SNC·LAVALIN

Newfoundland and Labrador Hydro
Lower Churchill Project
Pre-Feed Engineering Services

Muskrat Falls Hydroelectric Project

MF1010 – Review of Variants

FINAL

Document No:
722850-MF1010-40ER-0001-00

March 2008

SNC·LAVALIN

Newfoundland & Labrador Hydro

LOWER CHURCHILL PROJECT

TECHNICAL REPORT

MF1010 – REVIEW OF VARIANTS

Document No: 722850-MF1010-40ER-0001-00

FINAL **MARCH 2008**

Prepared by: David Robinson
Michel Tremblay

Verified by: Bertrand Masse

Approved by: Bert Peach

Certified ISO 9001

SNC·LAVALIN

TABLE OF CONTENTS

Page No.

List of Tables

Appendices

EXECUTIVE SUMMARY

The most recent previous study of the Muskrat Falls Hydroelectric Project was in a report (1999 Report) entitled "Muskrat Falls Hydroelectric Development – Final Feasibility Study, January 1999" by SNC-AGRA. A number of layout variants were identified, studied and evaluated, resulting in a shortlist of three (3) variants, 7, 10 and 11. After analysis of comparative costs, schedule and risk, Variant 7 was selected as the scheme of choice for further development and optimization. The final conceptual design of this variant was described in the 1999 Report and layout drawings were prepared.

Variant 7 comprised a tunnel diversion through the rock knoll, or "Muskrat", with control gates. An upstream cofferdam located above the upper falls and a downstream cofferdam just above the lower falls would allow the entire riverbed between the cofferdams to be unwatered. The intake and powerhouse would be close-coupled in an integrated block located immediately upstream of the lower falls. The powerhouse would contain four (4) turbine/generator units, including three (3) 206 MW propeller turbines and one (1) 206 MW Kaplan unit, for a combined capacity of 824 MW. To the north, an adjacent three-bay gated spillway would be constructed in the riverbed close to the downstream cofferdam. A north dam of RCC would connect the spillway with the north abutment, and would include an inflatable rubber dam to provide secondary spillway capacity. Closure of the river valley to the south would be by another RCC overflow dam with a fixed crest to handle partial flows in excess of the 1:1000 flood.

Access to the powerhouse would be from the north, around the rock knoll and over the top of the dams and intakes to the south abutment and then to the service bay of the powerhouse.

While all three (3) of the short listed variants appeared feasible and were not that far apart in the final evaluation, one (1) element that influenced the ranking was the lack of early access to the south bank. Since the 1999 Report, a bridge has been constructed across the Churchill River about 18 km downstream of the site, allowing the prospect of a construction road on the south shore to provide south shore access within three (3) months of start-up. This change in the site conditions merited a second look at the evaluation of the variants, which is the reason for and objective of this WTO.

For ease of comparison, the design of Variants 10 and 11 were considered to be developed similarly to that of Variant 7, utilizing the same three (3) bay gated spillway and RCC overflow dams with rubber dam.

Variant 10 has its powerhouse/spillway structures located in the south abutment, allowing their excavations to begin in the dry immediately after the south side access is afforded. No river diversion is necessary until Year 3 of the schedule, when the river flow is passed through the spillway sluices temporarily without rollways. Compared to Variant 7, this variant is characterized by large rock excavations from the approach and discharge channels, and a larger upstream cofferdam, but no diversion tunnels. Since the powerhouse /spillway structures are on the south abutment, the overflow dam is entirely on the north dam, having both a fixed crest and rubber dam.

Variant 11 in its completed state is the same as Variant 7 except that it has no tunnel diversion facility. Diversion flows are diverted through a channel excavated through the north abutment, allowing the river channel to be cofferdammed off for construction of the powerhouse and spillway. The river closure is completed during a third stage diversion when the river is passed through the spillway sluices temporarily without rollways, the diversion channel is unwatered and the north dam is completed. This variant avoids the tunnel diversion of Variant 7, and the large excavations of Variant 10, but has the diversion channel completion on the critical path. The powerhouse and gated spillway are located as they are for Variant 7.

