative. In the current study the 2010 flood peak at Maple Creek is estimated to be 110 m^3/s plus or minus 10 percent which exceeds the 1:500 flood estimate.

The best estimate of the frequency of the Maple Creek flood peak is to assume the return period is similar, 1:3700 years, to that for Junction Reservoir. This would be consistent with the methodology used to derive the peaks.

TABLE 2.9
PEAK FLOW ESTIMATES FOR JUNCTION RESERVOIR AND MAPLE CREEK FOR VARIOUS RETURN PERIODS

Flood Event	Junction Reservoir		Maple Creek	
	Flood Peaks from Snowmelt Runoff	Flood Peaks from Rainfall Runoff	Flood Peaks from Snowmelt Runoff	Flood Peaks from Rainfall Runoff
	m^3/s	m^3/s	m^3/s	m^3/s
1:10	36	29	15	7.7
1:20	54	45	--	--
1:50	88	76	32	18
1:100	118	104	41	23
1:200	161	140	--	--
1:500	220	200	68	35
1:1000	280	265	--	--

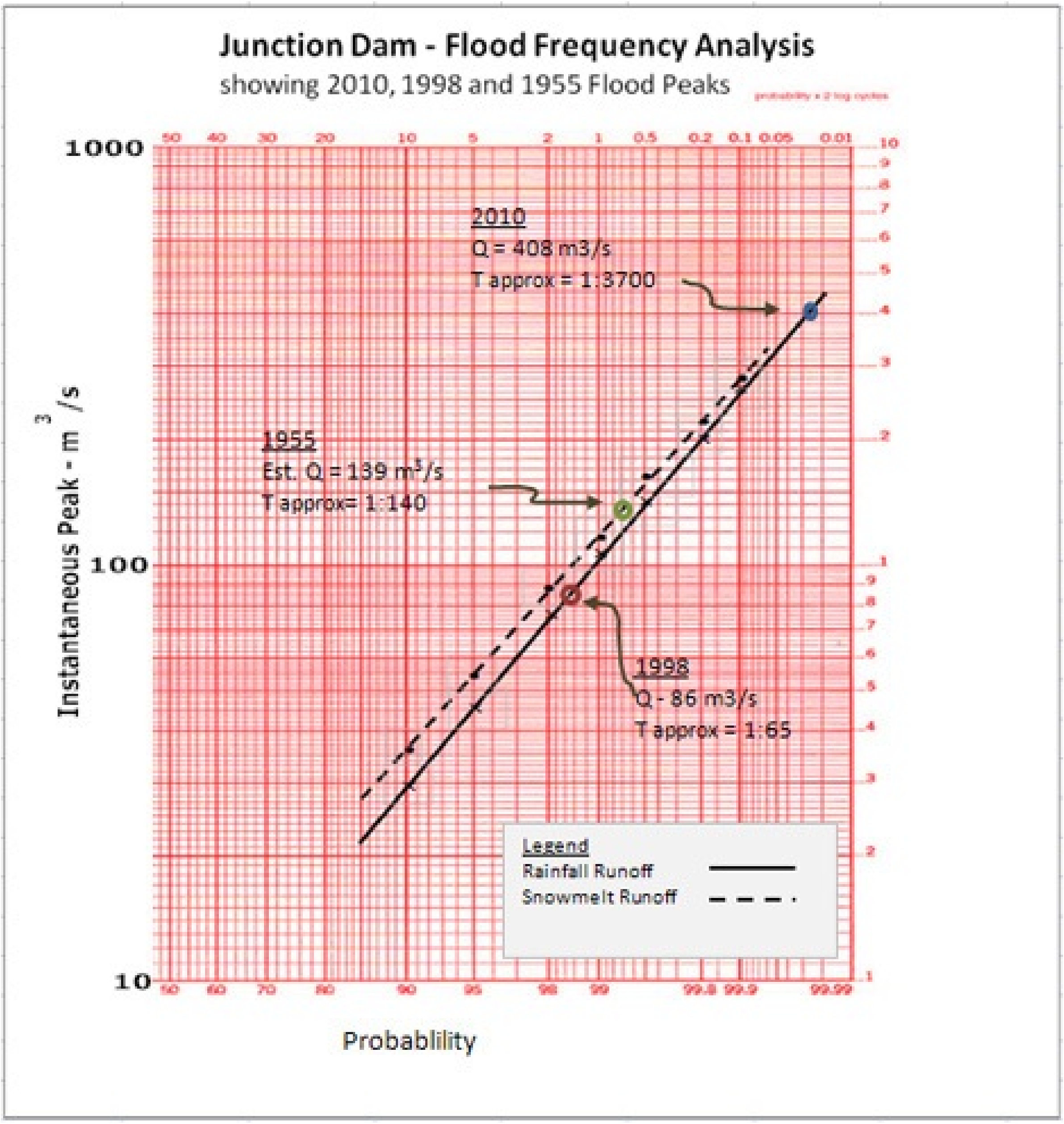

Figure 2.23: Frequency Curves for Junction Reservoir Inflow Peaks

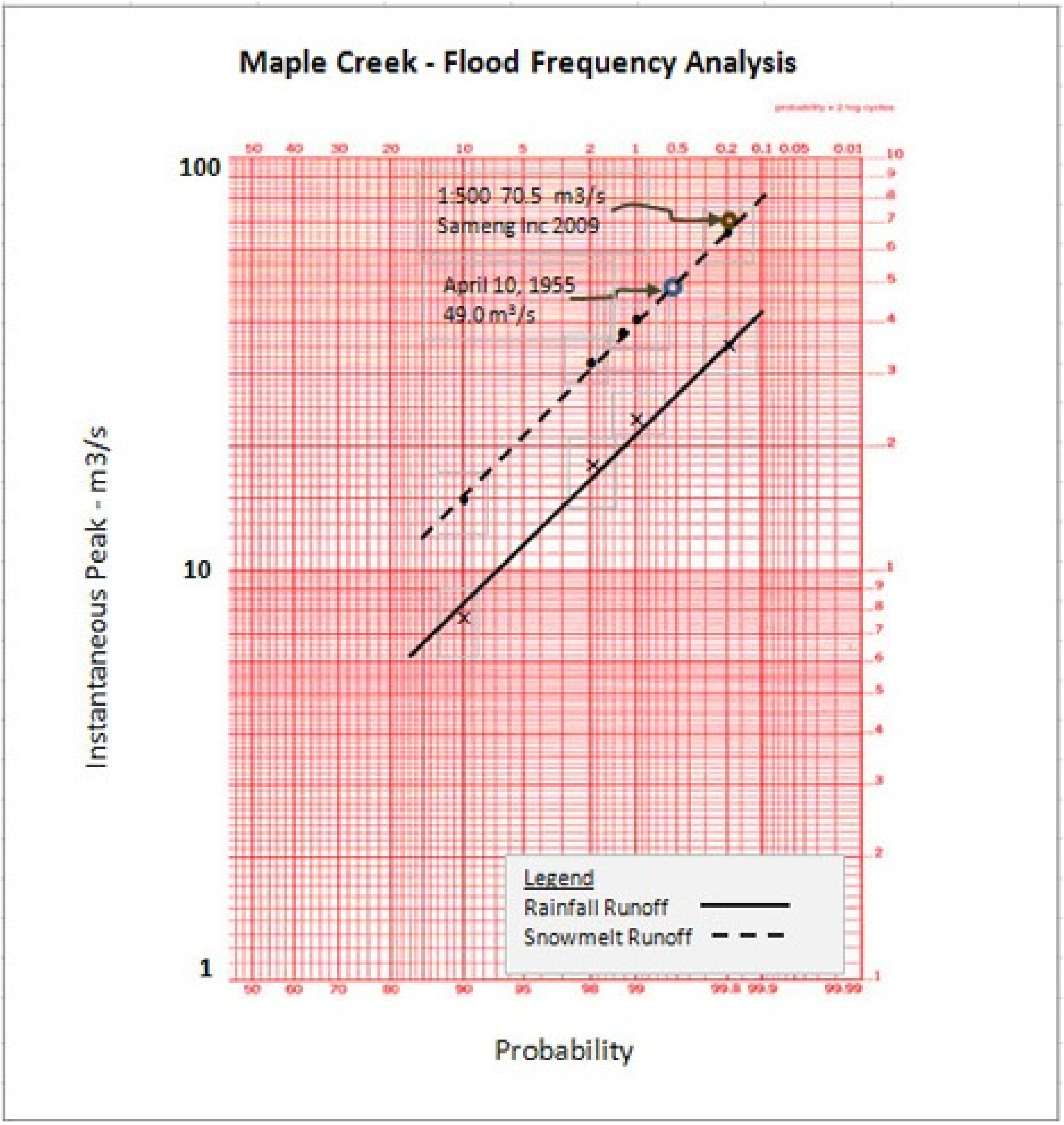

Figure 2.24: Frequency Curves for Maple Creek at Maple Creek

2.2.6 Flood Volume

AESB estimated the volume of the June 2010 flood at 29,100 dam^3. This is made up of approximately 28,200 dam^3 calculated from observed reservoir levels and an estimated 900 dam^3 of initial inflow. The estimates exclude runoff that was stored in McDougald, Downie and Harris Reservoirs. Given that precipitation was very high in April and May these reservoirs would have been at full capacity and any runoff diverted to surcharge the reservoirs would return to the creek after the peak and would not have a significant effect on the volume estimates. AESB estimates the return period of the runoff to be 1:250.

