



CITY OF NANAIMO

**MIDDLE AND LOWER CHASE RIVER DAMS
SPILLWAY HYDROLOGY STUDY**

WATER

MANAGEMENT

CONSULTANTS

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April 30, 2002

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Attention: Scott Pamminger, A.Sc.T.
Engineering Services Technician

Dear Sir:

Re: Middle and Lower Chase River Dams Spillway Hydrology Study

Water Management Consultants is pleased to present our report on the Middle and Lower Chase River Dams Spillway Hydrology study.

We have enjoyed working with you and City of Nanaimo staff on this interesting and challenging study.

Thank you for this opportunity to provide consulting services to the City of Nanaimo.

Yours truly,

CDN WATER MANAGEMENT CONSULTANTS INC.



C. David Sellars, P.Eng.
Project Manager

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APRIL 2002

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CONTENTS

| | | |
|----------|---------------------------------|-----------|
| 1 | INTRODUCTION | 1 |
| 1.1 | Study Objectives and Scope | 1 |
| 2 | SITE DESCRIPTION | 3 |
| 2.1 | Chase River Watershed | 3 |
| 2.2 | Middle Chase River Dam | 3 |
| 2.3 | Lower Chase River Dam | 3 |
| 3 | SPILLWAY CAPACITIES | 5 |
| 3.1 | Middle Chase River Dam Spillway | 5 |
| 3.2 | Lower Chase River Dam Spillway | 6 |
| 4 | HYDROLOGY | 8 |
| 4.1 | Watershed Model | 8 |
| 4.2 | Probable Maximum Flood | 9 |
| 4.3 | Flood Frequency Analysis | 12 |
| 5 | MITIGATION | 13 |
| 5.1 | Mitigation Options | 13 |
| 5.2 | Emergency spillway (Option 4) | 14 |
| 6 | CONCLUSIONS | 16 |
| 7 | RECOMMENDATIONS | 17 |
| | REFERENCES | 18 |

FIGURES

| | |
|-----|--|
| 2.1 | Chase River Watershed |
| 3.1 | Lower Chase River Dam Spillway Water Surface Profile |
| 4.1 | Distribution of 24-hour Probable Maximum Precipitation |
| 4.2 | Distribution graph for SCS Type 1A storm |
| 4.3 | Probable Maximum Flood |
| 4.4 | Estimates of 1000-year Flood by Drainage Area. |
| 5.1 | Proposed Emergency Spillway Plan |
| 5.2 | Proposed Emergency Spillway Profile |

APPENDICES

Appendix 1 Budget Cost Estimate for Emergency Spillway (Option 4)

1 INTRODUCTION

1.1 Study Objectives and Scope

The Middle and Lower Chase River Dams are located within the City of Nanaimo. They were originally constructed for mine water supply but the reservoirs are now used for recreation. The locations of the dams and reservoirs are shown on Figure 2.1 and a contour plan of the dams is shown on Figure 5.1. The dams have been classified by the Dam Safety Branch of the Ministry of Water Land and Air Protection as having a high consequence of failure and should therefore be able to pass the Probable Maximum Flood (PMF) in accordance with the Canadian Dam Safety Guidelines (Jolley, 2000).

Water Management Consultants was retained by the City of Nanaimo to carry out a study of the spillway capacity of each dam and to determine the magnitude of the Probable Maximum Flood. The objectives of the study were to:

- Determine the Probable Maximum Precipitation (PMP) over the Chase River Watershed
- Determine the resulting PMF
- Determine the capacity of the dam spillways
- Make recommendations for spillway improvements (if required) and other revisions which may be required so that the dams are not overtopped in a PMF.

The scope of work included the following:

1. Attending a project initiation meeting with City staff for assembling background information to be provided by the City and to discuss project scheduling and other relevant issues.
2. Analyzing the study area and dam spillway structures and detail specific recommendations that meet the goals and objectives of the study.
3. Calculating the PMP in the Chase River Watershed and determining the PMF tributary to the Middle and Lower Chase River Dams.

4. Calculating the spillway capacity for the Middle and Lower Chase River Dams, based on surveyed measurements provided by the City of Nanaimo.
5. Comparing PMF volumes for each dam with their spillway volume capacity, and identifying any inadequacies of the spillways.
6. Providing recommendations for improvements to the spillways needed to ensure safe passage of the PMF through each spillway.
7. Liaising with the Ministry of Water, Land and Air Protection, Dam Safety Branch, for their approval of the results of the study and recommended spillway improvements.
8. Providing conceptual design drawings of any needed upgrades to the spillways along with budget estimates for the construction work.
9. Arranging a meeting with City staff to explain the Draft Report.
10. Arranging a meeting with City staff to explain the Final Report within one month of completion of the Draft Report review by the City staff.

2 SITE DESCRIPTION

2.1 Chase River Watershed

The Chase River flows through the City of Nanaimo but the majority of the watershed area lies west of the city outside the city boundary. The upper part of the watershed is at elevation 1000 m on the slopes of Mount Benson and the river flows east and enters the sea south of the city centre. The Chase River Dams are at elevations ranging from 70 to 90 m above sea level just downstream of the boundary between the City of Nanaimo and the Greater Nanaimo Regional District. The watershed area above the dams is primarily forested and bush land with some historical logging activity. The catchment area above the Lower Chase River Dam is 20 km² and is shown on Figure 2.1.

2.2 Middle Chase River Dam

Middle Chase River Dam is also known as Upper Harewood Dam and Upper Colliery Dam. It was believed to have been constructed in about 1910. The dam is located in a steep sided ravine with both abutments apparently founded on bedrock (EBA, 1992a). The structure consists of a 0.6 m thick vertical concrete core with end dumped soil and rockfill on both the upstream and downstream sides of the core. The dam is 12.5 m high, 50 m wide at the crest and the reservoir has a storage capacity of 92,600 m³.

Adjacent to the left abutment is a 14 m wide unregulated spillway (Photo 1). The base of the spillway crest is uneven excavated bedrock and the downstream channel floor appears to be primarily bedrock. A concrete wall along the right side of the spillway channel separates the embankment fill from the spillway. Another concrete wall extends part way down the left side.

The reservoir above the dam has a surface area of about 27 ha and is part of the City of Nanaimo park system.

2.3 Lower Chase River Dam

Lower Chase River Dam, located about 0.3 km downstream from Middle Chase River Dam, was believed to have been constructed about 1910. The dam is also known as Lower Harewood Colliery Dam and Lower Colliery Dam. The structure consists of a thick vertical

concrete core wall which ranges in thickness from 0.3 to 1.2 m at the base (EBA, 1992b). End dumped soil and rockfill including mine waste form both the upstream and downstream shoulders of the dam. The dam is located in a steep sided ravine with both abutments believed to be comprised of silty overburden. The dam height is 23.3 m, the crest width is 77 m and the storage capacity of the reservoir is 173,000 m³ (EBA, 1992b),

An uncontrolled spillway is located in the right abutment and is constructed of concrete. The spillway comprises a two bay entrance channel leading to a chute discharging over the edge of a rock ravine. Photo 2 shows the entrance channel just upstream of a footbridge. The entrance channel is 12 m wide with a central pier. The concrete sidewalls in the entrance channel are 1.0 m high. Photos 3 and 4 show the concrete chute, which has slopes ranging from 0.1 to 20% in the lower part. The wing walls of the chute are 0.9 to 1.5 m high. Detailed surveys of the spillways were carried out by the City of Nanaimo for this study.

The reservoir above the dam has a surface area of about 25 ha and is part of the City of Nanaimo park system.

3 SPILLWAY CAPACITIES

3.1 Middle Chase River Dam Spillway

The elevation-discharge relationship for the Middle Chase spillway was calculated using the weir equation $Q=CLH^{1.5}$ where:

Q is the discharge in m^3/s

C is a coefficient

L is the crest length in m

H is the elevation difference in m between the crest and the reservoir level

The width of the opening under the bridge is approximately 12.2 m as determined from detailed surveys carried out by the City of Nanaimo. For high reservoir levels, the C coefficient was estimated to be 1.67 based on tables in King and Brater (1953). The entrance to the spillway is roughly flush with the leading approach of the reservoir bottom justifying a lower value for the coefficient.

A steady state backwater analysis was also carried out using the U.S. Army Corps of Engineers HEC-RAS modelling software to determine the Q-H relationship for the spillway. The model predicted discharges for the spillway similar to that calculated by the weir equation with a coefficient of 1.67.

The bridge over the spillway limits the height of the water surface profile over the spillway which could potentially limit the discharge capacity. However, it was determined from the backwater analysis that critical depth occurs under the bridge and the flow depth at the bridge is about 0.1 m below the soffit when the reservoir level is at the crest of the dam. Therefore the reservoir level defines the maximum capacity of the spillway. With the reservoir level near the crest of the dam (elevation 88.3 m), the head above the spillway sill is about 2.1 m resulting in a peak discharge in the spillway of $62 m^3/s$. The maximum capacity had been estimated in previous studies to be $42 m^3/s$ (EBA, 1992a). In the previous study, it had been assumed that the elevation of the water surface at the soffit of the bridge was equivalent to the reservoir level. However, the elevation of the water surface profile is lower at the bridge than in the reservoir and hence the spillway capacity is greater than previously computed.