Comparative costs rank Variant 11 as having the least cost by less than 5%, and Variants 10 and 7 within 1% of each other.

Comparing risks, Variant 10 is considered to be the least risky, followed by Variant 7 and then 11.

Comparative schedules were developed for each of the variants, with the following results:

- Variant 10 - requires the least duration, achieving first power in 55 months and full power in 61.5 months;
- Variant 11 – achieves first power in 64 months, and full power in 70.5 months;

- Variant 7 – achieves first power in 65 months, and full power in 72 months.

Including equalization costs for differences in completion and risk, Variant 10 is the overall least cost scheme by 21%.

Having all of its spillway overflow capacity on the north dam, Variant 10 has some interesting optimization possibilities including:

- Submerged gates in the spillway without rollways would increase the flow capacity and negate the requirement for the rubber dam on the north dam;

- Without a rubber dam on top of the overflow spillway, there would be no necessity for a service road access on top of the north dam. If the permanent access was relocated to the south shore, the north side access road and its excavation in the steep rock side hill of the rock knoll could be eliminated.

As the variant with the least risk, lowest overall cost and shortest schedule, the recommendation of this study is to select Variant 10 for further development.

1 INTRODUCTION

Since the last study on Muskrat Falls was completed in 1999 by SNC-AGRA, a highway bridge has been constructed across the Churchill River 18 km downstream of the site.

As this crossing could provide early access to the south bank, it could alter the ranking of three of the variants 7, 10 and 11, which were shortlisted in the above study. Alternatively, the use of a temporary bridge might also make early access possible. These alternatives will be one subject of study in WTO MF1090 "Review of Access Roads and WS&T Bridge". While the final result of that study may not be known within the time period of this study, it is considered that the temporary construction road along the south shore from the bridge to the site could be completed within a three (3) month period. Early indications also indicate that a temporary bridge located above the upper falls may also be feasible, but the high winter forebay levels will require high bridge piers, and this crossing may take about five (5) months to complete. For even earlier access, temporary short term barge service on the river below the lower falls could allow access for equipment to the south bank.

The work of this WTO was undertaken on the assumption that the temporary south shore access road will be in place within three (3) months of the project start date, and that limited work on the south shore could begin earlier, if required, utilizing barge access.

2 SCOPE

A layout and cost review study was requested by Hydro of the variants presented in the 1999 Report, particularly Variants 7, 10 and 11. This review was to confirm the optimum layout for the project, based on:

a) The cofferdam requirements for Variant 7 in the first season. This may require some re-optimization to minimize cofferdam requirements;

b) The current road access and spillway bridge requirements, considering the recently completed highway bridge across the Churchill River, approximately 18 km downstream of Muskrat Falls;

c) Capital costs - the capital costs would include for relative schedule and risk differences among the variants.

3 RESERVOIR AND FLOOD CRITERIA

3.1 GENERAL

Reservoir and flood criteria common to all variants and alternatives are included in Appendix A.

3.2 ICE STUDY

The ice study included in Section 4.4 of the 1999 Report was reviewed and the recommendations for a minimum forebay level during the winter months of elevation 24 m, as noted in Appendix A, was adopted as part of the diversion criteria for this study.

4 OVERVIEW OF THE VARIANTS

In the 1999 study, Variants 1, 2, 4, 5, 6 and 9 were found to have either an elongated schedule due to difficult cofferdam requirements or temporary bridge construction, high head losses, very complex tunnel and underground arrangements or high expected costs, so were not considered for further study. Nothing has changed in the interim period, which would appear to alter this earlier assessment.

Variants 3 and 8, originating from the 1980 study, were replaced by variant 7 in the 1999 study, so were not considered further.

Variants 7, 10 and 11 were found to be of sufficient interest for further study and preliminary cost estimates and schedules were prepared for each.

For the remaining three (3) Variants 7, 10 and 11, all three (3) were conceived having access from the north bank. Variants 10 and 11 were penalized in time and cost, in part due to the requirement for constructing access to the south bank, and thus were less desirable than Variant 7, which became the scheme of choice.