Flood volumes have a significant impact on reservoir levels. The maximum water level on Junction Reservoir on June 19 was 758.79 m. The maximum reservoir outflow was 195 m^3/s observed at 04:00 on June 19. It should be noted that this outflow was about 100 m^3/s less than would be expected for a 1:3700 year flood. The outflow for a typical 1:3700 year flood would be expected to be about 300 m^3/s (personal communication Brad Haid, AESB). This indicates that the volume was much less than the 1:3700 year event as discussed in the previous paragraph.

AESB estimated the unit runoff to be 23.6 mm over the GDA and 32.4 mm over the EDA. Using a basin average rainfall of 102.6 mm for the GDA and 107.7 mm for the EDA the resulting runoff coefficients would be 0.23 and 0.30 respectively.

2.2.6 Comparison to Probable Maximum Flood

Based on the probable maximum flood estimator for the Canadian Prairies (Water Resource Consultants Ltd. - March 2009) the probable maximum flood peak is estimated at 680 m^3/s and the runoff volume at 237,200 dam^3. This would produce a unit runoff of 193 mm resulting in a runoff coefficient of 0.43 based on a basin average PMP of 444 mm estimated by Hopkinson. The runoff coefficient should be higher in a more extreme flood.

The 2010 flood peak and volumes were respectively 60 and 12 percent of the PMF.