Table 3.1 shows the spillway discharges for different reservoir elevations.

Table 3.1: Middle Chase River Dam Spillway Capacity

| Reservoir elevation m | Head m | Discharge m ³ /s |
|--------------------------|-----------|--------------------------------|
| 86.2 | 0.0 | 0.0 |
| 86.4 | 0.2 | 2.0 |
| 86.8 | 0.6 | 10.0 |
| 87.5 | 1.3 | 30.0 |
| 88.0 | 1.8 | 50.0 |
| 88.3 (Dam crest) | 2.1 | 62.0 |

3.2 Lower Chase River Dam Spillway

The elevation-discharge relationship for the Lower Chase spillway was calculated using the HEC-RAS backwater analysis program. This program was developed by the U.S. Army Corps of Engineers to calculate steady state water surface profiles in channels. It is ideally suited to the conditions at the Lower Chase River Dam spillway as it can model both sub-critical and super-critical flow and channels of varying width and slope.

Fourteen cross-sections were used in the model to characterize the spillway. Four sections define upstream of the bridge, two sections are at the bridge, five sections define the 25 m reach downstream of the bridge where the grade is still near 1% and the side walls are constricting the width, and three sections define the reach downstream of the break point to the free over-fall. The break point in the spillway occurs approximately 25 m downstream of the bridge where the slope changes from 1 to 20%.

It was found that at low discharges of less than 5 m³/s, critical depth occurs at the bridge and remains as super-critical flow for the remaining length of the spillway as shown in Figure 3.1a. At higher discharges, however, the constriction in the spillway prior to the break point causes the flow to decelerate, resulting in sub-critical flow and an increase in water levels. As the discharge increases, the point at which flow changes to sub-critical moves upstream until the entire reach before the break point is sub-critical. This occurs when the discharge reaches 20 m³/s. Flow downstream of the break point is always super-critical.

Table 3.2 shows the spillway discharges for different reservoir elevations. The lowest point on the dam crest is 73.4 m so the maximum capacity of the spillway, with zero freeboard on the dam, is 35 m³/s. However, the HEC-RAS backwater analysis demonstrated that the increase in water levels due to the constriction upstream of the break point results in overtopping the sidewalls at a discharge of 25 m³/s. The water surface profile for a discharge of 35 m³/s and the right wing wall elevations are shown on Figure 3.1b. Overtopping of the sidewalls is unacceptable due to the dangers associated with uncontrolled flow and thus the sidewall elevations limit the capacity of the spillway to 25 m³/s.

Table 3.2: Lower Chase Dam Spillway Capacity

| Reservoir elevation m | Head m | Discharge m ³ /s |
|--------------------------|-----------|--------------------------------|
| 71.6 | 0.0 | 0.0 |
| 71.9 | 0.3 | 2.0 |
| 72.2 | 0.6 | 6.0 |
| 72.4 | 0.8 | 10.0 |
| 72.6 | 1.0 | 15.0 |
| 72.8 | 1.2 | 20.0 |
| 73.0 | 1.4 | 25.0 |
| 73.2 | 1.6 | 30.0 |
| 73.4 (Dam crest) | 1.8 | 35.0 |

Note: Maximum capacity is 25 m³/s without overtopping the spillway chute wing walls

Previous studies noted in EBA, 1992b indicated that the maximum capacity was 55 m³/s. The detail of the calculation procedures were not given but it is likely that it was assumed that critical depth would occur at the spillway entrance. In other words the spillway would act as a broad crested weir. In fact, the detailed spillway hydraulic analysis carried out for this reports shows that there are head losses in the approach channel and critical depth does not occur until a point downstream of the bridge. As a result the maximum capacity was determined to be lower than in previous studies.

It was concluded from this detailed study that the original design of the spillway was not very efficient. At the time it was constructed (about 1910) it was not possible to carry out the detailed hydraulic modelling that was used in the current study. The spillway chute does not have a steep enough gradient to overcome the contraction. The designers should have used less of a contraction to maintain super-critical flow for the full range of discharges.

4 HYDROLOGY

4.1 Watershed Model

To determine the water levels and discharges associated with a PMP over the Chase River basin, it was necessary to construct a hydrologic model of the watershed. HEC-HMS is the Hydrologic Modelling System developed by the U.S. Army Corps of Engineers and was the modelling software used for this study. HEC-HMS is designed to simulate the precipitation-runoff processes of watershed systems including converting precipitation to discharges and determining the effects of reservoir routing.

The hydrologic model created for the Chase River Watershed consists of reservoirs representing the Middle and Lower Chase River Reservoirs, a catchment basin above each reservoir, and precipitation values defining different storms. Due to the proximity of the reservoirs to each other, the model was designed for the Middle Chase River Reservoir to discharge directly into the Lower Chase River Reservoir without defining a reach between them.

The major basin for the model is above the Middle Chase River Reservoir and encompasses an area of just over 19 km² while the other basin, above Lower Chase Reservoir, is minor and only about 1 km². The catchment area was defined using a 1:20,000 scale contour plan as shown in Figure 2.1. The catchment area excludes any flow from the lake formed by Powerline Dam as high discharges in that area would continue to flow north rather than to the south.

To model peak flows from a basin it is necessary to define the catchment area and the processes the model will use to convert precipitation to discharge. There are three processes that need to be defined: loss, transform and baseflow.

The **loss process** determines the amount of precipitation that is lost to infiltration. For the Chase River model the U.S. Soil Conservation Service (SCS) curve number method was used to characterize losses. The SCS curve number method combines infiltration losses with initial abstractions to derive rainfall excess, which is the portion of the rainfall available for runoff. As part of the SCS method it is necessary to define the initial loss, percent impervious and the CN number, which is parameter characterizing soil moisture conditions. For both of the basins in the model the initial loss was set to zero, the percent impervious to 1%, as the basin has very little development and the CN number to 95 for the PMP. The reason a high CN number was chosen is discussed in Section 4.2.

The second process that needs to be defined is the **transform process**. Precipitation that does not infiltrate becomes direct runoff that travels across the ground to streams and rivers. The transform process determines surface routing of the runoff. For the Chase River model the SCS unit hydrograph method was used. The SCS unit hydrograph is a dimensionless unit hydrograph that was developed by the SCS from recorded data on small watersheds. The dimensionless unit hydrograph is built in to the HMS model and is selected by the user. The SCS unit hydrograph is a single parameter hydrograph defined by the lag time. This means that the shape of the hydrograph is a function of the basin lag time. The longer the lag time, the wider the hydrograph and the lower the peak.

The following equation from Haan *et al.* (1994) was used to calculate the lag time which is based on observations on natural watersheds.

$$T = L^{0.8} (S+1)^{0.7} / 1900 Y^{0.5}$$

Where T is the basin lag in hours
 L is the hydraulic length of the basin in feet
 S is the maximum soil water retention parameter defined as $1000/CN - 10$
 Y is the average land slope in percent

The lag time was calculated to be 47 minutes for the basin above the Middle Chase Reservoir and 15 minutes for the small basin above the Lower Chase Reservoir. There were no flow data available to calibrate the model and verify this calculation of lag time. However, based on experience on other watersheds where monitoring data are available, this value for lag time appears reasonable.

The third process, **baseflow**, determines the contribution to channel flow from groundwater. A constant monthly baseflow method was used to define the contribution from groundwater and recession flows from storms prior to the occurrence of the PMP. A constant discharge of $5 \text{ m}^3/\text{s}$ was assumed for the large basin and $2 \text{ m}^3/\text{s}$ for the small basin. This was based on mean monthly flows recorded during wet periods from gauged creeks on the east side of Vancouver Island.

The reservoirs in the model are characterized with an elevation-storage-discharge table. The elevation-storage relationships were derived from 1 m contour information. The discharge-elevation relationships were derived from Table 3.1 and Table 3.2.

4.2 Probable Maximum Flood

The Probable Maximum Flood (PMF) has become a standard design criterion for flood protection for major dams over past decades. The PMF is a hypothetical flood based upon a set of assumptions that attempt to define the maximum flood potential for a particular site. For small watersheds in coastal British Columbia, the PMF will occur as a result of the

occurrence of Probable Maximum Precipitation (PMP) in conjunction with adverse antecedent conditions (Melone and Sellars, 1988).