Now, with the new highway bridge in place downstream of the site providing an early access route to the south bank, Variants 10 and 11 will be compared to Variant 7. As Variant 7 was developed further and optimized in the 1999 study, the final scheme as recommended in the 1999 Report is noted as the “optimized Variant 7”, and will be considered as the base case.

The stabilization of the north spur was described in the 1999 Report and involves stabilization of slopes of the spur of land connecting the rock knoll with the north bank of the river. As this work is common to all variants, it will not be included in this review.

Similarly, the powerhouse is as described in the 1999 Report and is common to all variants. The intake and powerhouse would be close-coupled in an integrated block located immediately upstream of the lower falls. The powerhouse would contain four (4) turbine/generator units, including three (3) 206 MW propeller turbines and one (1) 206 MW Kaplan unit, for a combined capacity of 824 MW.

5 VARIANT 7

5.1 BASIC CONCEPT

This variant will be considered the “Base Case” against which Variants 10 and 11 will be compared.

This layout of the diversion facilities and that of the permanent structures are shown on two (2) Variant 7 drawings in Appendix E. Permanent facilities consist of an integrated intake and powerhouse block containing four (4) units with a combined capacity of 824 MW, and an adjacent gated spillway, all situated in the main river channel above the lower falls. Dams on either side would connect the structures to the abutments. In this scheme, the powerhouse would be located south of the spillway, in the deeper part of the riverbed, to minimize excavation.

Diversion of the river during construction would be provided by a cofferdam located upstream of the upper falls and diversion tunnels through the rock knoll. The use of diversion tunnels, with operable inlet gates, would provide more positive control of the upstream water level during the winter and, therefore, eliminate the frazil ice problem downstream and the risk associated with flooding of the construction site. This would enable a lower cofferdam to be used downstream.

Access for construction would be from the north, via a temporary road along the shoreline of the rock knoll. Access across the river would be over the upstream cofferdam and via a temporary construction road along the right bank and/or the riverbed. Permanent access would be via a road around the rock knoll and across the dam.

5.2 OPTIMIZED CONCEPT

Following the selection of Variant 7 in the 1998 study, this variant was developed further and optimized as in the following subsections.

5.2.1 General Description

Variant 7 would comprise a close-coupled power intake and powerhouse block and an adjacent three-gated spillway structure located in the riverbed just upstream of the lower falls. The spillway, on the north side of the powerhouse, would be connected to the north abutment by the north dam, of RCC construction with a five-section rubber dam on top for secondary spillage control. The river closure would be completed south of the powerhouse with the south dam, of RCC construction and a fixed crest overflow weir to pass flows in excess of a 1:1,000 event. A roadway bridge would top both the north and south dams, providing service access to the rubber dam and a permanent access from the north abutment to the south and to the powerhouse service bay.

5.2.2 Diversion Facilities

Two (2) unlined diversion tunnels would carry the river discharge through the rock knoll, and bypass the construction area. Intake gates at the head of the diversion tunnels would regulate the tunnel flows and the level of the headpond. By maintaining high headpond levels in winter, a stable ice cover would form upstream of the falls, which would minimize formation of frazil ice and the resulting ice dam downstream of the lower falls.

The tunnels would be inverted “U” shaped, 11 m wide by 10 m high, and each would have a separate intake structure with twin gates 6.6 m wide by 17 m high.

Following completion of the diversion tunnels and controls, the river would be diverted through the tunnels by an upstream cofferdam located just above the upper falls. This cofferdam would be built in two (2) stages, stage 1 to elev. 20, then stage 2 to elev. 31.5 m.

A downstream cofferdam at the crest of the lower falls would prevent flooding of the main construction area from downstream. This cofferdam would have a crest elevation of 7 m. Normal tailwater level would be elev. 3 m. As the cofferdam would be located on the edge of a slope downstream into a scoured hole in the tailpond, a small concrete “toe-dam” would be required to contain the cofferdam and stabilize the toe of the fill material.

For this variant, the river would be diverted after the peak flood of Year 2 through to the spring of Year 5, including two (2) years of peak floods and three (3) winter periods.