3.0 HYDRAULICS

3.1 MODELS

Hydraulic analysis of Maple Creek in the vicinity of the Town of Maple Creek was completed using the U.S. Army Corps of Engineers HEC-RAS 4.1 computer model. This model uses the geometry of the stream and flood plain and hydraulic friction factors to permit calculation of water levels that would result from various rates of flow in a given segment of a water course. In 2009, the Town of Maple Creek commissioned a study entitled “Maple Creek Floodplain Delineation Study” by Schaffer Andrews Ltd. and Sameng Inc. A copy of the model input files were provided and were used for the current evaluation of hydraulic conditions. The report and the model file are attached in Appendix E.

The model used 60 cross sections of the creek extending from downstream of the Town near the upstream end of Junction Reservoir to upstream of Highway 271 south of the Town. The model includes four bridges: Highway 271; a farm road bridge at Second Avenue; the CPR bridge; and the municipal road north of the CPR. The map in the report in Appendix E shows the location of the bridges and cross sections.

The Sameng analysis considered the 1:500 year flood peak of 70.5 m^3/s. The model was tested against this flow to confirm that it was operating as it was intended. The results were the same as those reported by Sameng.

A review of the model input data indicates that the surveyed cross sections only extend 20 to 30 m each side of the channel. The HEC-RAS model terminates the calculated flow capacity at the last data point on each end of the input cross section. Therefore, the calculations do not include the flood plain flow capacity beyond the surveyed cross sections. For ordinary floods, this would not miss very much of the flow capacity but in the extreme event of 2010, the flood plain would have been a very major part of the flow path and this model would only be a rough indicator of the hydraulic conditions. By using the topographic information from the FDRP plans, this deficiency can be readily corrected before final designs and planning proceeds. Upgrading this model was beyond the terms of reference for this study.

3.2 JUNE 18, 2010 FLOOD

As discussed in Chapter 2, the June, 2010 flood peaked at a flow of about 100 m^3/s at the Town. This flow was modelled and flood levels were calculated. Appendix E includes a Table of water surface elevations for the 2010 flood and for the 1:500 year flood. The 2010 flood ranged from 0.4 m to 0.9 m above the 1:500 year flood with the largest difference occurring upstream of the CPR.

Under the Canada/Saskatchewan Flood Damage Reduction Program (FDRP), a topographic plan of the Town was prepared. SWA provided a copy of the FDRP plan. The flooded areas reported by the Town were plotted on the plan that is included in Appendix E. For the north part of Town, the model levels are close to those experienced in the flood. The model results are conservative. For example, the model results indicate that the flood would have modestly exceeded the top of the CPR grade in Town but the actual flood did not quite reach this high. Farther south, the model results show levels about a metre higher than the flood descriptions. At Highway 271 on the south side of town, the model indicates levels over the bridge but the flood reports indicate that the flood went over the low sections of the highway but did not reach the bridge which is set higher than the rest of this segment of the highway. The Sameng report estimates that the model is accurate within about 0.5 m for the 1:500 year flood.

The model worked best in the north part of town where the bridges control water levels more than the channel. Farther south the flows were spread over the flats west of town and the flood plain carried a large part of the flow. The model cross sections included a small part of the actual flow path making the model results very conservative.

The areas on the west side of Town were flooded to a depth of more than 2 m above the ground level and the depth tapered to zero at the edge of the flooding in the centre of Town.

The model indicates that the bridges will have a significant impact on water levels in an extreme flood. In the 2010 flood, the model results indicate that the bridge on the municipal road north of Town would have caused about 0.9 m of backwater. The CPR bridge would cause about 0.3 m of backwater. The Second Avenue bridge west of town would cause about 1.5 m of backwater and the Highway 271 bridge would cause about 0.75 m of backwater. In a 1:500 year flood the model backwater impacts would be about 0.4 m, 0.2 m, 1.65 m and 0.45 m respectively. Because the model does not include all of the flood plain, it tends to over estimate the head loss at the Second Avenue bridge where flood waters would extend well beyond the surveyed section and to a lesser extent, the Highway 271 backwater would be over estimated. The CPR bridge impact would likely be under estimated because there would be more contraction and expansion loss as flood water converged into the bridge from the flood plain.

Other than Second Avenue west of Town, these backwater impacts are typical of what would be anticipated in extreme floods. The backwater at Second Avenue is quite extreme. A high calculated backwater at this bridge was confirmed by SWA and AESB staff comments and the recommendation to breech this road that was made during the flood. However, with flood plain flows, the actual backwater would not have actually been as extreme as this calculation indicates.

In order to determine the sensitivity of flood elevations to the backwater effect from high water surface elevations in Junction Reservoir, a run was made using a high reservoir elevation. Measurements during the flood indicated that the highest elevation reached by

Junction Reservoir during the 2010 flood was 758.79 m. The results of a model run using a slightly higher starting elevation of 759.0 m at Junction Reservoir, indicated that the increased starting water surface elevation tapered off to resume the normal flow elevations by the time cross section 3 was reached at the lower end of Town. That means that there is no measurable backwater effect from high reservoir levels in Junction Reservoir. Furthermore, it should be noted that the peak flow at the town site was past by the time the peak water level in Junction Reservoir was reached.

3.3 STAGE-DISCHARGE

In order to assist with future work along this reach of Maple Creek, the model was run for flows of from 10 m^3/s to 120 m^3/s to establish the stage-discharge relationships for key locations. Appendix E shows the results of this calculation. As noted earlier, these levels are very conservative and should only be used for preliminary planning. Before final designs are undertaken, the model should be upgraded.

3.4 FUTURE DAMAGE MITIGATION

3.4.1 Diversion

Although the AESB projects upstream of Town are designed for irrigation water supply, the operators were able to use the Harris Reservoir canals to divert an estimated 10 m^3/s of water from upstream of the Town to Harris Reservoir and a smaller amout to Gap Creek where the damage potential is very small. Although this eased the flooding in Town by a little over 0.1 m, it could not prevent the main flood damages.