Probable Maximum Precipitation (PMP) is "the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location at a given time of year" as defined in U.S. Department of Commerce, 1994 Hydrometeorological Report No. 57 (HMR-57). It is recognized that the derivation of a PMP requires considerable professional judgment informed by currently accepted practice. In a paper presented at a Canadian Dam Safety Conference (Sellars, 1991), it was observed that because of the inherent uncertainties in estimates of PMP and PMF, "collective judgment" provides a more reasonable approach than individual judgment. In other words, the standards used for developing a PMP should be based on the most common current thinking among practicing hydrometeorologists. Therefore, it is appropriate that the methodology for this study be based on current accepted practice for the area as detailed in HMR- 57 and recent updates to PMP for southwestern British Columbia carried out by Water Management Consultants for BC Hydro.

The standard methodology for deriving an estimate of PMP by hydrometeorological methods involves the following steps:

- Identification of large historical storms
- Separation of storms into convergence and orographic components
- Maximization of the convergence component from observations of moisture available in the storm and comparison to maximum moisture potentially available
- Identification of the largest maximized storm
- Combination of the maximized convergence component with the orographic component to produce the PMP.

HMR-57 divides storms into two main types:

- **General storms** have durations exceeding 12 hours and precipitation is widespread as a result of a major synoptic scale disturbance (low pressure system or strong frontal activity). In British Columbia, general storms produce the most widespread heavy precipitation in the cold season (November through March).
- **Local storms** have produced the greatest observed short-duration rainfalls over small areas in Washington and British Columbia. These are generally thunderstorms that occur during the warm season (April through October).

The study of regional PMP carried out by WMC for BC Hydro provided estimates of 24-hour PMP for 1.25 minute longitude and latitude grid cells throughout south-western British Columbia. Twelve grid cells define the PMP for Chase Creek Watershed. The total 24-hour PMP for these cells ranges from 204 to 300 mm. The 24-hour PMP values for the Chase River Watershed are shown on Figure 4.1.

Using the values shown in Figure 4.1, the average 24-hour PMP over the Chase River Watershed is 242 mm. To provide an estimate of the effect of area reduction, depth-area relationships reported in HMR-57 were used. For a 20 km² basin the 2% reduction due to area decreases the average point PMP values to 237 mm.

To estimate short duration rainfall the SCS Type 1A storm distribution was used, which is recommended for this area. (Haan *et al.*, 1994). The one-hour PMP storm based on the SCS Type 1A storm distribution was found to be 40 mm. This rainfall is likely to occur in the cold season (November through March) and would have very wet antecedent conditions. The distribution of the SCS Type 1A storm is shown on Figure 4.2.

Using a 24-hour PMP rainfall, a CN value of 95 and the Type 1A distribution, the PMF inflow peak was calculated to be 192 m³/s for the Middle Chase Reservoir. The CN value of 95 was chosen to simulate extreme wet antecedent conditions, which would be typical for a winter storm and consistent with standard PMF procedures which require adverse, conservative conditions to be modelled. The rainfall hyetograph and the runoff hydrograph are shown in Figure 4.3. The storm was modelled for 24 hours but the first 12 hours are shown in this graph to provide greater definition, which is possible because the peak occurs within the first 12 hours. The peak PMF at the Lower Chase River Dam was calculated to be 198 m³/s. There is very little peak flow attenuation provided by these small reservoirs.

As a sensitivity test it was found that increases in the lag time of 20% decreased the peak PMF by 8%. A decrease in the lag time of 20% increased the peak flow by 10%.

Consideration was also given to the occurrence of a local storm. The procedures used to develop a local storm applied generalized relationships from the HMR-57 study as follows:

- Use generalized local storm maps from HMR-57 to identify 1 hour, 1 square mile storm
- Adjust for duration using depth-duration curves provided in HMR-57
- Adjust for basin area using depth-area relationships

The analysis indicated that a local storm could produce a one-hour PMP of 60 mm. However, this storm would occur in the summer when the antecedent soil moisture conditions are dry. The highest intensities with this type of storm occur within the first hour so there would be considerable losses to satisfy soil moisture deficits and depression storage. Thus there would be less rainfall excess available for runoff and thus a reduced peak flow. HMR-57 notes that there is very little information in the region available to characterize this type of storm particularly in Canada where there are fewer recording rain gauges. Given the uncertainty inherent in characterizing a local storm and the large losses that would certainly occur, it was decided to base the PMP/PMF calculations using the conventional approach, which is based on a regional winter storm.

To verify this conclusion, a local storm with a one-hour precipitation value of 60 mm was input to the model. A CN value of 93 with the local storm produced the same outflow from the catchment as the winter storm and a CN value of 95. The CN value for characterizing soil conditions in the summer months would be expected to be much lower than 93. Therefore it was concluded that the conventional approach for calculating the PMP/PMF was appropriate.

The winter storm produces a higher peak flow than the summer storm even though the one-hour rainfall is less. This is because the 24-hour rainfall depth in the summer is much less than the winter storm, which contributes to the wet antecedent conditions and causes an increase of flow in the watershed prior to the occurrence of the peak rainfall intensity.

The 24-hour PMP for Chase River Watershed was compared with the 24-hour PMP previously derived for Jump Creek Watershed. Klohn Leonoff, 1990 estimated the 24-hour PMP for Jump Creek Watershed to be 301 mm. Charles Howard and Associates, 1997 estimated the 24-hour PMP to be 310 mm. The Chase River Watershed would be expected to have a lower PMP, as it is located at a lower elevation than Jump Creek Watershed. Therefore the 24-hour PMP of 237 mm calculated for Chase River Watershed in this study is consistent with these previous studies.

It is not considered appropriate to assign a return period to the PMF. This is because it is intended to represent an upper bound to potential flood flow. In other words, the probability that the absolute maximum flow will be less than or equal to the PMF is 1.0.

4.3 Flood Frequency Analysis

The PMF is significantly larger than the capacities of the spillways on the Chase River Dams. To investigate the magnitude of a lesser flood than the PMF, a regional flood frequency analysis was carried out to estimate the 1000-year flood. Five gauging stations on the east slope of Vancouver Island with records of annual instantaneous flood peaks were identified. As these drainage basins were all larger than Chase River, Mackay Creek in North Vancouver was included in the analysis. For each station, a flood frequency relationship was established using the flood frequency analysis computer program FFAME developed by the former Ministry of Environment Lands and Parks. Table 4.1 shows the results of the analysis for each gauging station.

Table 4.1: Results of regional flood frequency analysis

| Station ID | Station Name | Area (km ²) | 1000 yr Flood (m ³ /s) |
|------------|--------------------------------------|-------------------------|-----------------------------------|
| 08GA061 | Mackay Creek at Mont Royal Boulevard | 3.63 | 22.5 |
| 08HB002 | Englishman River near Parksville | 324 | 651 |
| 08HB011 | Tsolum River near Courtenay | 258 | 291 |
| 08HB025 | Browns River near Courtenay | 86 | 267 |
| 08HB075 | Dove Creek near the mouth | 41.1 | 74 |
| 08HD011 | Oyster River below Woodhus Creek | 298 | 389 |

The data from Table 4.1 were plotted as shown in Figure 4.4 to provide an estimate of the 1000-year flood for the catchment area above the Lower Chase River Dam. For the catchment area of 20 km², the 1000-year flood was estimated from Figure 4.4 to be 68 m³/s. This indicates that the 1000-year flood is about 34% of the PMF.

5 MITIGATION

5.1 Mitigation Options

The maximum capacity of the Middle Chase River Dam with zero freeboard on the dam was determined in Section 3 to be 62 m³/s. The maximum capacity of the Lower Chase River Dam spillway was determined to be 25 m³/s due to the limited capacity of the spillway chute. With zero freeboard on the dam and overtopping of the chute wing walls, the capacity is 35 m³/s.

To accommodate the PMF the required total spillway capacity for the Middle Chase River dam is 192 m³/s and for the Lower Chase River Dam the required capacity is 198 m³/s. These are very large increases above the current capacities and reflect the age of the dams. At the time of construction, around 1910, the PMF concept was not considered for dam spillways. The spillways were probably designed for the 100-year flood but lack of data at the time almost certainly resulted in the spillways being under-designed even for that criterion.

The following options could be considered to resolve this issue:

1. **Decommission the dams.** This would be costly and challenging. There would be environmental issues to overcome with draining the reservoirs and there would be loss of a valuable city amenity. However, a seismic assessment may conclude that decommissioning the dams is a required remedial action.
2. **Carry out an incremental downstream damage assessment.** To be classified as a high hazard dam the incremental damage over and above a PMF should be significant. In other words, the PMF would do considerable damage downstream anyway and the additional failure of the dams may not result in much more damage. The incremental damage is defined as the damage that would occur with a PMF plus dam failure minus the damage that would occur with a PMF without dam failure. However, if it were demonstrated that the incremental damage was minor, it would still be necessary to upgrade the spillways to at least the 1000-year flood. Otherwise catastrophic failure could occur with a relatively frequent flood event. The Middle Chase River Dam spillway is close to being able to pass the 1000-year flood. The Lower Chase River Dam spillway capacity would have to be significantly increased.