5.2.3 Reservoir Control Facilities

Maximum operating level of the forebay would be at elevation 39.0 m, and maximum flood level (PMF) would be at elevation 44.0 m.

The reservoir control facilities were optimized for least cost and would consist of:

- Three-bay gated spillway located adjacent to and north of the powerhouse, designed to pass 49% of the PMF with a 5 m surcharge. It would be No. 1 in the sequence of operation;
- An overflow RCC dam from the north abutment to the gated spillway, fitted with five (5) sections of rubber dam crest, designed to pass 26% of the PMF. It would be No. 2 in the sequence of operation;
- An emergency overflow dam on the south side of the powerhouse, designed to pass 25% of the PMF. It would be No. 3 in the sequence of operation.

The gated spillway would have three (3) vertical slide gates 13.75 m wide by 20.2 m high, set on a raised rollway with its crest at elevation 18.8 m, 13.8 m above the base slab. Top of the gates in the closed position would be at elevation 39.0 m. The top of the gate in the closed position would need to be increased to 39.5 m in order to have some freeboard, but for this comparative study, will be left at elevation 39.0 m.

The rubber dams of the north dam would be 2.4 m high when inflated to a crest elevation of 39.5 m, and at elevation 37.1 m deflated. There would be five (5) sections of the rubber dams each 33.5 m long, with a total crest length of 167.5 m.

The south dam would be used as an emergency overflow dam, having a crest at elevation 40.0 m, and a length of 370 m.

For floods up to the 1 in 1,000 year occurrence interval, only the gated spillway and the north spillway would be required to pass the flows.

5.2.4 Approach and Discharge Channels

Approach channel velocity would not exceed 0.65 m/s, in order to facilitate the formation of an ice cover.

Discharge channel velocity would be 1.5 m/s.

5.2.5 Access

Access to the site would be provided by a permanent access road from the north, which would run along the south side of the rock knoll excavated in the rock side-hill. The road would cross over the dams, spillway and intake structures to the right bank and then swing around to the powerhouse.

5.3 FLOOD HANDLING CHARACTERISTICS

The flood handling characteristics of the optimized Variant 7 are shown in Table 5-1, from the 1999 Report.

Table 5-1: Flood Handling Characteristics

Variant Option	Case	Flood Discharge Through or Over (m^3/s)				Flood Level (m)
		Gates	North Dam	South Dam	Total	
7	PMF	10,800	5,800	5,500	22,100	44.0
	Q_{1000}	8,450	1,650	-	10,100	40.0
	Q_p	2,667(1)	-	-	2,667	39.2
	Q_p	98(2)	2,214(3)	355	2,667	40.6

1. Flow through one gate.
2. Flow over top of closed gates, with powerplant out of service.
3. All five (5) sections of rubber dam deflated.

Note:

- PMF = 22,100 m^3/s
- Q_{1000} = 10,100 m^3/s
- Q_p = 2,667 m^3/s (Plant rated discharge)

The main flood handling facilities would be the gated spillway and the north overflow dam (in Variants 3 to 6). The diversion tunnels and south overflow dam are considered as emergency spillways that would only be required for floods greater than Q_{1000} up to the PMF.

The values in the Table 5-1 were verified using the discharge curves in Appendix B, and were found to be in close agreement.

5.4 CONSTRUCTION PLANNING AND SCHEDULE

5.4.1 General

The Project Master Schedule included in the 1999 Report was reviewed and updated using current methodologies and to ensure it is compatible with the schedules for Variants 10 and 11. The detailed schedule is included in Appendix C.

A project start date of January 1 of Year 1 has been assumed. This means that some tendering of long lead items would have begun ahead of that date, but construction on site would not begin until May or June of Year 1. No winter work has been considered for any of the variants.

Only elements of the construction plan which are not common to all variants are included below.

The critical path is through the diversion tunnels, then the powerhouse and unit erection.

5.4.2 Schedule

Year 1

- North bank access road is completed;

- Work on the portals and tunnels begin in September;
- Normal flows in river channel, and normal ice build-up in winter expected.