In Saskatchewan, flood damage reduction planning is normally based on the 1:500 year design flood which has been identified as 70.5 m^3/s for Maple Creek at the Town. Based on model calculations, the creek at the Town has a capacity of about 10 m^3/s at its bank full level. The lowest flood prone properties are less than 1 m above the creek banks. In order to eliminate flooding and provide freeboard, a diversion capacity of about 60 m^3/s would be needed to prevent flood risk in Town.

This capacity could not be achieved by expanding the existing Harris Reservoir canals because Harris Reservoir is only attached to Fleming Creek which only has about one-third of the drainage area upstream of Town. To be effective a diversion would have to begin downstream of the confluence of Fleming and Maple Creeks. It would require a new canal to carry the flood flows west to Gap Creek. Bridges on at least 5 roads and the railway would require expansions due to the increased flood potential. The diversion canal would disrupt existing land uses and would likely require significant changes to irrigation systems.

A diversion would be very expensive to develop, would add to problems on Gap Creek and would not be a practical solution to the Town’s flood problems.

3.4.2 Upstream Storage Reservoirs

The existing storage upstream of Town is designed for water supply and, although it can provide modest flow reduction as it did in 2010, it cannot significantly reduce floods.

To be effective for flood control, storage must be dedicated to that use by being empty at all times. A dam located downstream of the confluence of Fleming and Maple Creeks or two or more dams located farther upstream could be designed to control flood flow up to some design flood level. To manage major floods, a project considerably larger than Junction Reservoir would be required. To ensure that the dam does not fail and add to future flooding it would have to be constructed to a very high standard.

Flood control reservoirs would be very expensive to develop and because they only pay dividends in rare flood events, they cannot be justified in this case.

3.4.3 Channel Improvements

As noted earlier, the channel of Maple Creek can only carry about 10 m^3/s. Any flows above this capacity spread into the flood plain and flood damages occur due to the low lying developments along the creek. Improving the flow capacity of the creek could lower the flood levels and reduce the extent and depth of flooding.

A review of the flood level profile indicates the most obvious opportunity to lower flood levels is at Second Avenue where a backwater of up to 1.5 m will occur in major floods. A close look at the cross sections and channel profile indicates that this is mainly due to the small bridge that can only handle normal flows. In any significant flood, the water must spill over the road grade. An additional concern in this reach is the creek channel which is constricted. Appendix E contains a table of flood levels for several options. Option 1 shows the impact of removing the Second Avenue bridge and lowering the road and modest improvement of the creek channel in the vicinity of Second Avenue. The flood levels upstream of Second Avenue would drop by about 1.7 m but the benefits diminish at locations farther upstream. At Highway 271 the gain is only a few centimetres.

Since work downstream sometimes tends to improve conditions through the whole town, two options for channel straightening in the downstream area were tested.

Option 2 would improve the channel downstream of the municipal road north of Town and Option 3 would straighten the channel between the CPR and the north municipal road north. Option 4 looks at the combined result if Options 1, 2 and 3 were all implemented. The results are tabulated in Appendix E.

Option 2 results in some modest reduction in water level in the downstream reach but very little benefit farther into Town. Option 3 produced no significant gains. Channel improvements would have to be much more aggressive than simply straightening the existing channel to achieve significant gains. Bridge replacements which would be very expensive would have to be a part of the channel improvements to significantly protect the town.

The Option 4 benefits result mainly from the potential improvements at Second Avenue.

The only modest channel improvement project that would achieve significant gains would be at Second Avenue. However, access to the land west of the creek would be impaired and at least a low level crossing to provide dry weather access would be needed.

3.4.4 Dyking

The worst damages resulted from water entering the areas along the west side of Town. A dyke could be constructed between the creek and these flood prone areas. The normal criteria for dyke design in Saskatchewan is the 1:500 year flood level plus 0.6 m of freeboard. Based on the 1:500 year flood levels in Appendix E and the FDRP topography the top of the dyke would be at 764.1 m at the south side of the CPR or roughly equal to the railway grade. It would rise to 764.6 m north of Second Avenue, to 766.1 m south of Second Avenue and to 767.6 m north of Highway 271.

There is a problem with closing a dyke off at the south side. Highway 271 is below the estimated 1:500 year flood level and substantially below the 1:500 year flood level plus freeboard. It would have to be raised about 1.5 m to provide flood protection.

Alternatively the dyke could be extended south of Highway 271 to join Highway 21 about 200 m south of Highway 271 where the grade elevation is high enough. However, this would leave a gap in the flood protection where Highway 271 passed through unless Highway 271 was regraded locally to the dyke height.

The dyke would average 2 m high with a section south of Second Avenue approaching 3 m in height. If combined with Option 1 of the channel improvements, the dyke height would be about 2 m high for its full length.

A common problem with dyking is the impact on the drainage of areas behind the dykes. For the west side of the Town of Maple Creek this is not a serious problem. The local drainage is from south to north. Culverts through the CPR allow local runoff to drain to the creek downstream of the bridges where flood levels are low enough to prevent water backing into the part of Town south of the CPR.

There are a few flood prone properties north of the CPR where dyking would be more difficult and would likely require consideration of each property individually.

Dyking is considered to be a risky method of flood protection because if the dyke is overtopped or fails, it can add to the risk. Dykes can create a sense of security which in some cases adds the risk of injury and loss of life to the property damage that occurs. There is a tendency to increase development behind dykes based on the assumption that it is safe which can add damage if a failure occurs.