3. **Modify the existing spillways.** To pass a flow of $192 \text{ m}^3/\text{s}$, a spillway crest width of at least 50 m would be required which would essentially mean most of the dams being replaced. This would involve construction of coffer dams in the reservoirs, dismantling the existing dams and construction of concrete gravity dams acting as overflow structures. It would be less costly to decommission the dams.
4. **Construct an emergency overflow spillway in a new location.** The advantage of this option is that the existing dams and spillways can be left unchanged with perhaps only minor modifications. This option will thus have the least impact on the park in the vicinity of the dams and reservoirs (which are a major attraction) and the construction challenges will be less than options involving major modifications to old dam structures. This option was designed and costed at a conceptual level for this report.

5.2 Emergency spillway (Option 4)

The proposed emergency spillway to convey the PMF would be located in the right abutment of the Middle Chase River Dam. It would have an ogee crest with a length of 60 m leading to a rock-cut spillway channel about 500 m long by-passing the Lower Chase Reservoir. The spillway would discharge into the canyon below the Lower Chase River Dam. Figure 5.1 is a general layout of the proposed spillway. It was assumed that the side slopes would be vertical.

The emergency spillway was designed to convey the additional flow over and above the flow that can be carried by the existing spillways. The invert elevation of the spillway crest was set at 86.4 m, 0.2 m above the invert of the Middle Chase River Dam spillway. At the peak of the PMF the inflow to the Middle Chase reservoir is about $192 \text{ m}^3/\text{s}$. After a small amount of attenuation in the peak, due to storage effects in the reservoir, the emergency spillway would carry about $159 \text{ m}^3/\text{s}$ and the existing spillway would convey just over $32 \text{ m}^3/\text{s}$. The spillway channel would range between 3 and 5.5 m deep. Following the slope of the existing ground surface the spillway channel slopes would vary from 12 to 2%.

The width of the spillway was designed to pass the peak of the PMF with a water depth of 2 m through most of the channel. In the reach adjacent to the Lower Chase reservoir, however, it would be necessary to excavate the channel deeper at the downstream end to generate a grade of 2%. Since the depth of the channel is much greater in this section it was possible to reduce the width increasing the water depth to 3 m and matching the width of the channel in the lowest reach. Figure 5.1 is a plan view of the proposed spillway, which shows the variations in spillway width. A profile of the existing ground surface with the proposed spillway invert is illustrated in Figure 5.2.

It is necessary to minimize the discharge flowing into the Lower Chase Reservoir due to the limiting capacity of the lower spillway. Accounting for flow from the Middle Chase Reservoir, additional watershed runoff, and a limited amount of attenuation in the reservoir, the peak discharge through the Lower Chase spillway for the PMF is $34 \text{ m}^3/\text{s}$. As discussed in Section 3.2, the capacity of the lower spillway is only $25 \text{ m}^3/\text{s}$ due to the low sidewalls of the channel. However, raising the sidewalls downstream of the bridge by 0.25 m to 1 m would allow the spillway to accommodate $34 \text{ m}^3/\text{s}$ and pass the PMF. Alternatively the spillway could be widened. The water level would be close to the dam crest so raising the crest slightly to

increase freeboard should be considered. For a small reservoir, however, with low exposure to wind generated waves and a short duration of the PMF peak, zero freeboard for the PMF would be considered acceptable.

The emergency spillway invert at the Middle Chase Reservoir could be set at a lower elevation increasing the capacity and thereby further reducing discharge through the existing spillway to the Lower Chase Reservoir. However, to retain the existing water level in the reservoir, the crest would have to comprise a fail-safe system such as an erodible fuse plug crest, automatic gates that would collapse at a given water level or a rubber dam that would deflate. While such options could be considered at a feasibility design stage, there would be a concern with potential vandalism in such a public area. Therefore for the conceptual design study the design was developed assuming a simple fool-proof fixed crest.

A budget construction cost for the emergency spillway was developed based on the following assumptions:

- Overburden depth averaging 0.5 m
- Competent rock excavation requiring drilling and blasting
- Fencing required on both sides of the rock cut

The major elements of the work would be:

- Clearing and grubbing of the alignment
- Excavation and disposal of the overburden
- Excavation and disposal of approximately 30,000 m³ of rock
- Construction of a 60 m long ogee crest spillway intake at Middle Chase Reservoir
- Raising of the wing walls of the Lower Chase spillway chute
- Construction of fencing along both sides of the spillway channel

The construction cost for the emergency spillway shown in Figure 5.1 was estimated to be about \$750,000. This cost is very sensitive to the assumed rock quality and rock excavation costs, which were assumed to be \$20/m³. Allowing for 15% engineering, 10% taxes and administration and 20% contingency, the budget cost estimate is approximately \$1,000,000. A breakdown of the costs is provided in Appendix 1.

Given the high construction cost of the proposed emergency spillway, it would be worthwhile considering alternatives at the next stage of design. Depending on the foundation conditions along the alignment, a lower cost alternative may be construction of two emergency spillways, one for each dam. The Middle Chase River Dam emergency spillway would discharge into the Lower Chase River Reservoir. A second spillway would be excavated in the right abutment of the Lower Chase River Dam. To compare costs, detailed site surveys and foundation conditions along the proposed alignments would be required.

6 CONCLUSIONS

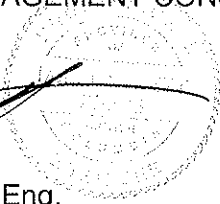
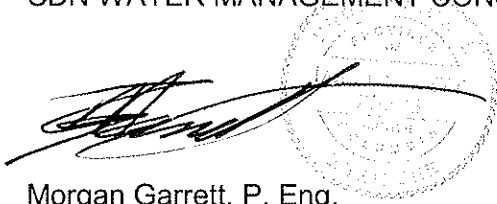
1. The Middle Chase River Dam spillway has a maximum discharge capacity of $62 \text{ m}^3/\text{s}$ with zero freeboard on the dam.
2. The Lower Chase River Dam spillway has a maximum discharge capacity of $25 \text{ m}^3/\text{s}$. This capacity is limited by the height of the spillway chute wing walls. If the wing walls are raised, the maximum capacity would be $35 \text{ m}^3/\text{s}$ with zero freeboard on the dam.
3. The Probable Maximum Precipitation (PMP) on the Chase River Watershed was estimated to be 237 mm in 24 hours based on a regional winter storm. The one hour PMP would be 40 mm based on a Soil Conservation Service Type 1A distribution. A summer local storm PMP occurring in the warm summer months was estimated to be 60 mm in one hour.
4. The Probable Maximum flood (PMF) from the winter storm would have a peak inflow to the Lower Chase Reservoir of $198 \text{ m}^3/\text{s}$ with moist antecedent conditions. A summer PMP storm would produce a lower peak flow because of drier soil moisture conditions in the watershed and a lower 24-hour rainfall amount.
5. The PMF is significantly larger than the current spillway capacities on both dams. Therefore major construction works would be required to pass the PMF.
6. The 1000-year flood for Lower Chase River Dam was estimated to be $68 \text{ m}^3/\text{s}$ which is about 34% of the PMF. An incremental damage assessment would likely show that the spillways would have to convey a flow of at least this magnitude.
7. An emergency spillway to convey the PMF could be constructed in the right abutment of the Middle Chase River Dam bypassing the Lower Chase River Dam. The advantage of this option is that there would be little construction work required on the old dams. The budget construction cost estimate is approximately \$1,000,000.
8. Other options that could be considered including decommissioning the dams or replacing the dams with concrete overflow structures.

7 RECOMMENDATIONS

1. It is recommended that the next step comprise an incremental damage assessment to determine whether the Chase River Dams are high hazard and therefore would be required to pass the PMF.
2. In the event that the dams are classified as high hazard as a result of the incremental damage assessment, it is recommended that a preliminary design level study be carried out to assess spillway alternatives and to compare costs. The study should consider the following options:
 - Replacing the dams with concrete overflow structures.
 - Constructing two spillways, an emergency spillway for each dam.
 - A single emergency spillway (as proposed in this report)
 - Dam decommissioning

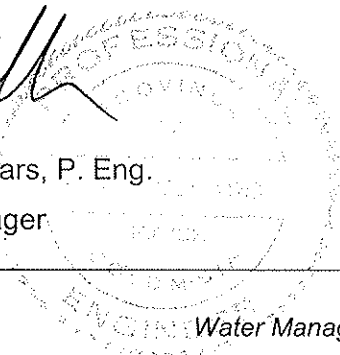
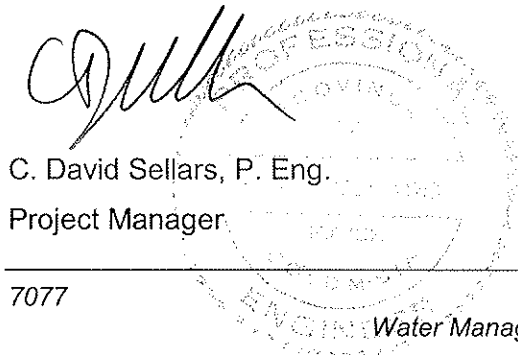
The study should include detailed site surveys and investigation of foundation conditions to provide a basis for comparative costing. The seismic stability of the two dams should also be considered, as the results would have a bearing on the mitigation option selected.

