REPORT ON

MIDDLE CHASE RIVER DAM
2003 DAM SAFETY REVIEW

Submitted to:

City of Nanaimo
455 Wallace Street
Nanaimo, B.C. V9R 5J6

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March, 2004 03-1411-103
EXECUTIVE SUMMARY

This report presents an assessment of the Dam Safety Review (DSR) of the Middle Chase River water-retaining dam. The assessment included review of instrumentation data, as well as a site inspection. The findings are as follows.

DAM SAFETY

Condition of Dam

- The dam is a HIGH consequence dam;
- The dam appears to be in comparable condition to its last major inspection in 1992;
- Although seepage is somewhat larger than might be expected from the dam’s estimated internal configuration, implying the existence of some internal defects, there is no evidence of dam instability under normal operating conditions;
- The hydrology of the catchment has been determined as has the spillway capacity. Because of changing standards, the spillway cannot pass the appropriate design storm for the dam’s current hazard rating;
- No detailed seismic stability assessment has been carried out. However, there is substantial (ie 95%) confidence that the dam withstood a 30-year return period earthquake prior to stability improvements with remediation of the downstream shell in 1980. Further work is needed to determine the dam’s adequacy under ground motions appropriate to the hazard rating and which are about ten times greater than those motions experienced to date; and
- An offtake pipe was believed to pass through the dam when remediation was undertaken in 1980 but was never located despite intensive efforts. This pipe represents an uncertain but possibly real hazard to the dam if it exists.

Operations and Maintenance of Dam

- Recommendations from the 1992 dam safety inspection have largely been carried out in regard to maintenance issue;
- Maintenance of the dam has been conscientious;
- Weir measurements to verify the ongoing adequacy of the dam have been taken at appropriate intervals and reviewed in a timely manner; and
• Resolution of issues associated with bringing the dam into line with current safety standards for extreme events (storm, earthquake) has started, but work remains to be accomplished.

Emergency Preparedness

• The emergency preparedness plan (EPP) has been recently updated and is appropriate.

Conclusion

• There are no present Actual Deficiencies at Middle Chase Dam;

• There are eight instances of Potential Deficiencies and which relate to dam performance under extreme storms or earthquakes; and

• There are four present Non-Conformances with regard to accepted dam safety principles.

RECOMMENDATIONS

Work is recommended to:

• Automate reading of the V-notch weir and incorporate measurements into the City’s real time monitoring (SCADA) system. Store weir flow data as ASCII files for future analysis.

Weir flow is a crucial validation of ongoing dam safety given the uncertainty over details of the dam’s construction and tracking this should give advance warning of a deteriorating situation. An automated system is low-cost and will substantially enhance public safety. It ought to be implemented within a year, and preferably sooner.

Part of the reasons for the findings in this DSR is that the dam is nearly 100 years old and was likely never formally designed. However, societal expectations have changed and old dam’s are required to be comparable to new dams in terms of safety for potentially affected people downstream. In the longer term, say within the next three years, the dam needs to be brought up to date from its current design of 1 in 100 year rare event (storm, earthquake) capability to something better than a 1 in 1000 year standard. Further engineering studies, and likely some construction upgrading are required. Specifically: 
Either a dam raise or spillway modification (or combination) is needed so that the dam can safely pass a 3000 year return period storm. The exact design criteria needs to be established with the Water Comptroller, but may require about a 30% increase in spillway capacity; and

The seismic resistance of the dam is possibly adequate (depending on the actual configuration of the upstream shell) and but needs formal evaluation and documenting, in particular with attention to concrete core wall cracking and stability during shell movement. It is not believed that any physical works are required at this time, but the extent of any upgrading must be defined by further studies.

The above three recommendations are the principal items requiring attention, and the thirteen action items identified in Appendix A relate to aspect of these three recommendations. Addressing these actions items will bring the City’s management of Middle Chase dam into conformance with current standards of dam safety. A detailed explanation and expansion of these recommendations is given in the report.

This DSR has been carried out in accordance with the recommendations of the Canadian Dams Association and in compliance with the Water Act of the Province of British Columbia, BC Reg 44/2000. It was carried out by H. Hawson, P. Eng and M. Jefferies, P.Eng, over the period July-November, 2003.
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Appendix A  Detailed Results of Dam Safety Review
Appendix B  Dam Inspection Report, July 2003
Appendix C  Investigation of Right Abutment seepage in 1993
1.0 INTRODUCTION

1.1 Purpose

This report presents a Dam Safety Review (DSR) of the safety of the Middle Chase River dam, and was carried out for the City of Nanaimo (City) who own the dam. The dam lies in the south part of the city, see Figure 1.1 for location, and is accessed by way of Nanaimo Lakes Road. The dam has also been known by the former names Upper Harewood Dam and Upper Colliery Dam.

Under the British Columbia Provincial Water Act, it is the responsibility of dam owners to ensure that dams and appurtenant works are inspected to evaluate compliance with acceptable standards of public safety. In practice for a dam like Middle Chase this takes the form of a DSR about every seven or so years. The last review of the dam was in 1992 by EBA. No unusual events triggered the present inspection.

It is understood that this report will be forwarded to the Water Management Branch of Land and Water British Columbia Inc. in compliance with the City’s responsibilities under its dam license, #61424 (dated 27 September 1985).

1.2 Background

Although constructed in 1910 for water supply to the then Harewood Colliery, it is understood that Middle Chase River dam was decommissioned in about 1945 and the lake is now used solely as a recreational resource for the city. A layout of the dam and its lake is provided as Figure 1-2.

Middle Chase River dam is an earthfill structure some 50 m in length and with a maximum height of 12.5 m. Key dam statistics and data are summarized on Table 1.1, while Photograph 1.1 shows a view of the dam. The spillway for the impounded lake is located at the junction of the dam with the left abutment, and is founded on rock. The spillway is designed to spill water under operating conditions, not only during extreme events, and is uncontrolled in that there are no mechanical gates, sluices or stop-logs used to control the water level. The spillway discharges into a lined channel with a bedrock base, illustrated on Figure 1.2.
Middle Chase dam appears to have been an engineered structure when constructed nearly a hundred years ago, but no records of its design and construction survive. Accordingly, part of this assessment includes estimates of the dam’s expected performance. These estimates are compared to the dam’s actual behaviour for various potential failure modes defined in the ICOLD study of dam safety. This then provides the context for the DSR.
Note: the offtake pipe is shown in the location it was believed to exist in 1978, but that it could not be found in the downstream shell during the 1980 remediation (discussed later).

Figure 1.2: Plan of Middle Chase Dam – NTS

Photograph 1-1: View Middle Chase River Dam 24 July 03 (composite photo) from upstream
Table 1-1: Middle Chase River Dam Statistics

<table>
<thead>
<tr>
<th>Type of Dam</th>
<th>Concrete core, rockfill/granular shell</th>
<th>1 filter zone on downstream side</th>
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<tr>
<td>Maximum Height</td>
<td>12.5 m</td>
<td></td>
</tr>
<tr>
<td>Width of Crest</td>
<td>5 m approximately</td>
<td></td>
</tr>
<tr>
<td>Length along Crest</td>
<td>50 m</td>
<td></td>
</tr>
<tr>
<td>Catchment Area</td>
<td>2600 Ha approximately</td>
<td></td>
</tr>
<tr>
<td>Upstream Sideslope</td>
<td>About 1V:1H, but no detailed survey data</td>
<td></td>
</tr>
<tr>
<td>Downstream Sideslope</td>
<td>2.0 H:1V nominal</td>
<td></td>
</tr>
<tr>
<td>Retained Water (normal pool)</td>
<td>93,000 m³</td>
<td></td>
</tr>
<tr>
<td>Normal Water Level</td>
<td>86.4 m</td>
<td></td>
</tr>
<tr>
<td>Freeboard, 1:100 year storm</td>
<td>About 0.3 m (midway along crest)</td>
<td></td>
</tr>
<tr>
<td>Offtake works</td>
<td>None working; historical system may be under right part of</td>
<td>dam and unplugged</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type of Spillway</td>
<td>Unregulated spillway designed to discharge under normal</td>
<td></td>
</tr>
<tr>
<td></td>
<td>operating conditions</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete walled channel founded on bedrock</td>
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1.3 History of Dam Safety Activities

Since acquisition of the dam by the City in 1975, there have been five principal safety related activities:

- 1978-80 Testing of the dam’s condition followed by remedial works;
- 1987 Development of a dam “Data File”;
- 1992 A first DSR;
- 1992 Development of a “Data Book” which included Operations & Maintenance requirements together with an emergency response plan;
- 1993 Investigation of concentrated Right Abutment seepage; and
- 2002 Hydrology study for the Chase River

The City has also undertaken monthly inspections of the dam using their own staff and which are documented as paper records held in files at the Water Supply Office (Public Works Yard, 2020 Labieux Road, Nanaimo, BC, V9T 6J9).

Inundation estimates in the event of dam failure are essentially based on modelling undertaken by the Water Management Branch and sent to the City in 1977 as a map of potential flooded land. This study has recently been supplemented by further estimates as part of the 2002-3 hydrology studies.

1.4 Authorization

Golder Associates Ltd were retained for this DSR by the City of Nanaimo by letter dated July 07, 2003 based on our proposal P32-1198 dated May 26, 2003.

2.0 SAFETY REVIEW PROCESS

2.1 Standard of Assessment

This DSR was carried out in accordance with BC Dam safety Regulation (BC Reg 44/00) and follows the Dam Safety Guidelines published by the Canadian Dam Association.

A comprehensive list of dam safety requirements has been developed by BC Hydro for DSR audits, and these requirements have been used as the basic checklist for this DSR.

2.2 Methodology

A DSR is a means for a dam owner to determine compliance with requirements, to improve the understanding of existing risks and identify opportunities for risk reduction.
This assessment is based on a review of existing reports and drawings provided by the City, a site inspection, and a review of data from existing monitoring instrumentation. Further, approximate calculations or other estimates have been made to provide a context for the review as there is no design and construction report. Archives and books on Nanaimo have been searched for historic information, although little was discovered other than a brief entry recording the use of the dam for water supply in 1915. Finally, considerable useful information has been provided by the Dams’ Branch in Victoria from the records of their various inspectors during the 1975-85 period.

The completed DSR pro-forma of the Canadian Dam Association is included in Appendix A.

A comprehensive detailed site inspection was completed by H. Hawson and M. Jefferies of Golder Associates Ltd. on July 23-24, 2003. A further site visit was made by H. Hawson on October 20, 2003. The reports are included as Appendix B.

Areas inspected include:

- Dam;
- Spillway; and
- Reservoir slopes.

The instrumentation data reviewed were collected from a V-notch weir installed it is believed in 1995, although regular records were only available from 1998.

The dam is in an active seismic zone, but has only experienced quite small ground motion in the past. We have estimated the maximum ground acceleration experienced by the dam to date.