<u>Year 2</u>

- Completion of diversion facilities, cofferdams constructed – river diverted;
- Overburden excavation for the north and south RCC dams;
- Rock excavation for the spillway and powerhouse;
- Headpond level controlled over the winter by diversion tunnel gates to allow ice formation and control of frazil ice.

<u>Year 3</u>

- North dams – RCC complete, conventional concrete in progress;
- Rock excavated for spillway and powerhouse;
- Spillway concrete near completion;
- Intake/powerhouse in progress;
- Diversion facilities to pass summer flood (first year);
- Headpond level controlled over the winter by diversion tunnel gates to allow ice formation and control of frazil ice (second year).

<u>Year 4</u>

- Dams complete;
- Spillway complete;
- Powerhouse in progress;

- Diversion facilities to pass summer flood (second year);
- Headpond level controlled over the winter by diversion tunnel gates to allow ice formation and control of frazil ice (third year).

Year 5

- Powerhouse complete;
- Impound reservoir;
- Unit 1 commissioning complete in April (65 months).

Year 6

- Full commercial operation in January (72 months).

5.5 COMPARATIVE COST ESTIMATE

An updated comparative cost estimate was prepared based on the project estimate in the 1999 Report. The unit prices were updated to 2007 base year as part of the database update from WTO GI1060, “Review of Structure Layouts and Interfaces”.

The detailed comparative cost estimate is included in Appendix D.

6 VARIANT 10

6.1 DESIGN CONCEPT

6.1.1 General Description

Variant 10 has essentially the same layout of the dams, spillway and powerhouse as for the optimized Variant 7 except that in place of the diversion tunnels the river would be diverted through the spillway channel temporarily without rollways, and the spillway and powerhouse would be shifted out of the river channel and onto the south abutment. The diversion layout and the permanent facility layout for this variant are shown on two (2) Variant 10 drawings in Appendix E - Drawings.

The objective of this variant is to allow for an early start on the spillway and powerhouse structures by locating them on the south abutment, which would require little or no cofferdamming, and would avoid the requirement for a tunnel diversion. It would require an early access to the south bank of the river either by a temporary bridge near the upper falls, or more likely by a construction road along the south bank from the new highway bridge 18 km downstream. While the temporary access issue will be the subject of a study in WTO MF1090, Review of Access Roads and WS&T Bridge, it is considered reasonable at this time to assume that unrestricted access will be available on the south bank of the river at the site within three (3) months of project start. To allow selected work such as the overburden and rock excavation to begin even earlier, the use of a barge would allow equipment to be transported across the tailpond section of the river until the permanent access is in place.

For Variant 10, permanent access may be from the north or south. If it is from the north, then the overhead bridge crossing the north dam would be sized for the heaviest load for replacement of equipment in the future. If it is from the south, then no overhead bridge would be required over the fixed crest portion of the north dam, and the bridge over the rubber dam could be a lighter service bridge solely for rubber dam maintenance. In addition, the costly high level access road excavated in the southern rock slope of the Muskrat would not be required.

The spillway facilities comprise:

- A three (3) bay gated spillway similar to that of Variant 7, except that it will be constructed without rollways for the diversion period, then have the rollways constructed one-by-one prior to or immediately after impounding;
- A longer north RCC dam than that of Variant 7, located on the north side of the gated spillway, containing the same rubber dam as Variant 7 and seven (7) fixed crest bays 33.5 m long at elevation 39.15 m.

The relatively short south abutment dam, connecting the powerhouse to the south abutment will not be an overflow dam.

6.1.2 Diversion Facilities

For Variant 10, the diversion would only be required for one (1) year beginning after the peak flood of the third year, since the spillway and power facilities could be constructed in the dry on the south abutment and only one (1) year would be required to construct the RCC dam across the river channel. This means that a construction flood with a 20-year recurrence interval may be used. However at the time of writing this report the magnitude of this flood was not known, so the 40-year interval flood values were used.

During construction of the spillway and powerhouse facilities in the south abutment, the river would continue in its natural state in summer and winter. The natural high ground of the south abutment may be sufficient to protect the works from flooding, however, it is possible that a low cofferdam may be required where the rock level is insufficient. During winter, ice-damming may cause high tailwater levels, which also may require a low cofferdam on the downstream side, although bedrock levels at the location of the rock plugs temporarily left in place at the spillway and powerhouse discharge channels may be sufficiently high to avoid this requirement.