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FIGURES

Figure 2.1 - Chase River Watershed

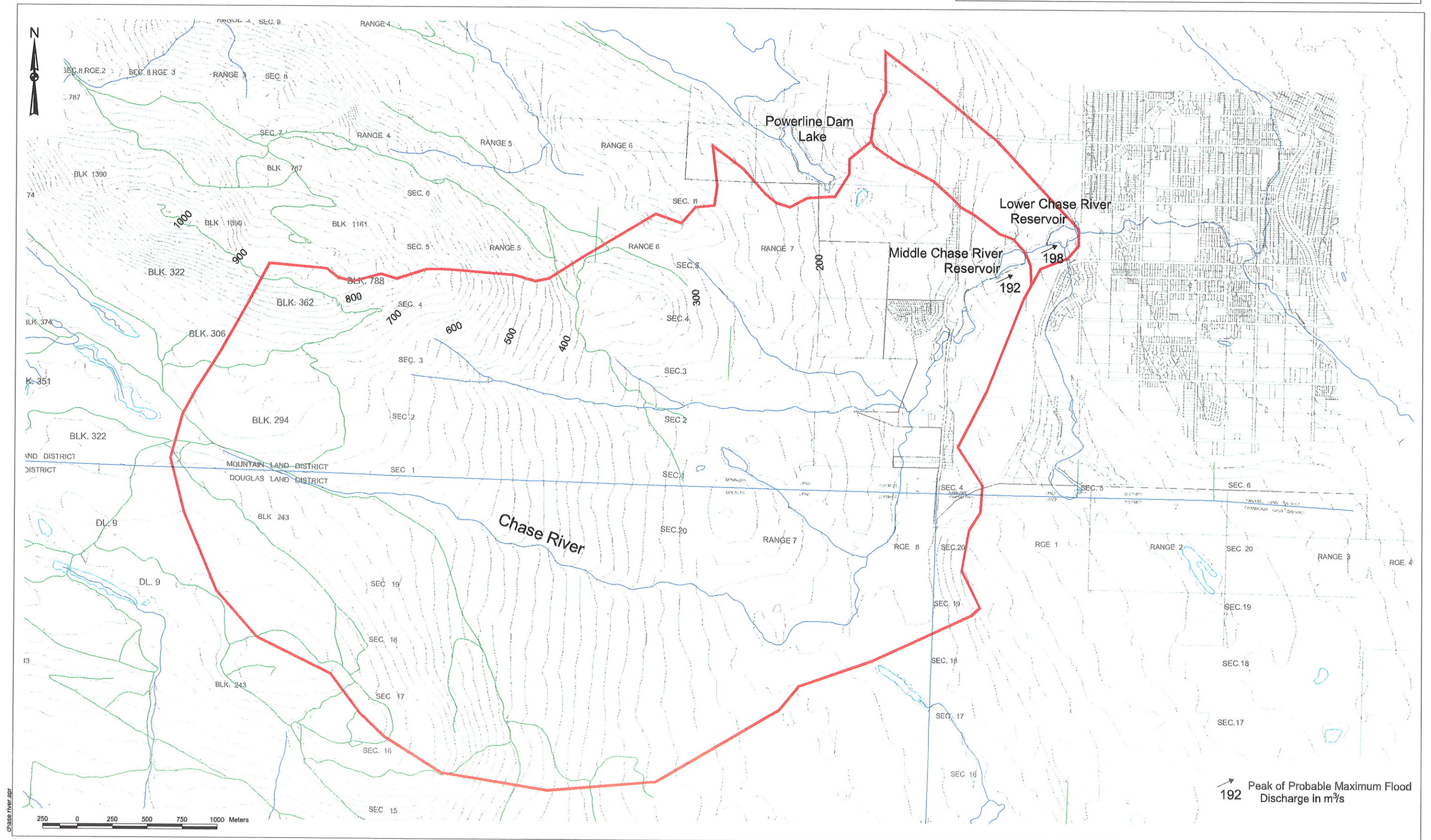


Figure 3.1a - Lower Chase River Dam Spillway Water Surface Profile
Discharge 5 m³/s

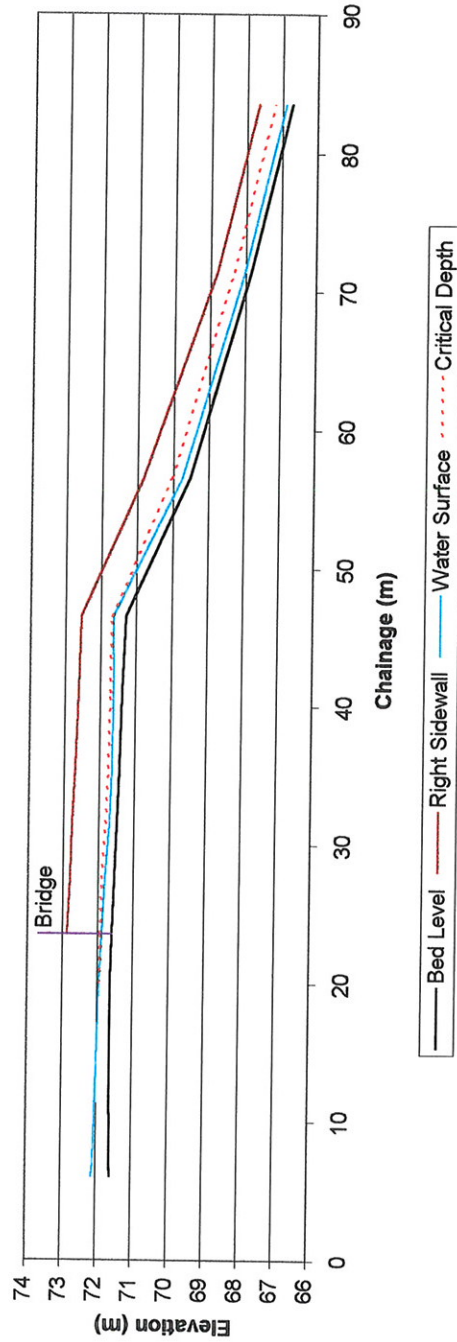


Figure 3.1b - Lower Chase River Dam Spillway Water Surface Profile
Discharge 35 m³/s