Because the dam is a historic structure for which no detailed records survive, we have also estimated the expected performance of the dam. This is in part based on prior stability calculations (Golder Associates, 1978b), part on prior hydraulic studies (Willis, Cunliffe & Tait, 1978; Dayton & Knight, 1987; and Water Management Consultants, 2002) and part on new calculations.

2.3 Deficiencies and Non-Conformances

During the review, the dam safety requirements assessed as not being met were assigned with a particular deficiency or non-conformance type in accordance with definitions in BC Hydro’s dam safety procedures as follows.
Actual deficiency: An unacceptable dam performance condition which has been confirmed based on BC Hydro’s dam safety standards and criteria. There are two types of Actual Deficiencies: under normal loads expected during the lifetime of the dam (denoted as $An$) and under unlikely loads that are not expected to occur (denoted as $Au$).

Potential deficiency: A potentially unacceptable dam performance condition which has not yet been confirmed. Potential deficiencies are separated into those which, on more detailed investigation, are expected to be confirmed as actual deficiencies and those that are not (including those where it may not be possible to demonstrate that they are not deficiencies). There are four types of Potential Deficiencies:

$Pn$, is a potential deficiency under normal loads, a potentially unacceptable dam performance condition under normal loads, that has not yet been confirmed.

$Pu$, is a potential deficiency under unlikely loads, a potentially unacceptable dam performance condition under unlikely loads, that has not yet been confirmed.

$Pq$, is a potential deficiency expected to be readily demonstrated not to be a deficiency. A potentially unacceptable dam performance condition may exist, but it is expected that with some investigation, the potential deficiency will be demonstrated as being not a deficiency.

$Pd$, is a potential deficiency expected not to be deficient, but difficult to prove, a potentially unacceptable dam performance condition that is not expected to be deficient, however it would be difficult or impossible to demonstrate.

Non-conformances: Failure to establish or to follow appropriate policies, procedures, operating instructions, maintenance requirements, or surveillance plans. But, importantly, a non-conformance in itself is not indicative of unacceptable dam performance. The following are the non-conformance categories: $NCi$ indicates that required information is not available; $NCo$ is an operational non-conformance; $NCm$ is a maintenance non-conformance; $NCs$ is a surveillance non-conformance; and $NCp$ is a non-conformance in other procedures.

3.0 MIDDLE CHASE RIVER DAM & RELATED STRUCTURES

3.1 History

The history of the dam is obscured by the lack of records and documentation before 1977. Important dates are:
~ 1910: constructed by Harewood Colliery;

~ 1945: taken out of service with Colliery;

1975 ownership passed to City of Nanaimo (Parks & Recreation);

1977 Inspection by Province leading to requirement for licensing;

1978 Investigation of dam’s condition and design of remedial work (Willis, Cunliffe & Tait + Golder Associates);

1980 Remediation of dam (directed by Willis, Cunliffe & Tait);

1987 Dayton & Knight produced dam “data file”;

1992 Inspection by EBA and development of “data book”;

1993 Responsibility for the dam transferred to Engineering from Parks & Recreation;

1993 Investigation and remediation of concentrated seepage from right abutment; and

1995 Extension of concrete spillway wall and installation of seepage monitoring weir.

3.2 Description

3.2.1 Foundation Conditions

Middle Chase Dam lies in a narrow steep sided ravine, with both abutment founded on bedrock. Bedrock comprises conglomerates of the Millstream Member. Exposures on either abutment indicate a massive rock formation with tight joints and fractures.

There have been no hydrogeological studies nor are there any piezometers in the abutments. The groundwater movement in the abutments is thus unknown, as is the influence of the retained reservoir. What can be inferred is that the reported abutment seepage is more likely from precipitation flowing into the abutment than the reservoir. This follows from the absence of seepage zones during the current inspection which was in a dry period but with full reservoir retained.

Bedrock may be overlain by a veneer of till (or possibly channel fill) in the centre of the ravine. One of the boreholes put down in the 1978 investigation of the dam penetrated
the dam fill and encountered a till-like soil in the deepest part of the ravine; this till-like material was only penetrated for 300mm before the drill got stuck.

The dam was originally constructed to supply water to a colliery. This raised concerns about whether mining operations might underlie the dams foundations. These concerns were addressed in a 1992 study by Westwater Mining Ltd and which found that no documented mineworking approached the dam foundation area.

### 3.2.2 Earthfill Dam

As noted above, Middle Chase Dam is a historic structure nearly 100 years old. No records appear to have survived from when it was constructed, and the understanding of the dam is based on work carried out over the past twenty years as now documented. Figure 3.1 shows the best estimate of the internal configuration of the dam.

A concrete wall provides the impervious barrier, and also forms the front face of the dam at normal pool level in the reservoir (Photograph 1-1). This wall is 0.6m thick at the top. There is no data or test results to show whether or not the concrete wall is reinforced, although the rebar found in the old spillway crest suggests that it might be. There are no expansion or other movement joints apparent in the concrete wall, although these might have been covered by the 1 ft crest raise concrete placed in 1980 (see below). It is unclear whether the wall thickens with depth.

The concrete core wall is presumed to extend to full depth and be founded on bedrock. Evidence for this presumption comes from the rather large excavation in 1980 as part of the dams remediation, see Photo 3-1 below.

![Photograph 3-1: Excavation of downstream shell of dam in 1980 (from Water Management Branch, 1982)](image-url)
1. This figure is taken from 1987 “Data File”, but it does not match the details shown on Sheet 9 of the As-Built drawings from 1980. It appears to be a schematic section.

2. The downstream shell is what was believed to exist prior to 1980. It was largely replaced with compacted sand and gravel.

3. Elevations are arbitrary being referred to a local benchmark.

Figure 3.1: Schematic cross-section of Middle Chase Dam – NTS
There have apparently been no concrete cores taken over the years and the strength of the concrete is unknown. However, the concrete is weathering well and there are no apparent signs of deterioration (see Appendix B).

On the upstream side, the core wall is supported by rockfill. The top of this was exposed during the 1980 remediation when the reservoir was lowered, but the rockfill is below the normal reservoir surface with a metre or so of water at the core wall with normal pool conditions. No underwater survey results have been discovered and the 1V:1H slope shown on Figure 3.1 is only substantiated in the upper part (Photograph 3-2). The upstream shell is believed to be rockfill based on the slope; however a photograph taken while the reservoir was low in 1980 indicates a more silt-like material on the surface (Photograph 3-3). There are no boreholes extending through the upstream shell.

The downstream shell has a nominal 2H:1V slope and was extensively rebuilt in 1980 (see below). No grading curves of the shell material are available but the inspector from the engineer (Willis, Cunliffe & Tait) directed placement of acceptable fill. It is presumed that this was a sandy gravel pit run material. It was compacted. The slope is covered with grass.

A gravel filter drain to intercept seepage through the dam and abutment was installed in the 1980 remediation. This gravel was topped with a layer of shot-rock.

Concentrated seepage was reported in the right abutment at about mid-slope during the 1992 inspection. This was thoroughly investigated in 1993, with the seepage being channeled to the filter drain as documented in Appendix C.

3.2.3 Spillway

The spillway is a channel comprising concrete retaining walls in the upper part and using the natural bedrock of the left abutment as the base. A footbridge spans the spillway at the inlet (Photograph 1-1). The original inlet works comprised a concrete weir section standing some 350 mm above bedrock and with steel channel section for stop-logs. The weir has now been removed in part, so that lake level is largely controlled by natural rock elevation without a weir as such.
Photograph 3-2: View of upstream shell while reservoir was dewatered in 1980 (from Water Management Branch, 1982)

Photograph 3-3: Silt on surface of upstream shell (from Water Management Branch, 1982)
3.2.4 Offtake Works

There are no presently working offtake works, the dam being used to only retain water for recreation.

The dam was built for water supply and a low-level outlet upstream gate existed at about the centre of the ravine in 1980 (Photograph 3-4). The stem of the valve was located by divers then, but obstructions prevent further investigation of what components/piping remained in place. The expected offtake pipe location in plan is indicated on Figure 1-2, but excavations during the remediation of the dam did not find any outlet pipe passing through the concrete wall despite an intensive effort.

![Photograph 3-4: Timber frame showing location of upstream valve in 1978](after Water Management Branch, 1992)

Thus, there is evidence that an offtake pipe once passed through the dam but this may or may not exist any longer (the estimated location is shown on Figure 1-2). If the pipe does exist, then it likely would comprise a wood stave pipe as this was used for the similar Lower Chase dam constructed by the same colliery at the about the same time.

3.2.5 Remedial Work 1978-80

The 1978 investigations and studies of the dam identified four aspects requiring remediation:

- Installation of a seepage collection and filter system;
- Construction of spillway training walls;
- Location and backfilling of the offtake pipe through the dam; and
- Repair of a hole in the upstream face of the dam.
When executing the work the contractor was unable to find the discharge conduit through
the dam despite extensive excavations; this aspect of the work was apparently abandoned
without locating the pipe and in effect resulted in reconstruction of about two thirds of
the downstream shell (Photograph 3-1). The excavations encountered and removed an
old log crib dam that was within the original downstream shell of the dam.

This reconstruction of the downstream shell lead to a larger drain system than the initially
envisaged trenches, but otherwise the works were constructed as intended.

The spillway retaining walls were constructed as intended.

The hole in the upstream face of the dam was repaired by pouring new concrete. It is
presumed that the repair was to the full 0.6m thickness of the wall, but this is unclear
from the records. This repair appears to be still in excellent condition (see Appendix B).

Although not included in the original contract documents, at some point during the
remediation it was decided to raise the height of the concrete core wall by 300 mm.

The progress of the remedial works is documented in photographs and notes by the
Provincial inspectors (some of which have been copied into this report as noted); nothing
apparently remains of notes form the engineering company directing the work. However,
this company did provide as-constructed drawings of the works.

3.3 Safety Standard

3.3.1 Consequence Classification

The Canadian Dam Safety Association guidelines are used by the Water Management
Branch in assessing the required standards of dams. Determination of the required safety
level and appropriate engineering standards are based on the consequence classification
of the dam.

Consequence categories are based on the incremental losses that a failure of the dam
might inflict on downstream or upstream areas, or at the dam location itself. Incremental
looses are those over and above losses which might have occurred in the same natural
event or condition had the dam not failed. Incremental losses are evaluated in terms of:

- Loss of life;
- Economic losses or damage to property; and
- Other (less quantifiable) social, cultural and environmental damage.
Table 3.1 shows the classification of dams in terms of these loss categories. The highest consequence category of the three considerations is the governing rating for the dam.