Dykes must be well constructed and carefully maintained. Since flood events are rare and may not occur for periods of many decades, dykes are often forgotten and maintenance is neglected.

Dykes are usually considered to be a last resort for flood protection but in this situation with much of the Town in the flood plain, it is the only practical approach.

The Maple Creek flood plain carries a substantial portion of the flow during major floods. In order to avoid obstructing the flow capacity on the flood plain, dyking should be kept as far from the creek as possible.

A specific location is not proposed at this time because issues such as land use and ownership that are not currently defined need to be taken into account in locating the optimum alignment.

3.4.5 Flood Proofing Fill

Where land is to be developed in flood prone areas, it is possible to avoid creating added risk by placing fill to raise the land above the flood hazard level before development. In general this method of development is limited to flood depths of about 1 m but deeper depths of fill to provide freeboard may be practical. As mentioned for dyking, fill should not extend too close to the creek to allow the flood plain to continue its flow conveyance function.

Usually flood proofing fill is most applicable to new developments but it can be used for existing problem areas in some special cases or when areas are being redeveloped. In Maple Creek, flood proofing fill may be useful for the industrial lands north of the CPR.

The specific areas where development with flood proofing will be permitted should be defined through completion of the updated hydraulic analysis and development of a FDRP type of hazard map. Flood proofing criteria are normally incorporated in zoning rules.

3.4.6 Land Use Control

The simplest and least expensive method of avoiding flood damages is to avoid development of flood prone areas. This is not a practical solution for existing developments but can be very effective for preventing the expansion of damages in future floods. Avoiding developments in the flood plain also avoids the hydraulic problems associated with blocking the flood plain flows.

Land use controls through zoning should be a part of any flood damage reduction plan.

4.0 RESERVOIR OPERATIONS

4.1 FLOOD TIMING

A brief review of the sequence of events is presented to assist in following the operations:

1. Spring snowmelt primed the drainage basin and filled the reservoirs.
2. April and May snow and rain kept the drainage basin soaking wet.
3. Cool May temperatures resulted in limited drying.
4. Light showers in early June continued the wet condition.
5. Over night on June 17 to June 18 an extreme rainstorm occurred.
6. 1:00 am June 18, Daryl Jones who lives on a farm upstream of Town noted flood problems at his home.
7. About 2:00 am, Daryl began phoning the Town Mayor and others to warn of the flood.
8. By 5:00 am, emergency activities in Town were being organized.
9. Water levels rose rapidly through the day and although sand bag temporary dykes were tried, it was not possible to get ahead of the rising water.
10. By late afternoon on June 18 the water levels in Town were receding and clean up began.
11. The flood flows began arriving in Junction Reservoir about 4:00 pm, June 18.
12. Peak inflow the Junction Reservoir occurred at mid night on June 18/19.
13. Peak reservoir level and outflow occurred about 4:00 am, June 19.
14. Flow over Highway 1 began at 1:30 am June 19 and the highway washed out through the morning of June 19.

4.2 SUMMARY OF OPERATIONS

The four large reservoirs in the Maple Creek drainage basin were developed as a water supply system for irrigation . Since the runoff is extremely variable in this region with inadequate supply in many years, the reservoirs are kept full whenever adequate runoff occurs in order that they will carry as much water as possible into the drought periods that frequently occur.

In 2010, runoff generated by snow melt and April and May precipitation provided abnormally high volumes of runoff and allowed the operators to divert sufficient water to the reservoirs to fill the available storage. Prior to the June rain storm all storage was full.

Based on discussions with SWA and AESB staff the following describes their activities in response to the June rain storm.

As a result of the wet conditions earlier, the creeks were flowing more than usual for June and the operators of the dams were monitoring the conditions. On June 16 there was a light rain and on June 17 weather forecasts indicated the potential for rain. On the morning of June 17, after checking weather forecasts and weather radar, SWA staff began checking with AESB staff in Maple Creek. AESB reported that because of the rain they had begun releasing water from Harris Reservoir to Gap Creek in order to move water from the upstream side of the Town to the other creek. It was noted that the creeks were high but not threatening the Town.

As noted in Chapter 2, the heaviest rain occurred over night on June 17 and 18. An AESB staff member, Daryl Jones, who lives south of Maple Creek began experiencing flooding problems at his home during the night. He recognized that the flood was becoming serious and began contacting Town officials and others who he felt might be impacted. Because of the time in the early morning, he had difficulty getting anyone acting but his efforts did get the flood response started sooner than would be the case if he hadn't made the effort. It should be noted that providing flood warning is not a normal part of his job and that the only reason he was aware of the problem first was because he happens to live beside a creek upstream of the Town. By 3:00 a.m. AESB reported that water was over Highway 21 and running cross country upstream of Town.

Early in the morning water overtopped the Harris Diversion Dam which had never happened before. The Harris Reservoir diversion canal was open, allowing a flow of at least its capacity of 14 m^3/s to flow to Harris Reservoir. The Harris Reservoir outlet gates were open allowing flows at their capacity to leave the reservoir. About 5 m^3/s of Maple Creek water can drain to Gap Creek and about 3 m^3/s can flow back to Maple Creek. The reservoir filled and surcharged which allowed some water to go into storage during the peak flow period. Apparently the Harris reservoir operation removed about 11 m^3/s of flow from the Maple Creek peak. Since these capacity values are approximate for these surcharged conditions, a value of 10 m^3/s has been assumed as the benefit of these operations. Testing with the hydraulic model suggests that the impact on flood levels at the Town was likely about 0.1 m.

At the peak, a flow of about 3 m^3/s was going to Downie Lake and it surcharged. This may have slightly lowered the peak flow at the Town.