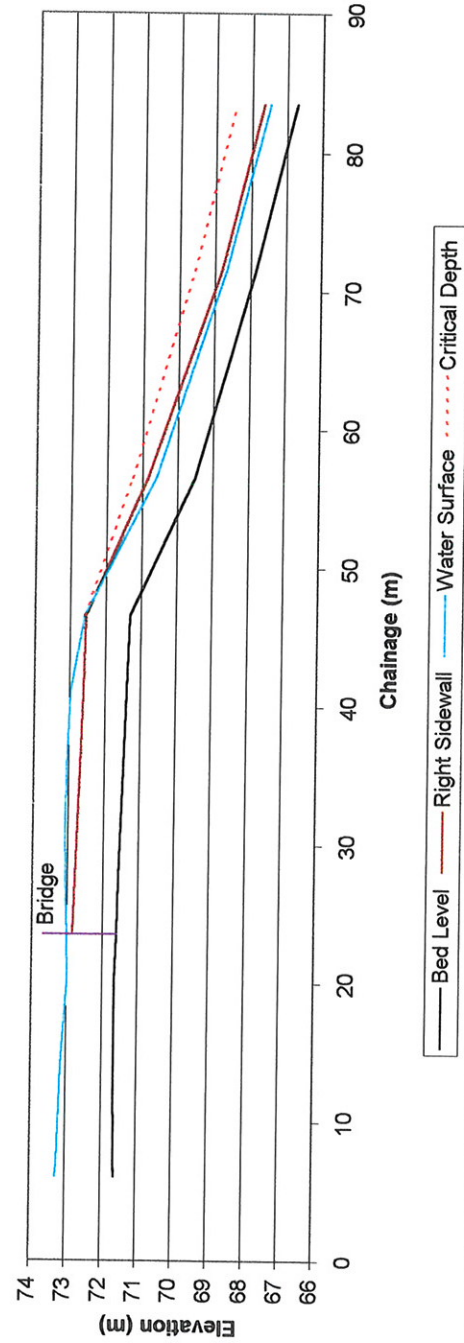


Figure 4.1 - Distribution of 24-Hour
Probable Maximum Precipitation

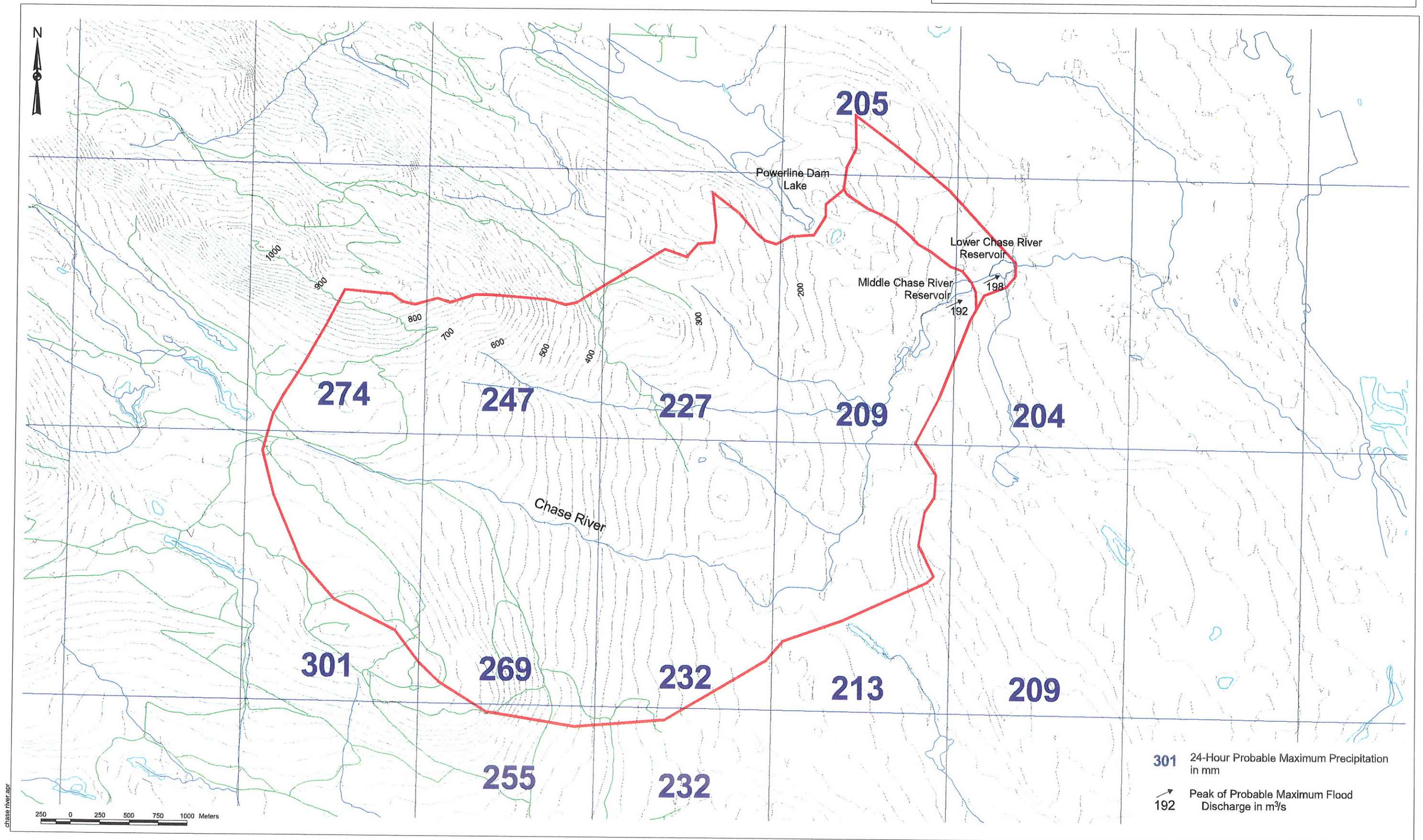


Figure 4.2 - Distribution Graph for SCS Type 1A Storm

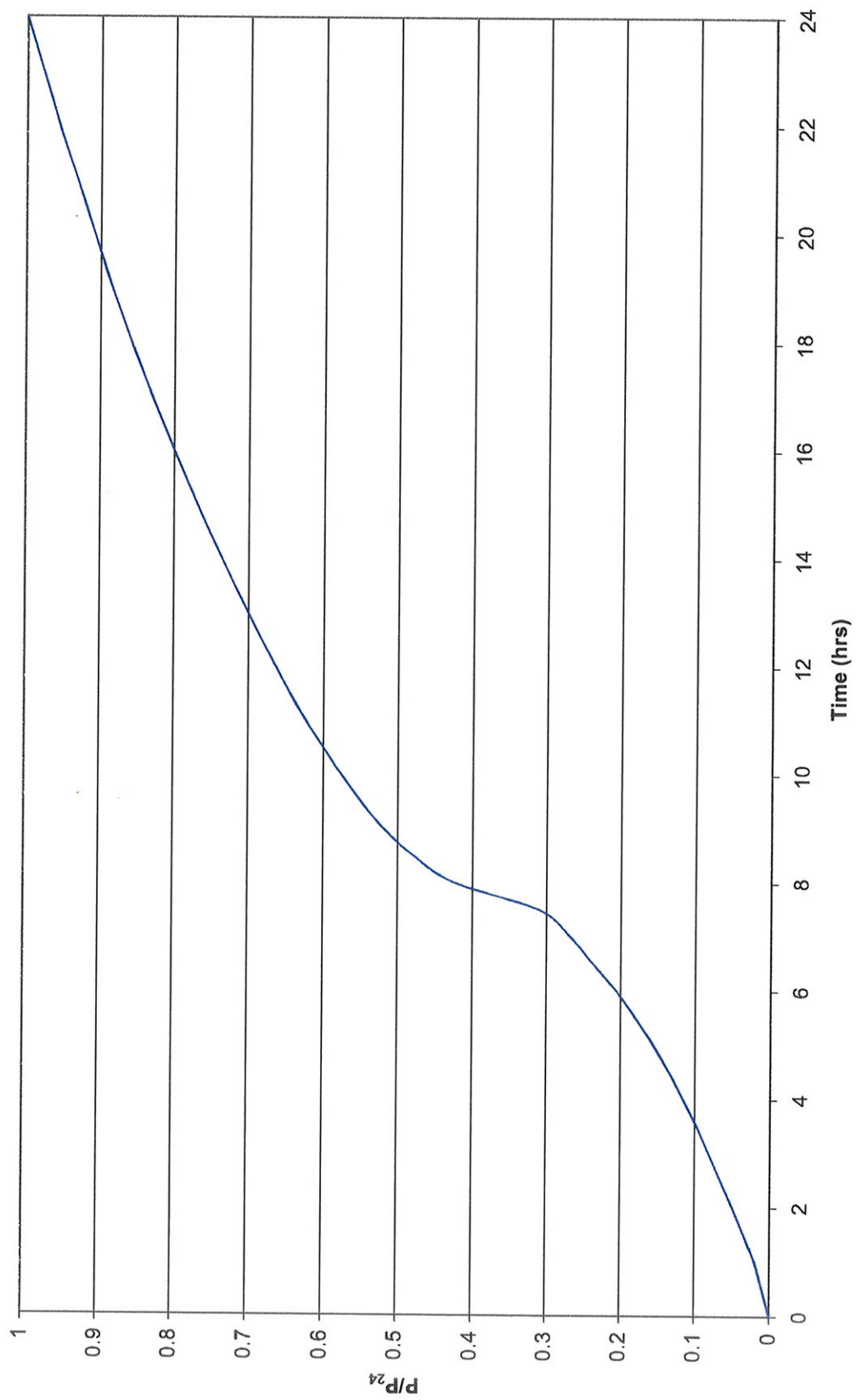


Figure 4.3 - Probable Maximum Flood

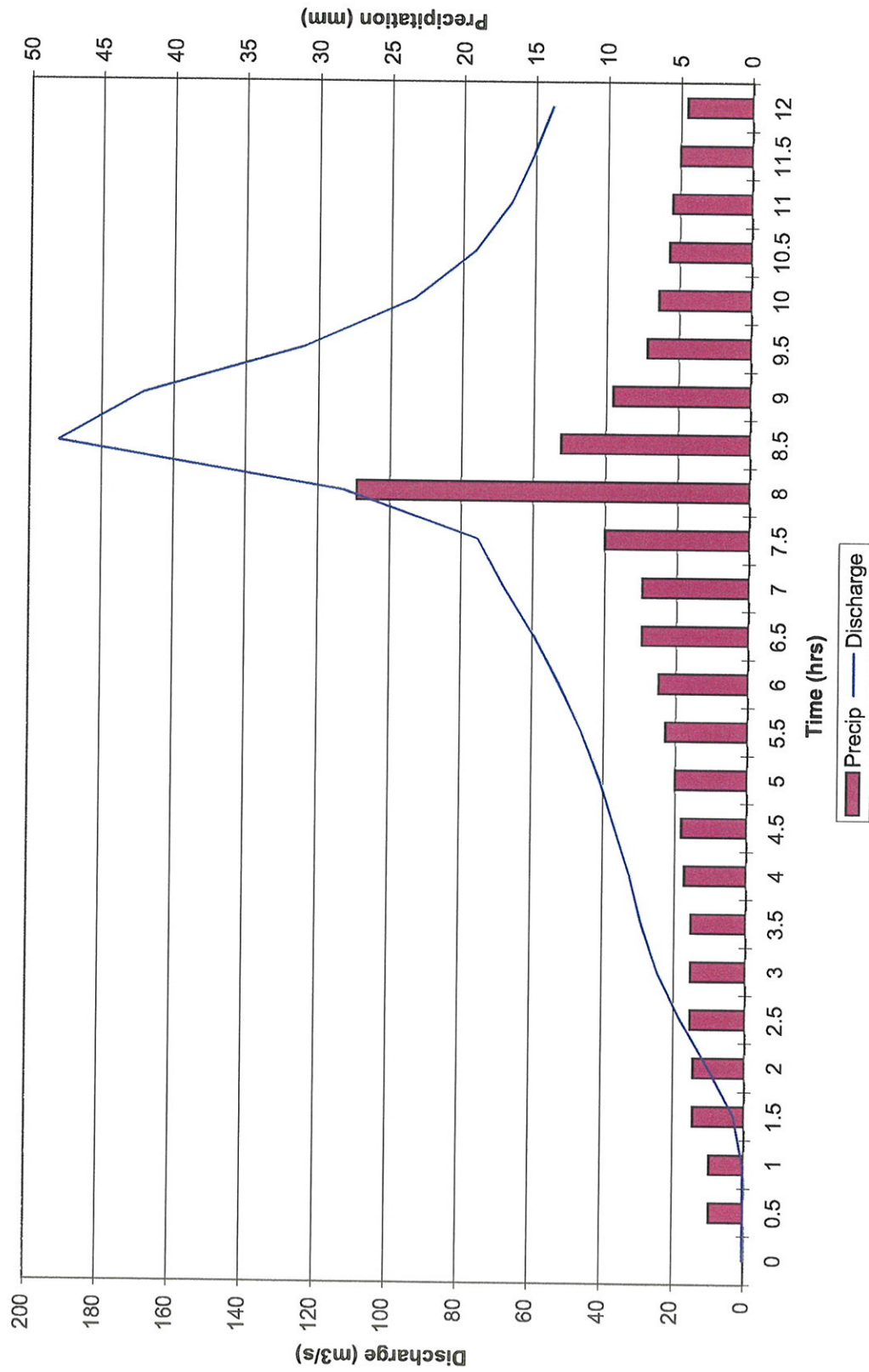


Figure 4.4 - Estimates of 1000-Year Flood
by Drainage Area

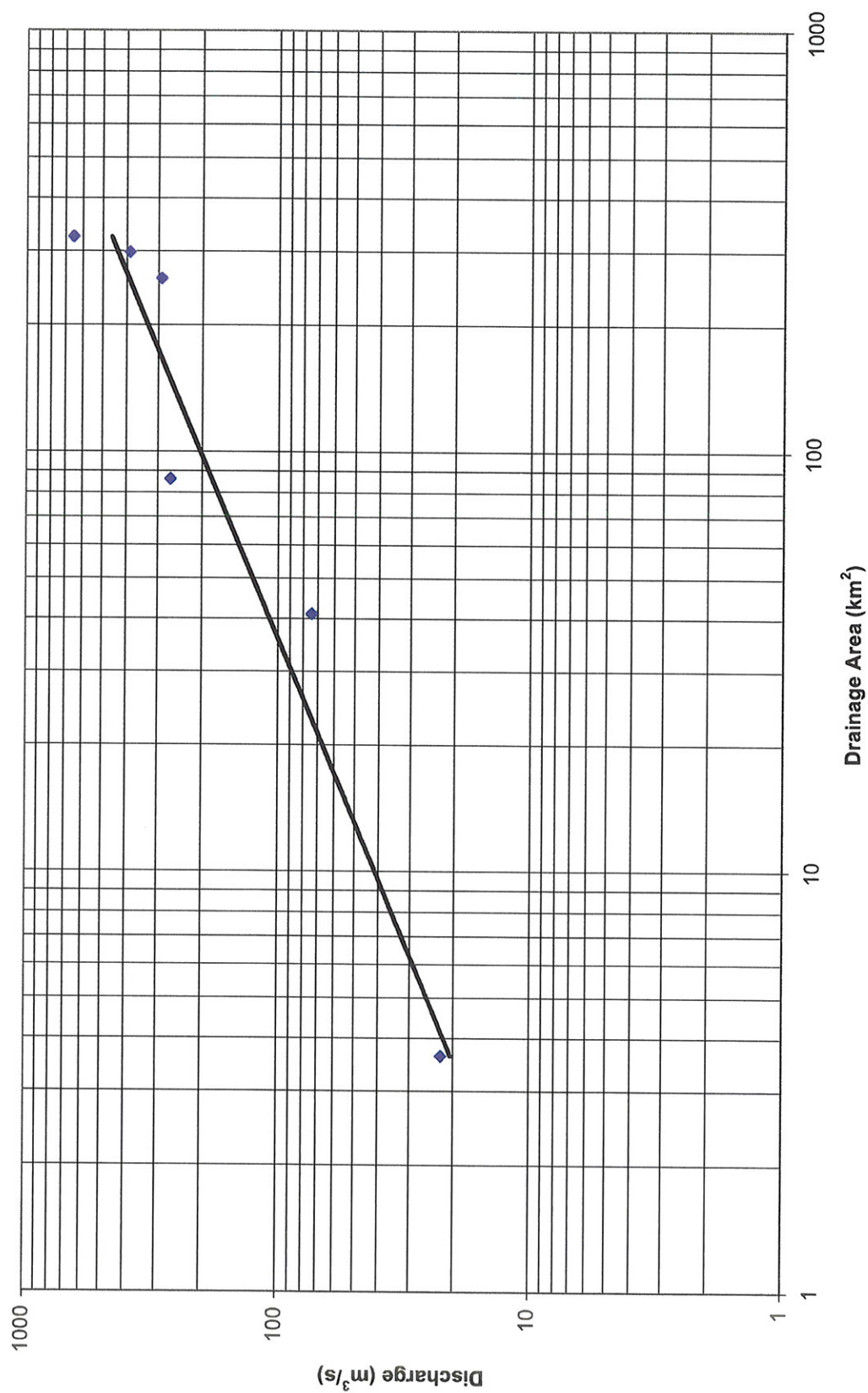


Figure 5.1 - Proposed Emergency Spillway Plan

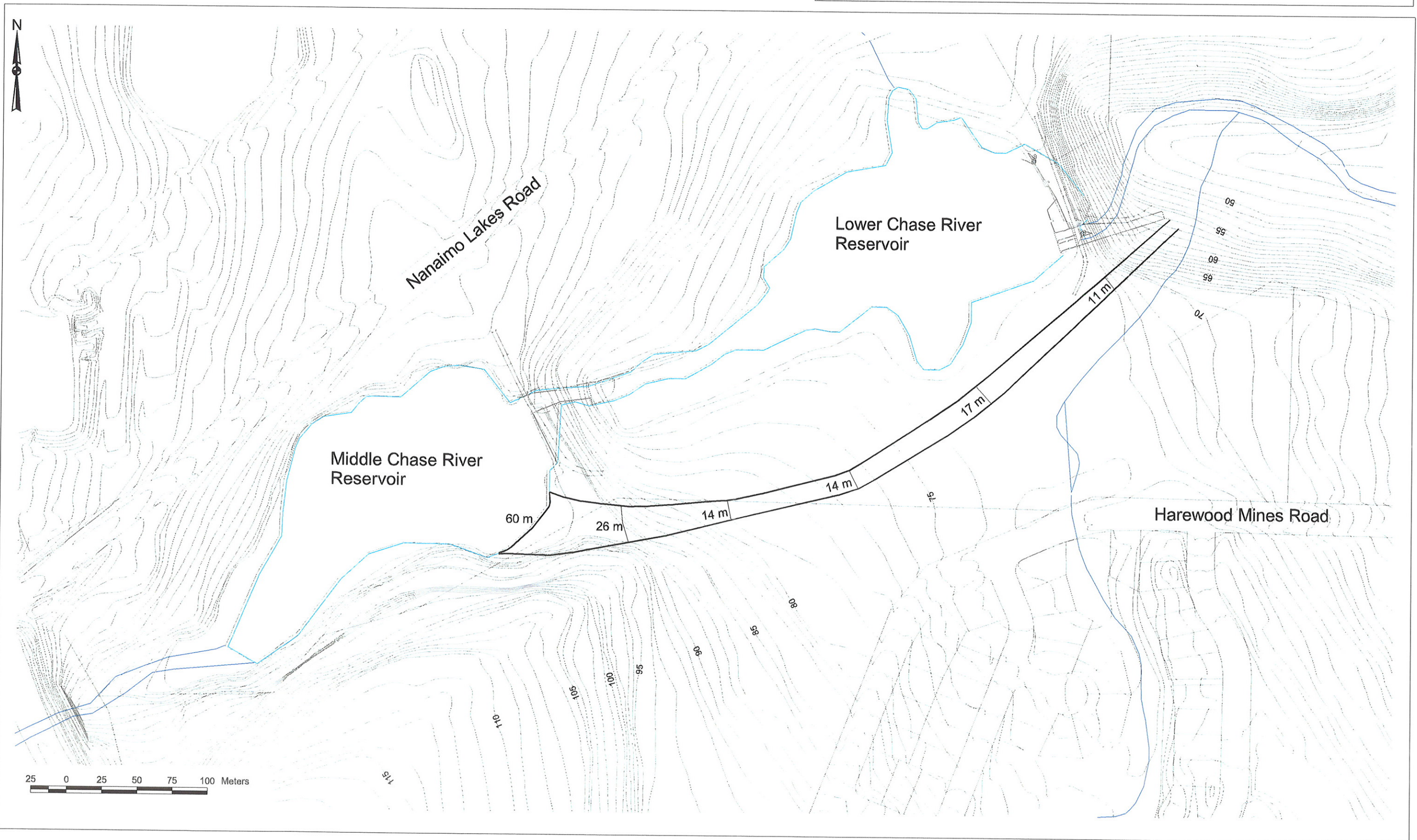
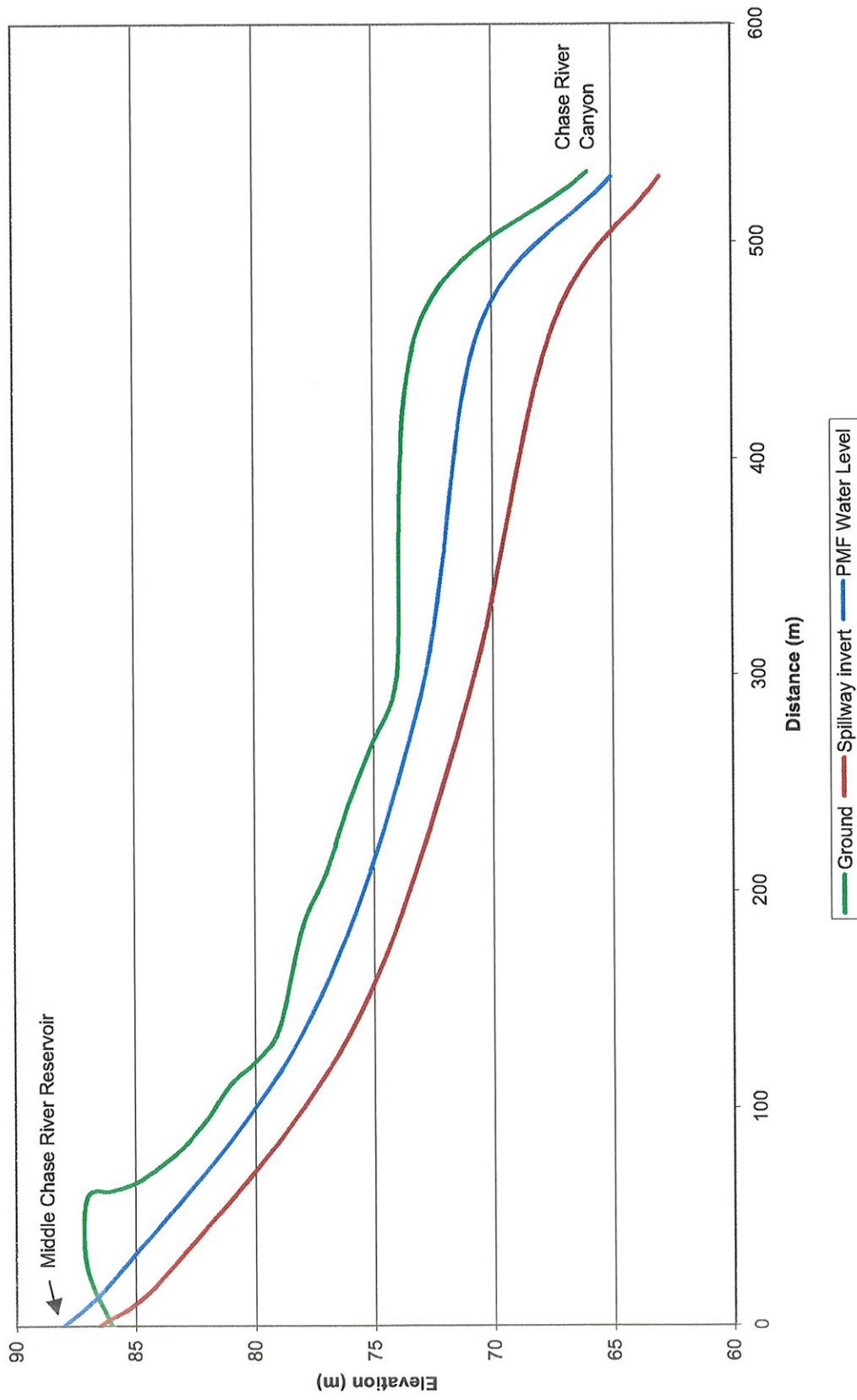


Figure 5.2 - Proposed Emergency Spillway Profile