### Table 3-1: Classification of Dams in Terms of Consequences of Failure (after Table 1.1 – CDA Guidelines)

<table>
<thead>
<tr>
<th>CONSEQUENCE CATEGORY</th>
<th>POTENTIAL INCREMENTAL CONSEQUENCES OF FAILURE[a]</th>
<th>SOCIOECONOMIC FINANCIAL &amp; ENVIRONMENTAL[b][c]</th>
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<tr>
<td></td>
<td>LIFE SAFETY[b]</td>
<td></td>
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<tr>
<td>Very High</td>
<td>Large number of fatalities</td>
<td>Extreme damages</td>
</tr>
<tr>
<td>High</td>
<td>Some fatalities</td>
<td>Large damages</td>
</tr>
<tr>
<td>Low</td>
<td>No fatalities anticipated</td>
<td>Moderate damages</td>
</tr>
<tr>
<td>Very Low</td>
<td>No fatalities</td>
<td>Minor damages beyond owner’s property</td>
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</tbody>
</table>

[a] Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without failure of the dam. The consequence (i.e. loss of life or economic losses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.

[b] The criteria which define the Consequence Categories should be established between the Owner and regulatory authorities, consistent with societal expectations. Where regulatory authorities do not exist, or do not provide guidance, the criteria should be set by the Owner to be consistent with societal expectations. The criteria may be based on the levels of risk which are acceptable or tolerable to society.

[c] The Owner may wish to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their liability for damage to others.

### 3.3.2 Hazard Rating

The Dam Safety Branch presently rate Middle Chase Dam as a HIGH hazard structure.

The dam failure inundation study of 1977 suggest that flooding in the event of dam failure could be extensive, see Figure 3.2. This study apparently assumed that failure of Middle Chase would also lead to failure of the Lower Chase dam. The 2002 hydrology study produced similar estimates. However, the potential maximum depth of flooding has not been mapped.

The potential inundation area comprises older houses and one school. Stored chemical or other possible environmental contaminants have not been evaluated or inventoried. Possible economic losses in the flooded area have not been quantified.

The warning time would depend on whether failure arose from deterioration of the dam (which might be undetected between monthly inspections), earthquake (no warning) or storm flooding (likely anticipated from forecasts with an inspector assigned to the dam with a radio). Although storm flooding seems the more likely extreme event failure
(discussed below) and the City’s procedures should provide substantial warning for this case, no warning has been adopted for the present hazard assessment pending further earthquake resistance estimates.

Based on the above, and in comparison to Table 3-1, it is difficult to argue for a LOW rating. The fact the dam is old with unknown features (especially the possible offtake pipe which if it exists and if it fails would do so with possibly no warning) would suggest that a risk of some fatalities cannot ever be discounted. Conversely, the dam is patently not in the same consequence class as the large BC Hydro dams such as Mica, Revelstoke or Bennett and which comprise the VERY HIGH rating.

We conclude that the present HIGH rating is reasonable and appropriate.

### 3.3.3 Required Safety Criteria

The required design/assessment criteria for dam safety depend on the consequence category. Tables 3.2 and 3.3 show the safety criteria published by the Canadian Dam Safety Association for earthquake and flood situations respectively. Based on these tables and the HIGH consequence category, it follows that Middle Chase Dam should:

- Withstand about a 3,000 year return period earthquake; and

- Safely pass about a 3000 year flood.

In choosing these targets, the range of standard given in the tables has been recognized as follows. At the upper end (less likely, or longer return period) Middle Chase Dam is obviously much different from the very large BC Hydro dams and thus the values used at Middle Chase should not approach those indicated as appropriate for VERY HIGH consequence dams.
Figure 3.2: Present estimate of possible inundation area.

Note: this figure was taken from the dam’s Data Book and the streets are as indistinct as shown on this figure
Equally, on the other hand, some conservatism is warranted at the lower end as the dam is not a modern engineered structure and aspects of potential damage remain unquantified. Therefore a reasonable risk for the Middle Chase dam would be mid-way between the two extremes, and this is 3000 years because the underlying risk scale in the tables is order of magnitude (logarithmic).

Table 3.2: USUAL MINIMUM CRITERIA FOR DESIGN EARTHQUAKES
(after Table 5.1-CDA Guidelines)

<table>
<thead>
<tr>
<th>CONSEQUENCE CATEGORY</th>
<th>MAXIMUM DESIGN EARTHQUAKE (MDE)</th>
<th>DETERMINISTICALLY DERIVED</th>
<th>PROBABILISTICALLY DERIVED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>MCE</td>
<td></td>
<td>1/10,000</td>
</tr>
<tr>
<td>High</td>
<td>50% to 100% MCE</td>
<td></td>
<td>1/1000 to 1/10,000</td>
</tr>
<tr>
<td>Low</td>
<td>Building Code</td>
<td></td>
<td>1/100 to 1/1000</td>
</tr>
</tbody>
</table>

Table 3.3: USUAL MINIMUM CRITERIA FOR DESIGN INFLOW FLOODS
(after Table 6.1-CDA Guidelines)

<table>
<thead>
<tr>
<th>CONSEQUENCE CATEGORY</th>
<th>INFLOW DESIGN FLOOD (IDF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>Probable Maximum Flood (PMF)</td>
</tr>
<tr>
<td>High</td>
<td>Annual exceedance probability between 1/1000 and the PMF (depends on consequences)</td>
</tr>
<tr>
<td>Low</td>
<td>Annual exceedance probability between 1/100 and 1/1000</td>
</tr>
</tbody>
</table>

3.4 Hydrology

There is no design report as such for the Middle Chase dam, but the hydrology of the Chase watershed has been assessed in three studies while the dam has been owned by the City. These studies were:

- 1978: by Willis, Cunliffe & Tait in connection with the 1980 dam review;
- 1987: by Dayton & Knight in connection with the dam’s Data File; and
The results of these studies have been plotted as storm flow versus the associated estimated return period on Figure 3.3. Also shown on this figure is the probable maximum flood (PMF) which is only quoted by Water Management Consultants.

There is some difference between the 1978 study and the other two. A trend line has been drawn through the results of these studies, weighted to the 1987/2002 results, to indicate a present best-estimate of how the flood increases with increasing return period. A 3000 year return period event, the appropriate risk standard for the dam as discussed above, corresponds with about a 80 m³/sec flood inflow to the reservoir.

![Figure 3.3: Summary of hydrological studies for Middle Chase showing estimated flood flow versus return period](image)

The PMF represents something in excess of a 10,000 year event which is in our opinion unwarrantedly conservative for the dam.

There has been no systematic measurement of weir flows over the life of the dam, and thus no measured data on which the hydrological estimates can be improved.
3.5 Earthquakes

With the exception of very low consequence situations, dams are held to a higher standard of public safety than implied by the National Building Code. This requires an assessment of the possible loadings caused by earthquakes.

No site specific seismic hazard assessments have been undertaken for this dam, or the Chase River watershed. However, a detailed earthquake hazard assessment was undertaken recently in connection with the City’s South Forks Dam (Sandwell, 2002). In addition, BC Hydro (1992) carried out a systematic assessment of earthquake risk and which included the John Hart Dam. Both dams are in a comparable earthquake risk situation to the Chase River dams. The results of these seismic hazard estimates are plotted in Figure 3.4.

![Graph](image)

Figure 3.4: Horizontal peak ground acceleration versus estimated return period

A 3000 year return period event, the appropriate risk standard for the dam as discussed above, corresponds with about a 0.5g peak ground acceleration at the dam site, depending on assumptions in the seismic model.
4.0 FAILURE MODE ANALYSIS

4.1 Potential Failure Modes

The causes of water-retaining earthfill dam failures have been investigated and summarized by the International Committee on Large Dams (ICOLD). Some of the potential failure mechanisms are relevant to Middle Chase dam. The most common failure mechanisms for water retaining earthfill dams are summarized in Figure 4.1.

![Figure 4.1: Summary of Failures in Water Retaining Earthfill Dams (ICOLD, 1995)]

Based on the site specific study, as well as the overall statistical analyses of dam failures, the most common modes of failure that are considered in this report are presented in Table 4-1, with comments regarding the relevance of each mechanism to Middle Chase dam. These potential failure modes are discussed further in the following sections, and considered with respect to the monitoring information available, and the observations made during a site inspection.

4.2 Seepage within Foundation/Abutments

4.2.1 Design Basis and Expected Behaviour

There is no design report for the dam, nor has expected foundation seepage been evaluated to date. There is also no indication that a grout curtain was ever constructed in the rock beneath the concrete core.

Because the dam is founded on tight bedrock and the retained head is small, seepage through the foundation would not be large. Assuming that the bedrock is exposed in the reservoir and that the rock permeability decreases with depth (as is usual), it is estimated that seepage through the rock would be less than 1 L/min at normal reservoir level.
### Table 4-1: Relevance of Most Common Failure Modes to Middle Chase Dam

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Comments/Relevance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope Instability</td>
<td>Instability of compacted downstream shell is unlikely. Instability of upstream rock fill is unlikely.</td>
</tr>
<tr>
<td>Earthquake</td>
<td>Dam is in an area of high seismicity. It has not been designed to resist earthquakes, but is unlikely to show liquefaction failure. The core wall may be vulnerable to cracking or toppling. Crest settlement/slumping would be expected in a substantial earthquake, but is unlikely to exceed normal dam freeboard.</td>
</tr>
<tr>
<td>Overtopping</td>
<td>Dam operating freeboard is quite low. Spillway is uncontrolled, with uncertainties on the design storm. Ongoing dam safety requires spillway be kept clear of all debris. Dam crest is erodible.</td>
</tr>
<tr>
<td>Seepage within embankment</td>
<td>The downstream shell is permeable. A seepage collection and drainage system was designed and constructed in 1980. The core wall is concrete.</td>
</tr>
<tr>
<td>Internal erosion</td>
<td>Internal erosion is a potential mode of failure if drainage filters does not perform as designed and the concrete core wall is cracked by slope movement or an earthquake.</td>
</tr>
<tr>
<td>Foundation seepage and internal erosion of foundation</td>
<td>Seepage from right abutment is intercepted and carried to filter. Dam is founded on tight bedrock with little seepage apparent. Uplift from foundation seepage is not a credible failure mechanism even though there is no grout curtain.</td>
</tr>
<tr>
<td>Rupture of conduits</td>
<td>There may be an old and decaying low level outlet pipe through the dam. The upstream end of the pipe may be buried in reservoir and attached to a decaying valve.</td>
</tr>
</tbody>
</table>
4.2.2 Performance Review

There is only one seepage monitoring weir and it cannot distinguish between the various possible seepage paths. It is thought likely that most of the baseflow seepage (ie that not attributable to rainfall) arises from either cracks in the core wall or poor core/bedrock contact. This latter condition was noted at the right abutment in 1978 during a dam inspection.

There has been seepage in the past from the right abutment, but this seems seasonal and related to rainfall rather than retained water. The investigation in 1993 resulted in minor works and these were documented as stabilizing any fines loss from the abutment (see Appendix C).

On balance it appears that the foundation is behaving as expected.