Rumours that McDougald Reservoir dam had washed out were reported later in the morning of June 18 but since this reservoir is not on any main stream this seemed unlikely. Access to the control works was not possible at the peak of the flooding but later it was confirmed that the reservoir was holding.

Later in the morning of June 18, the heavy rain was over and the flood waters were rapidly rising. In mid morning SWA recommended to the Town that Second Avenue should be breached because of its limited flow capacity. Through the day on June 18 reports of extreme water levels were reported and by late afternoon the highest levels reached Town. Harris Reservoir was about 1 m above FSL. The flat area west of Town was all flooded.

By 8:30 p.m. water levels were dropping in Town. Harris Reservoir was steady at its surcharged level and Downie Lake was about 0.3 m above FSL. At 9:30 p.m. Highway 1 was not flooded but by 1:30 a.m., June 19 it overtopped. Junction Reservoir levels peaked at 4:00 a.m. and Highway 1 was washed out on June 19.

No operations were carried out at Junction Dam during this flood. The reservoir was surcharged as the flood started and water was passing over the fixed crest spillway at a small rate. As the flood arrived on June 18, the reservoir level rose, storing a portion of the flow and allowing the excess to overflow. The reservoir rose to about 1.5 m above FSL and as a result, stored a large portion of the peak flow. The calculated peak inflow was over 400 m^3/s and the peak outflow was under 200 m^3/s or less than 50 percent of the inflow.

4.3 DISCUSSION OF OPERATIONS

4.3.1 Upstream of the Town of Maple Creek

The reservoirs upstream of Maple Creek are all off channel. They were designed as irrigation water supply reservoirs and have limited capacity to influence flood events. The diversion channel capacities are:

McDougald	7 m^3/s
Harris	14 m^3/s
Downie	10 m^3/s from Gap Creek to Downie Creek, then 30 m^3/s.

Only McDougald and Harris are upstream of the Town of Maple Creek. Downie is on a tributary of Gap Creek. If this flood event occurred when the reservoirs were low and had storage capacity, the Maple Creek flow upstream of the Town of Maple Creek could have been reduced by about 21 m^3/s or about 21 percent of the estimated flow. This would have lowered flood levels in Town by about 0.2 m. Even though these reservoirs were not low, they were used to the reduce the flow. The Harris Reservoir diversion canals were used to divert water from Maple Creek upstream of Town to Gap Creek and the McDougald Reservoir surcharged modestly. The reservoirs reduced the peak flow at the Town by an estimated 10 m^3/s or about 10 percent of the flood peak and half of the reduction that might have occurred if the reservoirs had been low before the flood.

It had been suggested that the reservoirs should have been lowered before the flood. This action would require knowledge that the flood was going to occur several days before the event or, if the reservoirs were operated for flood control, storage must be left empty all the time which then reduces the storage capacity in the droughts that follow. The value of the reservoirs for irrigation water supply would be compromised in exchange for relatively little flood control. Based on the hydraulic model of the creek discussed earlier, a reduction in flow of 10 percent would lower flood levels in Town by about 0.1 m.

Operating for flood control would provide minor reductions in flood levels. Operating for flood control would reduce the water supply potential. No change in operation is justified.

4.3.2 Impacts on Gap Creek

About 5 m^3/s of flow was diverted through Harris Reservoir to Gap Creek. The peak flow on Gap Creek is estimated to have been about 315 m^3/s so its flow was increased by roughly 1.5 percent due to this diversion.

Operations likely modestly increased the flow of Gap Creek but since the flows rejoin Maple Creek at Junction Reservoir and no facilities are directly impacted, no change in operation is justified.

4.3.3 Junction Reservoir

Since it is located on Maple Creek, Junction Reservoir automatically acts as a flood attenuation reservoir. Since its outflow over the spillway is proportional to the reservoir level, the level automatically rises with the flood flows and water is automatically stored during the peak inflow period then drains out after the peak flow. This resulted in more than a 50 percent reduction in the peak outflow.

If this reservoir was not a critical water supply reservoir, it could be kept below full to further attenuate flood events. However, lowering a reservoir a small amount below its spillway crest does not provide major flood control benefits because the extra storage is taken up in the start of the flood rather than at the peak.

This reservoir is automatically very effective in reducing floods as it is operated. There would be little gained by operating more aggressively for flood control. Any operation for flood control would jeopardize its water supply function. Therefore there would be no value in operating differently in future floods.

4.3.4 Community Contact

Because AESB has staff in Maple Creek, they were close to the flood activity and had considerable contact with the town. SWA staff reported being in regular contact with AESB but had little contact with the Town until after the flood. In essence, SWA relied on the presence of AESB to obtain their local information. The Town expressed concern that the SWA was slow to respond to this flood. The Town and residents who suffered damages were concerned with the timing of the province’s flood assistance payments.

5.0 DAMAGE SUMMARY

5.1 GENERAL

The rainstorm of June 17 to 18, 2010 caused wide spread damage to roads and public and private infrastructure throughout southwest Saskatchewan. This report discusses the particular problems associated with the extreme damages in the Maple Creek Area. Because many of the damages have not yet been fully repaired or assessed, data on the damages is limited. This chapter only provides a general overview of the high lights.

5.2 TOWN OF MAPLE CREEK

The largest concentration of damages occurred at the Town of Maple Creek where about one-third of the Town was flooded. All of the area west of Cypress Street was flooded and flooding extended east of Cypress Street in the north part of Town. The flood extended to Maple Street at Second Avenue, east of Jasper Street at First Avenue and east of Sidney Street at Pacific Avenue. An approximate outline of the extent of flooding as outlined by the Town is shown in Appendix E.