The diversion facilities would consist of:

- an excavated approach channel;

- a spillway structure configured without the rollways, but with its permanent 13.75 m wide by 20.2 m high gates for headwater control, and gate sills at elevation 5.0 m;

- an upstream cofferdam with similar cross-section to that of Variant 7, with a crest elevation of 25 m, and located 25 m upstream of the RCC dam;

- a downstream cofferdam with similar cross-section to that of Variant 7, with a crest elevation of 7 m, but located further upstream than was allowed in Variant 7, to avoid the necessity of the concrete toe dam. This level might have to be raised to about 12 m elevation due to measured winter water levels, but it would be subject to the effectiveness of the frazil ice mitigation measures. Refer to Figure A-3 "Water and Ice Levels at Muskrat Falls Below The Lower Falls" in Appendix A;

- one (1) set of upstream spillway stoplogs to elevation 40 m and one (1) lower set for the downstream end would be required to dewater the spillway sluice to allow for the sequential construction of the rollways.

With open sluices, the upstream water level would range from about elevation 15 m to 20 m for the predicted 40 year return period floods during the "off-peak" period between 20 May and 20 July, and up to 24.5 m for the peak period flood of 5,300 m^3/sec. For the peak period flood, the cofferdam may be allowed to overtop, as the RCC dam would be as high or higher than the cofferdam. Refer to Figure A-1 "40 Year Return Period Flood Levels at Muskrat Falls" in Appendix A.

For the winter period, the forebay would be controlled at elevation 24 m with the three (3) gates partially open, for frazil ice control.

The rollways would be constructed in each sluice sequentially, before and after the peak flood period prior to impounding, or shortly after impounding. A set of stoplogs would be required to close off the U/S and D/S ends of the sluiceway to allow the rollway area to be dewatered.

For the first rollway to be constructed, two (2) sluices would be open with their bases at elevation 5.0 m. For an off-peak Q_{40} = 2,600 m^3/sec, taken from Figure A-1 in Appendix A, the headwater would rise to elevation 21 m.

For the second rollway, two sluices would be open, with one (1) sluice at elevation 5.0 m, and the other with a completed rollway at elevation 18.8 m. For an off-peak Q_{40} = 2,600 m^3/sec, the headwater would rise to elevation 26.6 m which would overtop the upstream cofferdam and the spillway gate. This would not cause any problem for the north dam, as it would be higher than the water level. To avoid flooding the rollway construction work in the unwatered bay, a set of upstream stoplogs would be required to a level above that of the forebay.

Similarly, for the third rollway, two (2) sluices would be open, both with completed rollways at elevation 18.8 m. For an off-peak Q_{40} = 2,600 m^3/sec, the headwater would rise to elevation 32 m which would over top the upstream cofferdam. As the north dam would be complete or nearly complete, this overtopping would be acceptable. The upstream cofferdam would not require removal following its use during the diversion period. For the rollway under construction, a set of upstream stoplogs would be required to a level above that of the forebay.

6.1.3 Flood Control Facilities

As for Variant 7, the maximum operating level of the forebay would be at elevation 39.0 m, and maximum flood level (PMF) elevation is 44.0 m.

The reservoir control facilities would consist of:

- a three-bay gated spillway located adjacent to and north of the powerhouse, designed to pass 49% of the PMF with a 5 m surcharge. It is No. 1 in the sequence of operation, and identical to that of Variant 7;
- an overflow RCC dam from the north abutment to the gated spillway, fitted with five sections of rubber dam crest, designed to pass 26% of the PMF. It is No. 2 in the sequence of operation, and identical to that of Variant 7;

- an overflow dam on the north side of the rubber dam, comprising seven (7) bays with fixed overflow crest at elevation 39.15 m, designed to pass 25% of the PMF. It is No. 3 in the sequence of operation assuming that the rubber dam is lowered to pass the excess flows when the reservoir threatens to exceed elevation 39.15 m.