4.3 Slope Instability

4.3.1 Design Basis and Expected Behaviour

**Downstream Slope.** The stability of the downstream shell was investigated in 1978 as part of the work leading up to the 1980 remediation. The safety factor for slope stability computed as 1.7 under long term conditions and without an earthquake loading, which compares favourably with a conventional 1.5 required by design code (Canadian Foundation Manual) for long term stability.

Reconstruction of the downstream shell with compacted granular fill will have improved this stability.

**Upstream Slope.** The upstream slope stability has not been addressed in any study to date, other than the judgement made in 1978 that it was stable since it appeared to have not moved in seventy five years.

A difficulty with the upstream slope is that although the drawings show it as rockfill, which should indeed be stable, there are no borehole or test pit records to substantiate this. If it is indeed rockfill, then we concur with the 1978 judgement that it is adequately stable statically.

4.3.2 Performance Review

There are no slope movement monitoring records.
The site inspection (Appendix B) found no sign of slope movement in the downstream shell.

The upstream slope was underwater and could not be inspected (See Photograph 1-1). However, there were no signs of distress in the concrete core wall that would infer loss of upstream support.

4.4 Earthquake

4.4.1 Design Basis and Expected Behaviour

The dam remediation in 1978 explicitly excluded seismic stability as an issue. It is thought that the view taken then was that, as the dam had survived the 1946 earthquake, and was 75 years old in 1978, it was likely adequate given the safety standards of the day.

In the event the remediation in 1980 led to almost complete reconstruction of the downstream shell with compacted fill. This will have improved the resistance to earthquake related failure, but there are no studies of the remediated dam.

The appropriate site ground motion for the appropriate earthquake risk is so strong (0.5g) that slope movement will occur under this condition and this cannot be prevented by even substantial remedial works. However, slope movement does not necessarily represent failure provided that the associated crest settlement is less than the freeboard of the reservoir. Settlement under the design earthquake motion needs to be formally computed; nevertheless we would be surprised if it would exceed about 0.5 m for the downstream slope. What is more problematic is the upstream slope.

Under earthquake motion, movement of the rockfill would be expected, much like the downstream. Whether or not this might lead to a dam breach would depend on how the concrete core responded to the rockfill movement and/or loss of support from the retained water during the shaking. This aspect requires further investigation, and some tests on the concrete core wall itself. Our expectation is that the concrete wall might crack but would not fail provided that the wall comprises reinforced concrete – testing is required to establish what rebar is in place and at what spacings.

4.4.2 Performance Review

There is no strong motion instrumentation on or near the dam. However, the historical record of earthquakes has been used to estimate the ground motions that the dam has experienced in its near hundred year life. The results are shown on Figure 4.2.
The experienced peak acceleration of 0.03g corresponds to about a 20-year return period event based on the ground motion studies plotted in Figure 3.4.

![Graph showing peak ground acceleration over time](image)

**Figure 4.2: Estimated peak ground acceleration experienced by Middle Chase Dam**

Because there is 95% confidence that the dam has actually experienced its 30 year return period event in its 90 year history, it appears that the design trends shown on Figure 3.4 are conservatively biased for the Chase River area. A peak ground acceleration of 0.2g might be a better estimate of the appropriate motion for the 3000 year return period standard of care.

Comparison of dam performance at 0.03g, which is presumed adequate as no records or articles have been found indicating otherwise, provides no assurance of adequacy at even the reduced 0.2g ground motion level. This aspect requires further evaluation.

### 4.5 Overtopping

#### 4.5.1 Design Basis and Expected Behaviour

The spillway capacity has been evaluated concurrently with each of the hydrological studies discussed earlier. There are discrepancies in the estimated capacity of the
spillway as follows:

- 1978, Willis, Cunliffe & Tait: 42 m³/sec;
- 1987, Dayton & Knight: 84 m³/sec; and

The 1987 and 2002 estimates may in fact be closer than they appear as the 2002 estimates allow for the presence of the footbridge (1.6m clearance above the entrance sill) whereas the 1987 estimate is based on the lowest point of the dam crest (2.1m of retained water). At 1.6m clearance, the 1987 capacity curve indicates about 54 m³/sec discharge.

Comparing these estimates of spillway capacity to the hydrology discussed earlier indicates that the spillway likely does not have the capacity to pass the appropriate flood (which is about 80 m³/sec). Broadly, the spillway capacity need to be increased by about one third to meet current dam safety standards.

4.5.2 Performance Review

The elevation of the surface of the water impounded in the reservoir has been monitored weekly for the past nine years using a staff gauge beneath the footbridge. The results are shown on Figure 4.3.

![Figure 4.3: Measured water depths at spillway](image-url)
Retained water levels were much below the dam crest, at which level there is 2.1m depth of water at the spillway.

There appear to be no spillway performance records from earlier than 1995. Data is also missing for 1997, and very limited in 2001.

4.6 Seepage within Embankment

4.6.1 Design Basis and Expected Behaviour

The dam was constructed with what appears to be an engineered concrete wall. It is assumed that this engineering attention extended to how the wall met the bedrock and that the engineers who directed construction endeavoured to ensure a good contact. The wall also has little obvious cracking. In this situation, very little flow would be expected through the dam.

4.6.2 Seepage Performance Review

Seepage through the dam is monitored using a V-notch weir at the toe. This weir collects seepage through the dam core, seepage from the right abutment, stored precipitation onto the downstream shell, seepage through the foundation, and seepage beneath the spillway training wall at the concrete/rock contact. Measurements have been made weekly for the past four years.

The measured flow at the weir has been plotted on Figure 4.4 and which is based on an estimated weir coefficient $C_d=0.7$ (no calibration sheet is available, and this $C_d$ should, if anything, overestimate actual seepage). There are week by week fluctuations as would be expected given that the weir measures more than seepage through the dam. What is intriguing, however, is an apparent declining seepage trend in the long term flows. This suggest that the flow features are sealing up, possibly by silt in the base of the reservoir.

In terms of absolute flows, the flow measured during our inspection was approximately 12 L/min. As noted above, about 1 L/min of this could be attributed to flow in the bedrock beneath the dam. A further 2-4 L/min was flow from the spillway that was getting underneath the spillway concrete wall and into the downstream shell (see Appendix B). There was no abutment flow during our inspection and no recent rainfall. Thus, about 8 L/min represents flow through the impervious barrier of the dam. This flow can be expressed as a hydraulic conductivity “index” as a measure of the integrity of the dam by dividing this residual flow by the plane area of the concrete core (about 250 m$^2$) and an average hydraulic gradient ($= 5$m head/0.6 m wall thickness). The resulting conductivity is about $1 \times 10^{-7}$ m/s, which is consistent with somewhat extensive fine cracks.
such as might be found from concrete shrinkage or between the concrete wall and the bedrock.

![Graph showing seepage flow through weir]

Figure 4.4: Measured flows at V-notch weir at toe of slope (using estimated weir $C_d=0.7$)

4.7 Internal Erosion in Embankment (Piping)

4.7.1 Design Basis and Expected Behaviour

The dam is a composite structure with a concrete wall providing the impervious barrier. This wall is apparently continuous and therefore there should be no seepage through the dam as such, and hence no expectation of any internal erosion even in the absence of filters.

4.7.2 Performance Review

The exposed concrete is in apparently excellent condition. Seepage at the V-notch weir is consistent with minor cracking of the concrete and without any concentrated seepage that might lead to erosion of the dam fill.
4.8 Rupture of Conduits

4.8.1 Design Basis and Expected Behaviour

How the original offtake pipe passes through the core wall is unknown. It is not possible to evaluate what design basis might have been adopted when the dam was designed and constructed.

The danger that internal pipes represent to dams was unlikely to have been appreciated when the Middle Chase dam was built nearly a hundred years ago. As such it is unlikely that any special measures were taken, especially as the control valve was located in the reservoir rather than in a chamber at the dam face (as was done for the similar Lower Chase dam).

The key question is whether that internal pipe was removed. The excavations during 1980 were extensive and orientated around the known location of the upstream valve. Yet no offtake pipe was found. This implies that either the offtake pipe either follows a strange route from the intake valve and continues to exist in the rightmost side of the downstream shell or that the offtake pipe was removed early in the dam’s life.

4.8.2 Performance Review

There is no evidence in terms of concentrated seepage or surface expression that a decaying wood stave pipe exists in the right hand side of the downstream shell and extending upstream through the core.

5.0 SAFETY REVIEW

5.1 Dam Safety

5.1.1 Implementation of 1992 Recommendations

The 1992 DSR recommended twelve actions. Of these, seven have been implemented to date (see Table 5.1). Of the remaining actions, some are not regarded as urgent (clearing dead trees as there are few). One recommendation, the addition of the log boom to the spillway, is regarded as undesirable because there is more likelihood of an accident to the public while swimming than any spillway blockage risk reduction to the dam.

Two recommendations, that for the earthquake risk assessment and dam upgrading as needed, together with the extension of the spillway training wall against the left abutment, are reiterated in this DSR as appropriate and requiring action.
Table 5.1: 1992 DSR Recommendations

<table>
<thead>
<tr>
<th>Recommendation</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Add stilling basin to weir to collect fines</td>
<td>Done</td>
</tr>
<tr>
<td>Install reservoir level staff gauge</td>
<td>Done</td>
</tr>
<tr>
<td>Replace wooden handrails on footbridge over spillway</td>
<td>Done</td>
</tr>
<tr>
<td>Install log boom at spillway inlet</td>
<td>Not carried out to date</td>
</tr>
<tr>
<td>Remove trees prior to toppling in reservoir</td>
<td>Done, and ongoing</td>
</tr>
<tr>
<td>Remove dead trees from reservoir</td>
<td>Not carried out to date</td>
</tr>
<tr>
<td>Determine source of existing seepage</td>
<td>Done</td>
</tr>
<tr>
<td>Remediate seepage, once source identified</td>
<td>Done</td>
</tr>
<tr>
<td>Core the concrete wall to determine depth of deterioration</td>
<td>Not carried out to date</td>
</tr>
<tr>
<td>Carry out earthquake resistance/upgrading study</td>
<td>Not carried out to date</td>
</tr>
<tr>
<td>Upgrade the hydrology of the Chase River system</td>
<td>Study completed 2002; follow on work ongoing. Used in this DSR.</td>
</tr>
<tr>
<td>Extend spillway training wall on left side</td>
<td>Not carried out to date</td>
</tr>
</tbody>
</table>

5.1.2 Results of July 2003 Site Inspection

Appendix B provides a detailed record of the inspection of the dam on 23-24 July 2003. Broadly:

- The dam was well maintained, with no excess vegetation or rodent intrusions;
- No indications of slope instability and/or crest settlement were found;
- No zones of concentrated seepage or other indications of internal erosion/piping were found;
- The spillway was clear, but showing erosion of the unprotected part of the left abutment. Trees are being undermined and will eventually topple into the channel, possibly causing spillway storm flow to get over the training wall and erode the downstream shell; and
- The spillway entrance had one grounded log with the potential to partially block the inlet during a storm, so restricting the already limited spillway capacity.
Comparing what was found in this inspection with that reported in 1992 indicates that the dam is in much the same condition as it was then.