The deepest flooding on the west side of Town resulted in sewer back-up which increased damages in the area south of the CPR.

Flooding north of the CPR was less extreme but several industrial properties were impacted.

The duration of the flooding was relatively brief with most of the Town drained within one day of the flood peak.

5.3 HIGHWAYS

Most highways in the region were overtopped and briefly out of service but most only required minor repairs.

Highway 1 was washed out causing major disruption to traffic and replacement costs for the crossing structure. On June 19, Maple Creek outflows from Junction Reservoir exceeded the capacity of the two 3.6 m culverts. Approximately 175 metres of the west bound lanes entirely washed away and another 200 metres were damaged. The east bound lanes remained intact except that about 75 metres of the passing lane washed away. The highway was closed to traffic from the junction of Highway 21 to the Alberta border from June 19 to 26 while the east bound lanes were restored. The detour to Highway 7 added over 2 hours to the travel time. Two-way traffic at reduced speeds of 60 km/hr was allowed after temporary repairs to the east bound lanes were completed on June 26. Large trucks were still not allowed to use this route. More elaborate cross overs permitted more types of traffic after July 15 but speeds remained restricted. These restrictions remained in place until December 9 when the repairs to the west bound lanes were completed.

The culverts under the west bound lanes were replaced by a single arch with a span of 11.8 m and 4.8 m rise. The flow capacity was increased to 232m^3/s or an estimated 1:1000 year flood. The east bound lanes will be closed in 2011 to complete the new arch structure. The costs of the replacement and temporary works is expected to exceed $10,000,000.

5.4 AESB IRRIGATION STRUCTURES

The diversion structures for Downie Lake, Harris Reservoir and McDougald Reservoir were damaged by the flood and required repairs. Junction Dam did not have significant damage.

6.0 FORECASTING AND EMERGENCY RESPONSE

6.1 FORECASTING

6.1.1 2010 Forecasting

Flood forecasting utilizes knowledge of the interrelation between weather and streamflow. By studying the relationships between weather and flow it is possible to estimate the flows that will result from various kinds of weather. The relationships can be defined by complex computer simulation models to simpler statistical correlations. In Saskatchewan where most runoff arises from snowmelt the flood forecasting completed by SWA emphasizes spring flows. The accumulation of snow through the winter is monitored and as spring approaches it is possible to provide projections of potential runoff a few weeks before runoff and to refine these projections as the runoff progresses. The spring forecasts provide time for decisions to be made regarding operations for water supply and for flood mitigation.

In summer when runoff is rare, forecasting is less effective. The weather events that cause runoff are of short duration and, unlike snowmelt that takes days to produce runoff, rainstorms cause runoff immediately. On very large rivers like the Saskatchewan River system, flows can take several days to travel from the head waters and summer flood forecasting can significantly assist emergency planning. On small streams the time between the storm and runoff is very short and provides no opportunity for effective damage mitigation.

In the Maple Creek area the runoff arises a few miles south of the problem area on the steep slopes of the Cypress Hills. In the 2010 flood, the flood damages were already developing while the rain was occurring. Flood forecasting based on measured precipitation would not provide any lead time for emergency actions.

Flood forecasting based on weather forecasting is not practical. Weather forecasters can only report on the probability of storms in a region, not at any specific location until the actual storms begin to form then, using radar, more specific locations can be identified. However, by the time an intense rainstorm has formed and identified on radar, it is only minutes from its destination. If emergency flood preparations were undertaken for every forecast of potential intense rain, these preparations would be occurring several times each year with a high waste of effort for the many false alarms. Preparation would be made so often without useful results that the emergency responders would lose interest and would still not be effective in the rare real emergency.

In 2010, the SWA reports indicated that they were aware of the wet conditions leading up to June 17 and were monitoring the weather. However, these conditions were common across Saskatchewan and identifying specific locations for flood preparation was not practical.

The Town of Maple Creek can expect some indication of potential spring floods on the basis of snow accumulation but should not expect any significant warning of summer rainstorm floods.

In 2010, the Town did receive a few hours of warning when an AESB staff member who lives south of Town experienced flooding in the early morning of June 18 and, after taking precautions with his problems, phoned the Mayor and others in Town. However, this bit of warning was more of a coincidence than a planned benefit. Forecasting the Town's problems was possible because this person happened to be an experienced water project operator not because it was part of his job. His extra efforts couldn't prevent the flooding but at least allowed some opportunity to reduce the risks and damages. The Town expressed concern about the availability of forecasts. Unfortunately, it is not practical for SWA to provide detailed forecasts for all 500 urban centres and a similar number of rural municipalities across the province through the summer. The benefits would be minimal since, at best, the gain might be a few hours of warning in exchange for a very large annual costs of a monitoring and staffing program.

Over time SWA has worked on programs like FDRP to help communities reduce flood risks to minimize the need for last minute emergency flood response.

In the case of Maple Creek, there could be a small gain in the timing of warnings if there was some measurement of the stream conditions in the head waters, with automated transmission to the Town and SWA. However, the cost of installing and maintaining such equipment would be substantial and in the long run, after a few dry years, it would be difficult to retain funding. If the flood began at night as was the case in 2010, it is unlikely that anyone would be available to monitor the system. As noted earlier, this basin was among the first to have streamflow measuring stations but over time, they were all abandoned about 40 years ago.

Forecasting is not expected to be a reliable method of dealing with summer floods in this area.