PHOTOS



Photo 1 - Middle Chase River Dam spillway looking upstream

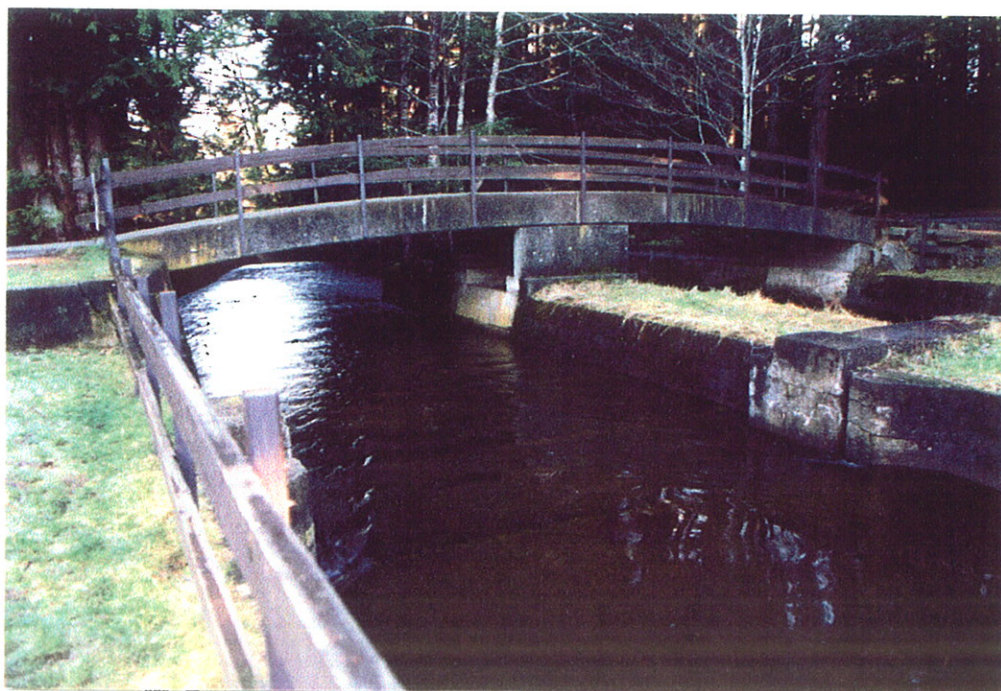


Photo 2 - Lower Chase River Dam spillway looking downstream



Photo 3 - Lower Chase River Dam upper section of spillway looking downstream



Photo 4 - Lower Chase River Dam spillway chute looking upstream

APPENDIX 1

Budget Cost Estimate for Emergency Spillway (Option4)

| Chase River Dams Spillway Hydrology Study | | | | |
|--|----------|----------------|------------|---------------------|
| Budget Cost Estimate Breakdown for Emergency Spillway (Option 4) | | | | |
| | Quantity | Unit | Unit Price | Extension |
| CAPITAL COSTS | | | | |
| Emergency spillway | | | | |
| Concrete crest | 25 | m ³ | \$ 1,500 | \$ 37,500 |
| Till excavation and disposal | 5,000 | m ³ | \$ 10 | \$ 50,000 |
| Rock excavation and disposal | 30,000 | m ³ | \$ 20 | \$ 600,000 |
| Fencing | 1,000 | m | \$ 10 | \$ 10,000 |
| Lower Chase spillway | | | | |
| Raise wing walls | 30 | m ³ | \$ 1,500 | \$ 45,000 |
| Subtotal , Capital Cost Estimate | | | | \$ 742,500 |
| Engineering 15%, Contingency 20%, Taxes/admin 10% = 45% | | | | \$ 334,125 |
| TOTAL CAPITAL COST | | | | \$ 1,076,625 |