5.1.3 CDA Standard Evaluation

Based on the above review of dam design/performance, and the site inspection, the dam safety review database has been used to assess the Middle Chase Dam. The detailed results are given in Appendix A on an issue by issue basis. For each issue, the following is given:

- The dam safety principle;
- The rating of Middle Chase dam with respect to that principle;
- A description of why that rating was assigned; and
- A recommendation to address the shortfall if the dam is non-compliant

The descriptions of the various ratings was given in Section 2 above.

Because the standard safety review database was used, not all questions are relevant to Middle Chase dam. In these instances Not Applicable has been shown for the relevant issue in Appendix A.

No Actual Deficiencies were identified. Twelve instances of Potential Deficiencies and Non-Conformances were identified, and these are summarized on Table 5.2. The numbers shown within the [ ] refer to the dam safety principles in Appendix A.

Potential deficiencies largely relate to three issues: the changing standard of care which means that Middle Chase dam no longer meets the public’s view of required safety in regard to (1) extreme storms and (2) earthquakes; and, (3) the unresolved issue of whether or not a decaying, historic offtake pipe exists in the downstream shell. This third issue is not new, being recognized in 1980, but could not be resolved then.

Non-conformances relate to information on the likelihood of extreme events and the implementing automatic seepage weir monitoring as a surveillance improvement to address the small possibility that a decaying offtake pipe might exist in the dam.
Table 5.2: Summary of Identified Deficiencies & Non-Compliances

<table>
<thead>
<tr>
<th>Deficiency Type</th>
<th>Instances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Potential Deficiency</td>
<td></td>
</tr>
<tr>
<td>Expected to be deficient under unlikely loads</td>
<td>5</td>
</tr>
<tr>
<td>[Principles: 6.1, 7.1, 7.2, 7.3, 9.5]</td>
<td></td>
</tr>
<tr>
<td>Expected not to be deficient, quickly demonstrated</td>
<td>2</td>
</tr>
<tr>
<td>[Principles: 8.7, 9.7]</td>
<td></td>
</tr>
<tr>
<td>Expected not to be deficient, difficult to demonstrate</td>
<td>1</td>
</tr>
<tr>
<td>[Principle: 8.3]</td>
<td></td>
</tr>
<tr>
<td>Non-Conformance</td>
<td></td>
</tr>
<tr>
<td>Information</td>
<td>1</td>
</tr>
<tr>
<td>[Principles: 5.1]</td>
<td></td>
</tr>
<tr>
<td>Surveillance</td>
<td>2</td>
</tr>
<tr>
<td>[Principle: 3.4, 8.9]</td>
<td></td>
</tr>
<tr>
<td>Other Procedures</td>
<td>1</td>
</tr>
<tr>
<td>[Principles: 10.2]</td>
<td></td>
</tr>
</tbody>
</table>

5.2 Emergency Preparedness

The emergency preparedness plan (EPP) has been updated as part of the 2003 dam safety work by the City and is appropriate.
6.0 CONCLUSIONS & RECOMMENDATIONS

The Middle Chase dam is in as good a condition today as it was at the last DSR in 1992. No evidence of any form of distress was observed. There are no Actual Deficiencies.

However, the dam is likely rather vulnerable to “unlikely” situations (storm, earthquake), and may contain a decaying pipe which is a well-recognized threat to dam safety. There are a total of twelve Potential Deficiencies and Non-Conformances with regard to dam safety. Work is recommended to:

- Automate reading of the V-notch weir and get the measurements into the City’s real time monitoring (SCADA) system. Weir flow is a crucial validation of ongoing dam safety given the uncertainty over details of the dam’s construction and tracking this should give useful advance warning of a deteriorating situation.

Part of the reasons for the findings in this DSR is that the dam is nearly 100 years old and was likely never formally designed. However, societal expectations have changed and old dam’s are required to be comparable to new dams in terms of safety for potentially affected people downstream. In the longer term, say within the next three years, the dam needs to be brought up to date from its current design of 1 in 100 year rare event (storm, earthquake) capability to something better than a 1 in 1000 year standard. Further engineering studies, and likely some construction upgrading are required. Specifically:

- Either a dam raise or spillway modification (or combination) is needed so that the dam can safely pass about a 3000 year return period storm. The exact design criteria needs to be agreed with the Water Comptroller, but is likely to lead to about a 30% increase in spillway capacity; and

- The seismic resistance of the dam is probably adequate and but needs formal documenting, in particular with attention to core cracking during shell movement. It is not thought that any physical works are required at this time, but the extent of any upgrading would need to be defined by further studies.

The above three recommendations are the principal items requiring attention.
Because of the dam’s age, and the way it came into the City’s ownership, records for the dam are far more sketchy than desirable and it must be recognized that aspects of the dam will remain uncertain despite the best efforts of the City. The emphasis on routine surveillance in the City’s operation of the dam is appropriate for the circumstances and necessary for public safety.

This dam safety review is submitted by Golder Associates Ltd.

M. Jefferies, P. Eng.  
Associate

H. Hawson, P. Eng.  
Principal
### 7.0 REFERENCES

<table>
<thead>
<tr>
<th>Library</th>
<th>Document</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>City</td>
<td>Reference</td>
<td>Document</td>
</tr>
<tr>
<td>Library</td>
<td>Dayton &amp; Knight (1987); Data File: Upper Harewood Colliery Dam (a.k.a. Middle Chase River Dam and Harewood No. 1 Dam)</td>
<td></td>
</tr>
<tr>
<td>Library</td>
<td>Willis, Cunliffe &amp; Tait (1979). Contact documents for Dam Rehabilitation Program, issued for Tender.</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX A

DETAILED RESULTS OF 2003 DAM SAFETY REVIEW
Appendix A: Detailed Results of Dam Safety Review

1.0 DAM SAFETY MANAGEMENT SYSTEM

1.1 The dam system, its functions and responsibilities shall be identified.
   Type: Cnf Conformance: Conforms
   Description: The acceptance of responsibility for the dam by the Engineering Dept of the City is clear, and individuals have been tasked. Surveys are being done both of the dam and of the underwater upstream slope, with revised OMS manual in draft awaiting this survey data.

1.2 The dam shall be classified in terms of the reasonably foreseeable consequences of failure for consideration in design, evaluation and management of dam safety.
   Type: Cnf Conformance: Conforms
   Description: The dam's classification of HIGH in current Provincial records has been reviewed during this inspection and found consistent with the present circumstances.

1.3 The dam safety management system shall include a process so that reported potential and actual deficiencies are followed up until resolution.
   Type: Cnf Conformance: Conforms
   Description: A list of outstanding dam safety issues is maintained by Engineering and used to track progress in resolving those issues. Part of the current backlog of issues relates to the dam originally being operated by the Parks & Recreation rather than Engineering group within the City, and what was found during interviews with the Engineering group was an awareness of issues and how to get dam safety on track.

1.4 Documentation shall be maintained so that a permanent record exists of the design and performance of the dam, and the management of its safety.
   Type: Cnf Conformance: Conforms
   Description: Files are maintained and were made available to the Inspection team; the City's filing system for reports and drawings was thorough. These included prior safety reviews. However, because the dam is a historic structure nearly 100 years old, there are neither design reports nor construction records. Paper records of the dam's performance and condition are kept.

1.5 All staff with responsibilities for dam safety activities shall be adequately qualified and trained.
   Type: Cnf Conformance: Conforms
   Description: Training videos are available and some staff have attended dam safety conferences. Training videos and the OMS manual are scheduled to be reviewed with each new employee in this area of work and every two years for the experienced Watershed Inspectors. Training and reviews is documented within Engineering (as a sign-off sheet within the corresponding dam operating log) as well as with Human Resources.

2.0 DAM SAFETY REVIEW

2.1 The dam safety management system shall include a process for independent periodic review ("Dam Safety Review") of safety management at a dam.
   Type: Cnf Conformance: Conforms
   Description: Implemented. Reviews carried out in 1978, 1992 and 2003 (this review)
3.0 OPERATION, MAINTENANCE AND SURVEILLANCE

3.1 Operation, maintenance and surveillance requirements for dam safety shall be documented in procedures ("OMS Manual") that allow the operators to operate the dam in a safe manner, maintain it in safe condition, and monitor its performance well enough to provide early identification of any conditions which might threaten dam safety.

Type: Cnf  
Description: OMS manual ("Data Book") created in 1992 and updated. The dam and reservoir is a passive system with no operation as such - the only operation is to ensure that the spillway is kept free of debris. The OMS manual is being revised concurrent with this DSR and includes updates of procedures to present standards.

3.2 Operating procedures shall be followed to ensure that the dam, together with applicable structures and equipment required for flood discharge, is operated safely.

Type: Cnf  
Description: The key operating requirement is that the spillway be kept debris free. This requirement is emphasized in the revised OMS manual. Procedures are in place to ensure that debris can be rapidly removed, either using City resources or external contractors, as required.

3.3 Maintenance procedures shall be followed to ensure that the dam, together with applicable structures and equipment required for flood discharge, is maintained in a safe and fully operable condition.

Type: Cnf  
Description: There are no moveable gates or machinery required for safe operation of the dam, and which require maintenance. Maintenance is primarily the routine clearing of debris from the spillway and cutting of grass on the downstream slope of the dam (to allow identification of any slope movements). The dam was found to be well maintained during the July 2003 site inspection.

3.4 Surveillance procedures shall be followed to monitor dam performance well enough to provide early identification of any conditions which might threaten dam safety.

Type: NCs  
Non-conformance: Surveillance

Description: Surveillance is presently monthly and based on the Watershed Inspector noting observations against a check list. Although these observations have been coordinated against potential failure modes in the recently revised OMS manual, the crucial importance of the weir flow to dam safety (given the possibly decaying outtake pipe within the dam) requires more than monthly inspections as a pipe-related failure would likely develop far faster.

Recommend: Implement automated weir flow readings on the City's SCADA system and with an appropriate alarm level set. To avoid false alarms, the reservoir level should be similarly monitored. These automated readings should also be stored at daily intervals in perpetuity as minimum, average and maximum values for the 24 hour period.

3.5 The dam owner shall ensure that the dam is adequately safeguarded to prevent unauthorized modifications or operation of the dam by someone other than the dam owner or an agent of the dam owner.

Type: Cnf  
Description: There are no operable valves or gates, and the only credible deliberate hazarding of
Appendix A: Detailed Results of Dam Safety Review (Continued)

4.0 EMERGENCY PREPAREDNESS

4.1 An emergency plan shall be established and maintained for any dam whose failure could be expected to result in loss of life, as well as for any dam where advanced warning would reduce upstream or downstream damage.

Type: Cnf Conformance: Conforms

Description: The EPP has been recently revised and its distribution includes identified responders to possible emergency situations.