6.2 EMERGENCY RESPONSE

The first priority of emergency response is always to protect from loss of life and injury. The few hours of warning combined with the natural progression of the flood ensured that no one was lost or injured. Protection of property was not as effective. The initial flooding started in the south near the hospital because this is where the flood waters first reached Town. Attempts to build temporary dykes were not successful because, as the flood peak moved north on the creek, flood water entered Town at new locations. Attempts to keep ahead of the rising water were not successful because the rising water kept moving north and bypassing dykes.

It was not possible to protect the whole west side of town in a few hours.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

7.1.1 Meteorology

1. The June 2010 precipitation event, while much above average, has been exceeded during the period of record. The basin average rainfall was roughly a 1:100 year event. Studies of potential maximum rainstorms indicate that the worst possible conditions could generate as much as four times as much rain as this storm.

2. Antecedent moisture conditions in the basin were well above normal leading up to the June 2010 flood with May being one of the wettest months on record.

3. Near record low temperatures in May 2010 reduced evaporation from reservoirs and evapotranspiration leaving the basin in an extremely wet condition.

4. The combined meteorologic conditions resulted in extreme runoff potential.

5. A slightly worse storm in June 1998 with lower antecedent moisture conditions due to below normal precipitation and normal temperatures produced a much smaller flood event.

6. Although the storm event is a key driving force for this type of flood, antecedent conditions are at least as important.

7.1.2 Hydrology

1. The return period for the June 2010 flood peak inflow to Junction Reservoir of 408 m^3/s is estimated to be 1:3700 years. This is more than four times the previous record peak in 1998 of 86.3 m^3/s despite having a greater 24-hour precipitation in June of 1998. The difference is attributed to very moist antecedent conditions preceding the June 2010 flood and normal antecedent conditions prior to the June 1998 event. This would have a significant impact on the runoff efficiency.

2. The flood peak from the Maple Creek portion of the drainage basin is estimated to 110m^3/s plus or minus 10 percent which is well above the estimated 1:500 flood of 70.5 m^3/s used in the 2009 flood plain delineation by Sameng Inc. Since about 10 m^3/s was diverted from Maple Creek to Gap Creek via Harris Reservoir, the peak flow at the Town was about 100 m^3/s.

3. The flood volumes were in the 1:250 year range or considerably less than would be expected for such a large peak flow.

7.1.3 Hydraulics

1. The HEC-RAS model developed by Sameng provides a good starting point for hydraulic analysis of floods at Maple Creek. To properly analyse extreme flood flows that extend beyond the surveyed limits used in this model, its input data should be extended, using the FDRP map data.

2. Bridges have a significant impact on flood levels. Most of the bridges cause backwater that would be normally expected but the Second Avenue bridge is much worse than would normally be accepted with backwater exceeding 1.5 m in floods.

3. Backwater from Junction Reservoir does not affect the Town.

4. Diversions upstream of Town are not a practical or effective solution to flooding.

5. Upstream flood control reservoirs are not a practical solution to flooding.

6. The only practical channel improvement that would provide moderate flood relief is the removal of the obstruction at Second Avenue.

7. Dyking is the most practical structural option to reduce future flood damages. Since dykes can increase the risk to life and injury if they fail or overtop, they must be built to a high standard and actively inspected and maintained.

8. Future developments in the flood plain should only be permitted if flood proofed to a safe level. Redevelopment of existing areas in the flood plain should include flood proofing.

9. Zoning control should be established to prevent further expansions to the flood damages.

7.1.4 Operation

1. Existing water control works in this area are for water supply, not flood control.

2. Operation of the water supply works reduced the flood peak by about 10 m^3/s or about 10 percent and lowered flood levels by about 0.1 m.

3. Diversion of Maple Creek water to Gap Creek increased the flow of Gap Creek by about 1.5 percent.

4. Junction Reservoir absorbed over 50 percent of the peak flow.

5. More direct contact by SWA with local officials during the flood would have helped reassure everyone that the province was aware of the situation and was looking for ways to help.

6. The timing of financial assistance was an issue for all flood damaged areas in the province and the experience from 2010 should help improve the delivery of timely help in the future.

7.1.5 Damage Summary

1. Widespread damage to roads and infrastructure occurred.

2. About one-third of the Town of Maple Creek was flooded with the deepest flooded areas receiving sewer backup as well as surface flooding.

3. Highway 1 was washed out and completely closed for a week while temporary repairs were done, then functioned at reduced capacity for 6 months while permanent repairs were done. It will be partially closed again in 2011 for completion of repairs. Total costs will be in the order of $10 million.

4. Portions of the AESB reservoir diversion structures south of Town were damaged and had to be repaired.

7.1.6 Forecasting and Emergency Response

1. Spring forecasting can provide some indication of the potential for flooding but summer flood forecasting is not practical.

2. The emergency response to the 2010 flood was very effective in preventing injury or loss of life. Little protection of property was possible.

3. It is not practical to protect property in Maple Creek from flood damages through emergency action. The exposure to the creek extends too far along the whole west side of town and the time for a flood to develop is too short

7.2 RECOMMENDATIONS

1. The hydraulic model should be updated to reflect levels observed in 2010.

2. Dyking is the only practical option to protect existing developments in Maple Creek from flood damages. Very high design standards should be applied. Inspection and maintenance are essential to the long term safety of dykes. The detailed design must consider the tendency of water from the creek south of Town upstream of Highway 271 moving east into areas that would not flood directly from the creek on the west side of Town.

3. Further investigation of the possibility of reducing the backwater at Second Avenue by installing some type of low level crossing is needed.

4. Flood proofing should be required for any future developments or redevelopment of the flood plain.

5. Zoning should be established to ensure that damage potential does not expand in the future.

6. Developments should not encroach too far into the flood plain, obstructing flood flows.