4.2 Appropriate procedures and resources shall be available and documented in the emergency plan to support the actions to be taken in the event of an emergency at the dam.

Type: Cnf Conformance: Conforms

Description: The EPP defines a list of emergency scenarios. The actions to be taken in these situations are defined and appropriate. The EPP includes notification of other public agencies, but relies on these agencies to warn potentially affected residents. Given the historic nature of the dam, the EPP appropriately emphasizes public safety over immediate emergency repairs at the dam.

4.3 A program shall be in place to maintain emergency preparedness at the dam so that the operators and other responders are always prepared to act appropriately in the case of a dam safety emergency.

Type: Cnf Conformance: Conforms

Description: Program implemented as part of recent EPP revision.

5.0 EARTHQUAKES

5.1 The Maximum Design Earthquake (MDE) selected for design or evaluation of the dam and appurtenant structures shall be consistent with accepted practice for structures with similar consequences of failure.

Type: NCi Non-conformance: Information

Description: An earthquake hazard assessment was recommended in the 1992 inspection by EBA but has not been implemented. Estimates of an appropriate MDE are given in the main text of this DSR based on the results of such assessments for two nearby dams.

Recommend: Perform a seismic assessment of the dam to reconcile the historic experience at the dam site with other seismic hazard estimates.
6.0 FLOODS

6.1 The Inflow Design Flood (IDF) selected for design or evaluation of the dam and appurtenant structures shall be consistent with accepted practice for structures with similar consequences of failure.

Type: Pu

Potential Deficiency: Expected to be deficient under unlikely loads

Description: Studies are underway to revise the IDF. Estimates have been developed for this DSR based on this recent work together with prior studies for the City that were done in 1978 and 1987. A PMF event is viewed as excessively conservative and unlikely, see the text of the report.

Recommend: Complete the Chase River hydrology study and review the estimated IDF used in this assessment.

7.0 DISCHARGE FACILITIES

7.1 The discharge facilities shall be capable of passing the IDF without exceeding any structural limitations on the dam.

Type: Pu

Potential Deficiency: Expected to be deficient under unlikely loads

Description: About a 30% increase in spillway capacity is required to pass the IDF estimated in this study.

Recommend: Investigate options for upgrading the spillway and/or raising the dam crest to get a greater head on the existing spillway; modifications to the footbridge may be needed.

7.2 The approach and exit channels of discharge facilities shall be adequately protected against erosion and shall be free of any obstructions that could adversely affect the discharge capacity of the facilities.

Type: Pu

Potential Deficiency: Expected to be deficient under unlikely loads

Description: There is substantial erosion of the lower left side of the spillway which relies on the natural abutment, not a concrete wall. This erosion is undermining trees and will lead to debris in the channel. The wall on the right side of the spillway may not be high enough to protect the downstream shell from erosion during near IDF events.

Recommend: Cut down the undermined trees and evaluate to what extent the spillway concrete needs raising/extending as part of the spillway upgrading recommended in 7.1

7.3 Sufficient freeboard shall be provided for all operating conditions including extreme floods, wind conditions and earthquakes.

Type: Pu

Potential Deficiency: Expected to be deficient under unlikely loads

Description: Freeboard under the IDF may be compromised as the spillway has inadequate capacity. Freeboard under earthquake induced dam movements is estimated to be acceptable, but must be confirmed (see item 8.7). The reservoir is small with a short fetch and is also sheltered, so that waves are not expected to present any additional concerns for the dam's integrity.

Recommend: This deficiency will be resolved with upgrading of the spillway, see 7.1 and 7.2.

7.4 The dam outflow structures shall be capable of handling ice and debris.

Type: Cnf

Conformance: Conforms

Description: There are no trash racks or other debris screens, but these would interfere with the recreational use of the dam. Further, given the small size of the dam it is unlikely that large logs could ever pass through the spillway. However, there is little decayed
Appendix A: Detailed Results of Dam Safety Review (Continued)

7.5 All flow control equipment shall be capable of opening and closing under required operating conditions.
Type: N/A Other: Not Applicable
Description: There is no flow control equipment at the dam

8.0 GEOTECHNICAL STRUCTURES

8.1 The slopes of an embankment dam and its abutments shall ensure that the dam, foundation and abutments are stable under all reservoir levels and operating conditions.
Type: Cnf Conformance: Conforms
Description: No indications of any slope movement were found during the dam inspection. The downstream shell was upgraded during the 1980 remediation and the factor of safety should exceed the already adequate quoted value of the 1978 investigation. The upstream slope is steep, but has apparently remained stable for nearly 100 years. There are no indications of any abutment movement.

8.2 Adequate filter and drainage facilities shall be provided in an embankment to intercept and control the maximum anticipated seepage, and to prevent significant migration of particles.
Type: Cnf Conformance: Conforms
Description: An engineered filter and drainage system was installed during the 1980 remediation of the dam. As discussed in the main report, measured outflows from the dam are comparable to what might be expected from only minor cracking of the concrete core wall or imperfection in wall/bedrock contact. Correspondingly, the principal role of the seepage collection system is for inflows from the abutments as the concrete core wall in itself is a satisfactory hydraulic barrier and immune to deterioration from internal erosion. There is no evidence of accumulating fines in the still water pool upstream of the weir plate.

8.3 The hydraulic gradients in an embankment dam, in its foundation abutments, and along embedded conduits and other appurtenant structures, shall be sufficiently low to prevent piping and heave in the existing material.
Type: Pd Potential Deficiency: Expected not to be deficient, difficult to demonstrated
Description: There are no piezometers in the abutments or embankment with which to establish hydraulic gradients.
Recommend: This deficiency would require the installation of piezometers to prove the existence of what are estimated to be very low hydraulic gradients (based on the low seepage flow and types of material). Installation of piezometers is a low priority and it is more important to address spillway capacity and earthquake stability aspects.

8.4 An embankment dam shall retain the reservoir safely in spite of any cracking that may be induced by settlement, hydraulic fracturing or frost action.
Type: Cnf Conformance: Conforms
Description: There are no signs of ongoing settlements, consistent with the dam having been in place for nearly 100 years. Measured seepage is consistent with at most minor and fine cracks in the internal concrete wall.
The upstream slope is rockfill and below normal reservoir pool level, so that the concrete core wall forms the exposed upstream slope of the dam. No evidence of concrete deterioration was found during the site inspection. The downstream slope is grassed with the toe also being protected by rockfill. The site inspection showed that the slope was in good condition.

All embankment and foundation materials susceptible to liquefaction shall be identified, and the post-liquefaction stability of the embankment dam shall be evaluated. If appropriate, remedial measures shall be undertaken to protect against failure of the embankment dam.

The earthquake vulnerability of the dam was identified as a potential deficiency in 1992 and remains to be addressed. The ability of the dam to withstand the estimated MDE is uncertain, the principal vulnerability being failure of the concrete core wall because of temporary loss of support. It is thought that such failure would result in accelerated erosion of the dam rather than a quick failure, but this need to be fully reviewed in a detailed study.

The rock is sound, and issues of potential undermining were addressed in the 1992 Investigation. Foundation conditions are excellent.

In situ foundations and abutments as well as embankments and backfill, shall be free from gravity-driven movement that would impair the operational capability of appurtenant hydraulic structures or threaten their structural integrity and hydraulic performance.

Fill surrounding appurtenant hydraulic structures shall be free of localized concentrations of seepage that could lead to piping. The foundations and embankment shall be protected from
Appendix A: Detailed Results of Dam Safety Review (Continued)

There spillway is founded on rock and in the abutment. No localized seepage was evident at the interface between the dam and spillway wall that could lead to piping.

9.0 CONCRETE (AND OTHER RIGID) STRUCTURES

9.1 The analysis and evaluation of the strength and condition of a concrete dam (or other rigid structure), its foundation and appurtenances shall be consistent with accepted practice for structures with similar consequences of failure.

Type: Cnf Conformance: Conforms
Description: Not a concrete dam. The concrete spillway walls where they form the abutments/supports for the footbridge are spalling from concentrated loads imposed by the concrete bridge beams, see the Inspection Record (Appendix B). This is not a dam safety issue as the damage is well above the water levels in the spillway channel.

9.2 Concrete dams, their foundations and appurtenant structures shall have adequate resistance to sliding at the dam-foundation interface, within the dam and at any plane in the foundation, to withstand all reasonable loads and load combinations to achieve adequate dam safety.

Type: N/A Other: Not Applicable
Description: Not applicable.

9.3 If required to achieve dam stability, foundation drainage systems shall be designed, maintained and operated to achieve their purpose during and after all reasonable loading conditions.

Type: N/A Other: Not Applicable
Description: Not applicable.

9.4 The concrete shall have sufficient strength that the loads will not result in excessive deformations or overstressing.

Type: N/A Other: Not Applicable
Description: Not applicable.

9.5 During and after extreme events such as the IDF and the MDE, the dam shall continue to safely retain the reservoir water.

Type: Pu Potential Deficiency: Expected to be deficient under unlikely loads
Description: There have been no evaluations of possible damage to the present spillway during the IDF.
Recommend: This deficiency is expected to be addressed in the upgrading of the spillway for hydraulic capacity and does not require specific attention here.

9.6 Structural integrity and functionality of support structures for mechanical and electrical equipment that relate to dam safety shall be preserved during and after extreme events including the IDF and MDE.

Type: N/A Other: Not Applicable
Description: There is no installed electrical or mechanical equipment.
Appendix A: Detailed Results of Dam Safety Review (Continued)

9.7 Appurtenant structures shall be capable of withstanding all reasonable loads and load combinations.

Type: Pq  Potential Deficiency: Expected not to be deficient, quickly demonstrated
Description: It is anticipated that the pedestrian footbridge is not earthquake resistant and may topple into the spillway. Provided that this debris was cleared in a timely manner, loss of the footbridge would not be a dam safety issue. The spillway walls require chacking for earthquake resistance; in our opinion they are likely adequate.
Recommend: Need to check overturning loads versus dowel bar spacing and capacity for the spillway walls during earthquake loading.

10.0 RESERVOIR AND ENVIRONMENT

10.1 The stability of reservoir slopes shall be evaluated under all conditions, if any potential slope failure poses an unacceptable risk to public safety, the dam or its appurtenant structures. If necessary, such slopes shall be stabilized or the public otherwise protected from the effects of slope failure.

Type: Cnf  Conformance: Conforms
Description: Reservoir slopes are low and surrounded by largely flat ground; instability of these slopes is not a threat to the dam or spillway

10.2 The need for reservoir evacuation or emergency drawdown capability as a dam safety risk control measure shall be assessed on a case-by-case basis. If appropriate, alternative safety measures shall be taken to reduce the risks.

Type: NCp  Non-conformance: Other Procedures
Description: Emergency reservoir drawdown is a possible action in a post earthquake situation, and might also be required if the possible offtake pipe passing through the dam fails. This scenario has not been addressed by the City.
Recommend: There is no low level outlet. However, a siphon was used during the 1978 remediation and allowed a substantial lowering of the reservoir. Use of a siphon should be evaluated in detail.

10.3 The reservoir shall be monitored for potential dam safety hazards which should be Remedied or considered in the evaluation of dam safety.

Type: Cnf  Conformance: Conforms
Description: The reservoir is monitored, with action as required.
APPENDIX B

DAM INSPECTION REPORT, JULY 2003
1 Introduction

Middle Chase dam was inspected by H. Hawson and M. Jefferies during the afternoon 23 July 03 and the following morning 24 July 03. The purpose of the inspection was to be part of a 7-10 years Comprehensive Inspection & Review (CIR) of the dam, the last such CIR having been carried out by EBA Ltd in 1992, in accordance with regulatory requirements. The inspection was not carried out because of some identified potential problem with the dam.

The inspection comprised a walk-over of the dam and the surrounding reservoir slopes after review the previous week of documentation about the dam that was provided by the City.

The dam comprises a concrete core structure with the core exposed on the upstream side in the upper part, see Figure 1. The dam crest is part of a trail in the City’s park system and is asphalt surfaced with a pedestrian bridge crossing the spillway. Photograph 1 shows a picture of the upstream side of the dam taken during the inspection.

The dam is nearly one hundred years old, and is believed to have been constructed in about 1910 to provide a water supply to the then Harewood Colliery. The dam was evidently an engineered structure from the concrete core, but no drawings have been located showing any internal details nor is their any design or construction report. No photographs have been found of the dam in its early days.
The dam was acquired by the City in 1975 and is now part of the City’s parks. The reservoir is used for swimming and fishing, with no deliberate offtake of the impounded water.

In this memo, the usual convention is followed in which “left” and “right” refer specific sides of the dam or abutments and where the observer is looking in a downstream direction.

2 Watershed & Reservoir Conditions

2.1 Weather

The weather during both days of the inspection was bright, sunny and dry. Air temperatures were warm.

2.2 Watershed

Dry conditions had existed for well before the inspection and there was minimal inflow into the reservoir at the time of the inspection. This is consistent with the time of year of the inspection.

2.3 Reservoir

2.3.1 Level

Reservoir level is uncontrolled as the dam is operated solely as a recreation resource with a passive spillway. At the time of the inspection the reservoir was very slightly above the lowest part of the spillway crest with only a trickle of water being discharged. This level is actually below the base of the staff gauge as that gauge does not extend to the spillway crest (Photograph 2), discussed further below.

2.3.2 Debris

The reservoir is largely free of debris, although one small log was floating close to the spillway entrance (Photograph 1). There are a few dead trees standing in the upper part of the reservoir, Photograph 3.
2.3.3  Bank Stability

The banks around the reservoir only rise a few metres above the water surface, and the ground is generally flat lying. The whole surrounding area is part of the park system and readily accessible. No substantive bank erosion was observed while walking around the reservoir.

2.4  Discharges

The reservoir water is not discharged, the outlet works having been taken out of commission during remediation of the dam in 1980 (it is unclear whether the offtake works were even operable in 1980). Flows down the spillway were only a trickle during the inspection, reflecting the dry conditions in the watershed during the previous weeks.

3  Embankment Condition

3.1  Upstream Slope

The upstream earthworks lie below the normal reservoir surface (see Figure 1 and Photograph 1) and were not visible at the time of inspection.

The concrete core wall was inspected. No settlement or shrinkage cracks were seen. Although some weathering of the concrete was evident, nothing substantive to the dams function was observed.

No signs of movement in an upstream direction were evident in the dam crest.

The repairs of the hole in the upstream face that were carried out during the 1980 remediation continue to be in good condition as observed from the abutment.

3.2  Downstream Slope

The downstream slope is grassed, and this had been cut recently. Photograph 4 shows a view of the slope from the downstream toe. The grass makes it difficult to see minor cracks, but walking the slope did not reveal any sinkholes or other evidence of movement or internal erosion. No evidence of animal burrowing was observed.
There were no cracks in the path on the dam crest indicating any underlying slope movement, nor were any incipient scarps or other indications of slope instability discovered.

The slope profile shown in section on Figure 1, which was taken from the 1987 Data File, does not correspond closely with the slope geometry observed. It appears to be somewhat of a schematic figure. However, in as far as can be ascertained without a survey, the slope geometry closely matched the profiles shown on the as-constructed Drawing (Sheet 9) produced by Willis, Cunliffe & Tait following the 1980 remediation works. Using a hand-held clinometer, the upper part of the downstream slope measured 28 deg slope (= 1 on 1.9) while the rockfill covered toe area measured 20 deg (= 1 on 2.7).

3.3 Seepage/Piping

There is a pathway on the right hand side of the downstream slope (Photograph 4) and it is understood that there used to be seepage from the abutment at about 2/3 dam height. No seepage was evident here during the inspection.

There was no evidence of any concentrated seepage from the abutments and flowing into the dam slope.

At the toe of the dam, there is a zone of rockfill as shown on Photograph 5. No seepage could be seen through the rockfill. A V-notch weir, together with an extension of the spillway wall, is installed to collect seepage at the toe of the dam and is shown on Photograph 6. It is understood that this weir was installed in 1995. The seepage from this weir was measured and found to be approximately 12 ltr/s.

There were no fines evident in the seepage over the V-notch weir, with the water being crystal clear.

Some of the water reporting to the V-notch weir does not come from flow through the dam but arises instead from a leak beneath the spillway wall, illustrated on Photograph 7. About a quarter of the measure weir flow derived from this source.
The leak is at the contact between the concrete training wall of the spillway and the underlying bedrock.

No evidence was found of seepage by-passing the V-notch weir and exiting into the channel downstream of the dam. Corresponding, it is inferred that the abutments remain in excellent condition.

3.4 Crest

Photographs 8 and 9 show views of the dam crest from the left and right sides respectively. The dam crest comprises an approximately level path.

There was no evidence of erosion, subsidence, or slope movement.

4 Outlet Works

The dam was originally provided with outlet works apparently in the form of a penstock (wood stave ?) passing through the dam and controlled by a valve at the upstream end.

The offtake valve was located in the reservoir during the 1980’s remediation, and the as-constructed drawings indicate that it was left in place. The penstock was not located during the remediation, despite extensive excavations looking for it.

No evidence of a decaying pipe was seen during the present inspection, despite a specific search for such a linear depression in the downstream slope.

However, it remains true that the historical evidence is that a penstock passes through the dam and that this penstock is most likely open.

5 Spillway

5.1 Spillway Control

The spillway crest was originally formed by a concrete wall about 1 ft (300mm) or so above the bedrock and about 3 ft (1 m) wide. Photograph 10 shows a view of the right hand side of the present spillway crest, taken from underneath the footbridge. This
concrete crest has been broken out on the left hand side of the central support for the footbridge (Photograph 1). Photographs in the records of the Water Management Branch indicate that this was apparently also the situation in 1982.

The spillway entrance was clear of debris with the exception of one log that had grounded near the right hand side of the entrance, see Photograph 11.

The spillway was apparently originally equipped with stop-logs which were retained in steel channels cast into the concrete, Photograph 11. These channels are now corroded and unusable. It is understood that stop-logs have never been used by the City.

There are no gates or other devices to control spillway flow.

5.2 Channel

The spillway channel has bedrock at its base, with a concrete training wall along the full length of the right hand side but only about the top third of the left hand side, see Photograph 12. The channel was near dry during the inspection.

The channel was clear of any substantive debris that would impede flow.

The training wall on the right hand side, which it is understood was extended in 1995, has effectively contained spillway flows. There was no evidence of any erosion of the downstream shell caused by water overtopping the spillway.

There is evidence of substantial erosion of the left hand channel downstream of the concrete wall. This erosion zone is evident in Photograph 12 while Photograph 13 shows a closer view. In this area the spillway channel wall comprises the 2m or so of till over the bedrock base to the spillway and this till is being substantially eroded by storm events. Several trees have been undermined and will fall into the spillway as erosion continues.
5.3 Energy Dissipation

The spillway is not provided with any energy dissipation structure, with flows simply continuing past the dam within the original river channel. Photograph 14, which looks down the spillway, illustrates this aspect. However, the bedrock base to the channel prevents any undermining of the dam, and there is no evidence of toe or abutment erosion that could jeopardize dam safety.

5.4 Bridge

A footbridge has been provided over the spillway (Photograph 1). It is not known when this bridge was built, but it is not original. The concrete spillway sill suggests that there were originally three piers in the spillway but these were reduced to one central pier when the present bridge was constructed.

The bridge uses concrete beams and these have been placed directly on their supports without any movement bearings or other bedding. Spalling of the concrete beneath the beams is now evident in several locations (Photograph 15), although as best as could be ascertained this presents no hazard to current pedestrian traffic on the bridge.

6 Instrumentation

6.1 Condition

The dam has no piezometers, movement gauges or settlement monitoring points.

The single V-notch weir has some biofouling and could be usefully cleaned as this fouling obscures the engraved scale. However, it is understood the water levels are normally recorded using a tape to measure the depth of flow in the notch; the accuracy lost using this method is not regarded as material.

6.2 Readings

Weir flows have been recorded by the City’s inspector on a weekly and then monthly basis since 1998. These flows have not been plotted by the City, but are filed. The
flow during our inspection was near the top level of the biofouling on the wier (see Photograph 6).

7 Other Observations

The dam was reported to have been built at the end of a rail spur. A piece of rail was found embedded in the right hand wall of the spillway, Photograph 16.

HH/MGJ/mcm
NOTE

1. This figure is taken from 1987 “Data File”, but it does not match the details shown on Sheet 9 of the As-Built drawings from 1980. It appears to be a schematic section.

2. The downstream shell is what was believed to exist prior to 1980. It was largely replaced with compacted sand and gravel.

3. Elevations are arbitrary being referred to a local benchmark.

Figure 1: Schematic cross-section of Middle Chase Dam
Photograph 1: View of upstream side of dam 24 July 03 (composite photo)
Photograph 2: Left hand side of spillway beneath bridge. Notice that reservoir level staff gauge does not extend to the bedrock defining the spillway entry level.

Photograph 3: View of reservoir from dam crest
Photograph 4: Downstream slope of dam (composite photograph). Note access trail.

Photograph 5: Rockfill at toe of downstream slope
Photograph 6: V-notch seepage monitoring weir at toe

Photograph 7: Seepage beneath spillway wall
Photograph 8: View on dam crest from left

Photograph 9: View on dam crest from right
Photograph 10: Concrete weir on right hand side of spillway

Photograph 11: Old stop log channel
Photograph 12: View of spillway showing partial extent of concrete wall on left side

Photograph 13: Erosion of spillway bank on lower left
Photograph 14: View looking at base of spillway

Photograph 15: Spalling of concrete beneath bridge beam bearing
Photograph 16: Old embedded rail in right wall of spillway
APPENDIX C

INVESTIGATION OF CONDITION OF OLD OFFTAKE PIPE,
SEPTEMBER 2003
EBA Engineering Ltd.

CITY OF NANAIMO
455 Wallace Street,
Nanaimo, B. C.,
V9R 5J6

May 11, 1993
File No: 0802-82259

Attention: Wayne Hansen
Superintendent, Water Supply and Distribution

Re: Middle Chase River Dam
Assessment of Increase in Seepage

EBA Engineering Ltd. (EBA) was commissioned to assess the apparent increase in seepage observed at the Middle Chase River Dam. This letter report presents the results of EBA's study and gives recommendations for further monitoring and remedial works.

The Middle Chase River dam has a history of seepage through the embankment. In 1980, concern about the amount of water flowing through the dam prompted the City to lower the reservoir and excavate much of the downstream shell to determine the source of water. Unfortunately, the origin of the seepage could not be located, therefore a concrete wall with a 0.5m wide weir was installed at the toe to allow monitoring of the quantity of seepage.

City of Nanaimo records available from October 1983, to May 1986, indicate the depth of water flowing over the weir varied from 1" (25mm) to 1.25" (32mm). Based on a formula developed for the City from "Handbook of Hydraulics" these equated to flows of 0.13 ft³/s (3.7 l/s) to 0.18 ft³/s (5.1 l/s). There was no pattern of increased or decreased flow noted nor was the reservoir level or rainfall data recorded.

In March 1992, EBA completed an inspection of the Middle Chase River Dam as part of the Phase 1 Dam Safety Program. At this time the flow through the weir was estimated to be 5 l/s.

In May 10, 1993, B. Patrick of EBA met Al Haskins of the City of Nanaimo at the site. Monitoring records of the flow over the weir from June 1992, to May 1993, were provided. These indicated the depth of flow range from 1.25" (32mm) in July 1992, to 2.5" (64mm) in January and February 1993. These equate to flows of 0.18 ft³/s (5.1 l/s) to 0.51 ft³/s (14.4 l/s). It is noted that for a period of about a week at the end of August 1992, the depth of flow was 3.5" (89mm) of 0.85 ft³/s (24.1 l/s). This is likely due to water being spilled from the No. 1 Reservoir to remove floating debris.
It is apparent from these measurements that there has been a general increase in flow over the past 10 years. This may be due to:

- Deterioration along bedding planes in the bedrock which allows more flow.
- Loss of fines (i.e.: piping) from the embankment

An increase in the seepage was apparent from three locations:

- Near the downstream end of the concrete training wall along the right side of the spillway. The seepage between the base of the concrete wall and the rock surface appears not to have increased. However, there is significantly more flow from the rock at a point approximately 0.2m below this interface. It appears a bedding plane daylights on the embankment side of the wall and this is the source of the increased flow.

The spillway of the Middle Chase River Dam is formed by bedrock which dips down at approximately the angle of the spillway. It is evident that weathering causes separation of bedding planes and eventual dislodgement. It is likely this is the cause of the increase in seepage noted near the spillway wall, i.e. a bedding plane separation that connects the spillway to the area where the water emerges.

It is recommended that when flow in the spillway has decreased such that access is available to the spillway chute, a close inspection be carried out of the rock invert. If it is found that there is a bed separation, remedial works should be carried out to prevent further degradation.

These may include a program of installing dowels to anchor the slab of rock above the bedding plane and shotcreting to stop the flow of water along the bedding plane. It is estimated this work will cost approximately $5000 to $7000 to complete.

- Approximately mid-height of the embankment at the right abutment contact. There is bedrock visible on the abutment and the flow seems to be coming from there. Flow was noted from this area in March 1992, however the quantity appears to have increased significantly.

- Approximately 1.5m left of the source at the right abutment contact. This zone is at a similar elevation as the one from the contact, but is seeping from the rock fill.

The source(s) of the seepage at the two locations near the right abutment is not obvious. It is recommended that a monitoring system be established at each of these points.

It may be possible to install a small diameter (50mm) pipe and pack around it, such that the flow is channelled through the pipe. The quantity of flow can be measured using a graduated vessel and a stop watch.
As well, a system to collect any fines being washed out by the flow should be installed. This will consist of a settling basin such as a pail or trough. The location of this seepage is accessible to the public, therefore measures will have to be taken to protect these monitoring systems from disturbance. The installation of these monitoring systems should cost less than $500.00.

The reservoir level and daily rainfall should be recorded as part of the seepage monitoring program.

Should the monitoring indicate that the flow is not increasing and no fines are being washed out, remedial works may not be deemed necessary. However, if the flow continues to increase or it is established that fines are being eroded, it will be necessary to attempt to determine the source of the seepage and install measures to prevent further erosion.

If the source must be found, it is recommended that the abutment contact be exposed in the area of the seepage. If it is found that the seepage is flowing through fractures in the bedrock of the abutment, a graded filter can be installed at the downstream side. This will prevent further loss of material while allowing the water to escape, thus preventing a build up of pore pressure, within the abutment or embankment. As this will not stop the quantity of flow from increasing, an investigation must be undertaken to establish where the water is entering the bedding plane and then measures taken to seal this. The estimated cost of this work, excluding sealing of the bedrock is approximately $4000.00. If it is deemed necessary to investigate the source of this water and place a seal, the costs could be in the order of $10,000.00.

Should it be found that the seepage is flowing through the embankment, it will be necessary to carry out a larger scale investigation. This is likely to include a lowering of the reservoir and excavations to follow the seepage to its source, likely at the side or base of the concrete cut-off wall in the interior of the dam. Remedial measures will depend on the source of the flow. The cost of this work is difficult to predict but is likely to be in excess of $25,000.00.

EBA trusts this letter is sufficient for the City of Nanaimo’s purposes at this time. Should further information be required concerning this matter, please contact the undersigned.

Yours truly,

EBA ENGINEERING LTD.

Senior Geotechnical Engineer
EBA Engineering Ltd. (EBA) were commissioned by the City of Nanaimo to undertake a monitoring program of seepage and sediment at the Middle Chase River Dam.

During an inspection in the spring of 1993, seepage was noted from two areas on the downstream shell near the contact with the right abutment approximately mid-way up the dam. This area was at a higher elevation than seepage observed during recent inspections, therefore, in early September, 1993, a pipe was installed to collect the water from the main seepage area. As well, a sump was installed downhill of the pipe discharge in an attempt to collect any material being carried in the seepage flow.

On Wednesday, September 8, 1993, EBA began to monitor the flow from the pipe. Between September 8, and 17, there was no flow evident, refer Figure 1. However, on September 18, a flow of 6 litre/minute (L/min) was measured. Between September 18, and October 3, the flow ranged from 0 to 24.0 L/min.

The amount of sediment was difficult to estimate as the discharge from the pipe did not flow directly into the sump. Instead it flowed for about 1m over the dam fill, therefore, fines may have been picked up after the flow discharged from the pipe.

On September 28, 1993, R.A. Patrick, of EBA met Wayne Hansen of the City of Nanaimo on site to discuss the seepage. It was agreed that the sump should be relocated to collect the water directly from the pipe and that another pipe should be installed in the second area of seepage which was to the left of the first one.

A plastic jug was placed at the end of the first pipe such that the discharge from the pipe flowed into it. Subsequently, the sediment collected was dried and weighted. The weight of sediment on September 28, and 29, was 24.5 and 23.1 grams respectively. On October 2, 189.5 grams of material was collected. The sediment comprises fine to coarse sand with a trace of fine gravel.
Around October 4, 1993, a second pipe was installed and flows of 30 L/min were measured from it on October 4, and 5, 1993. The flow from the first pipe decreased and was measured at 1 L/min on October 5, 1993.

On October 6, 1993, B. Patrick and W. Hansen met with B. Bugslag of the BC Ministry of Environment, Dam Safety Section. The Dam Safety Section had inspection records dating back to the late 1970's and there were several references to heavy seepage from the areas currently being monitored. Therefore, it is evident the seepage is not new, but intermittent.

It is also noted that heavy seepage from this area was noted in a 1978 Golder Associates report and the seepage stopped when the reservoir level dropped less than 0.5m.

The source of the flow is possibly to be one or more of:

- A crack in the concrete cut-off: Unlikely as the upper 1m of the wall is visible and no distress is evident.

- Degradation of the concrete: Not likely as photographs from remedial works carried out in 1980 indicate the concrete was in reasonable condition.

- A gap between the cut-off and the abutment: There are no construction records available and no inspections of this contact have been carried out recently, therefore, it is unknown how this connection was made.

- An open joint in the rock which forms the abutment: The dip of the bedding in the bedrock is downstream.

Based on the available information, it appears the most likely source of the seepage is an open joint in the bedrock. If a plane is projected up from the seepage area at approximately the dip angle of the bedrock, it emerges just below reservoir level. The intermittent and variable nature of the flow could be due to fines and/or debris partially infilling the joint as well as fluctuations in the reservoir level. It is also noted that there is another area of seepage on the right abutment at a lower elevation. This location would also coincide with the dip of the bedding extended downward from just below reservoir level.

As discussed, EBA feels that the recent increase in flow and the volume of sediment are signs that the problem could be worsening and recommend continued monitoring. This will include installing a more permanent sump at the discharge of the two pipes to enable the volume of sediment to be measured. As well, it is recommended that a V-notch weir be installed at the base of the dam to allow measurement of the total seepage quantity. If fines continue to be washed out, consideration should be given to placing an inverted filter over the area of seepage to restrict the movement of materials.
EBA trusts this information is sufficient for the City of Nanaimo's purposes at this time. Should there be any question regarding this matter, please contact the undersigned.

Yours truly,

EBA ENGINEERING LTD.

R.A. Patrick, M.Sc., P.Eng., Manager, Vancouver Island

smo
ATTENTION: Wayne Hansen
Superintendent, Water Supply and Distribution

RE: Middle Chase River Dam
Seepage Monitoring

Please find attached a graph illustrating the results of the seepage monitoring during October, 1993 at the Middle Chase River Dam.

The flow from the two pipes installed near the right abutment contact has reduced over the past 2 weeks from a total flow of approximately 50 litres/minute to a total flow of about 20 litres/minute. The amount of material being carried in the seepage has decreased significantly from 20 gms/day to less than 10 gms/day.

Based on the recent results which indicate the flow rate and the material loss have decreased, EBA concludes that the frequency of monitoring can be reduced to 3 times/week from the current daily readings. However, should significant variation occur in the flow or volume of sediment, daily readings should again be taken until conditions have stabilized.

It is again suggested that a V-notch weir be installed at the base of the downstream slope to enable monitoring of the total volume of seepage.

EBA trusts the above is sufficient for your purposes at this time. Should additional information be required, please contact the undersigned.

Yours truly

EBA ENGINEERING LTD.


 Enclosure