6.1.4 Approach and Discharge Channels

The powerhouse would be the same for all variants.

The approach channel would be excavated for the full width of the units from elevation 1.0 m at the intake structure, then would rise at a slope of 3h:1v to elevation 10.0 m. The channel would continue upstream for 70 m at this elevation, and then rise at a slope of 3h:1v to the natural rock surface.

The average approach velocity throughout the approach channel would not exceed 0.65 m/s.

The tailrace channel would be excavated to the same profile as for Variant 7, which would result in an average velocity over the channel base at elevation –10 m of not more than 1.5 m/s.

6.2 SPILLWAY HYDRAULICS

The hydraulics for Variant 10 are as they were established for Variant 7 in Section 6 of the 1999 Report, and as shown below in Table 6-1.

The floods are as follows:

- PMF = 22,100 m^3/sec
- Q_{1000} = 10,100 m^3/sec
- Q_p = 2,667 m^3/s (Q_p = rated plant flow)

The main flood handling facilities are the gated spillway and the north overflow dam. The south overflow dam is considered as an emergency spillway that would only be required for floods greater than Q_{1000} up to the PMF.

Table 6-1: Flood Handling Characteristics

#	Flood	Flow m³/s	Case	Flood Flow Distribution m³/s			Flood Level m
				Gates	Rubber Dam	Fixed Crest	
1	Diversion - peak	5,300	3 bays – no rollways	5,300	Not completed		24.5
2	Diversion – off peak	2,600	3 bays – no rollways	2,600	Not completed		17.2
			Rollway Construction cases				
3	Diversion – off peak	2,600	#1 - 2 bays – no rollways	2,600	Not completed		21
4	Diversion – off peak	2,600	#2 - 2 bays – 1 rollway	2,600	Not completed		26.5
5	Diversion – off peak	2,600	#3 - 2 bays – 2 rollways	2,600	Not completed		32
6	Diversion – off peak	2,600	3 bays – no rollways	2,600	Not completed		24
	Winter control		(6 m gate openings)				
7	PMF	22,100	Completed plant – no units operating	10,800	5,800	5,500	44
8	Q_{1000}	10,100	Completed plant – no units operating	8,200	1,600	300	40
9	Q_p (Rated plant flow)	2,667	One gate operable,no PH flow, rubber dam inflated	2,667	0	0	39.2
10	Q_p	2,667	Gates closed, no PH flow	47	2,000	620	40.4

Diversion peak flow is the Q_{40} flood flow, which is based on a 5% probability of occurrence over two (2) years. However, the Variant 10 diversion is required only for one (1) peak flood season so the Q_{20} flood would be applicable. As the magnitude of this flood is not available, the Q_{40} value was used.

The maximum upstream cofferdam height was set at elevation 25 m, only 0.5 m above the forebay level resulting from the Q_{40} flood. This is slightly conservative, in view of using the Q_{40} flood value rather than that of the Q_{20} flood.

Higher forebay levels than the cofferdam crest elevation would result during the construction of the rollways; however, during this activity, the north dam would be higher than the forebay levels so this would be acceptable.

6.3 CONSTRUCTION PLANNING AND SCHEDULE

6.3.1 General

The key start date would be the same as for Variant 7, that is, January of Year 1. The critical path is through the south shore access route, the spillway and powerhouse excavation, the intake and spillway facilities to allow diversion through the spillway, then the powerhouse and unit installation.

6.3.2 Schedule

<u>Year 1</u>

- Temporary access to south shore completed;
- Excavations in progress on south abutment for spillway and intake/powerhouse;
- River flows in normal riverbed channel.

<u>Year 2</u>

- Complete south abutment excavations;
- Spillway civil works completed;
- Intake & powerhouse in progress;
- River continues in normal riverbed channel.

<u>Year 3</u>

- Intake completed with gates/stoplogs;
- Spillway complete with gate/stoplogs on base sill (no rollways);

- Cofferdams completed and river diverted through spillway after summer peak flood;
- Forebay level controlled to elevation 24 m during winter flows for frazil ice control.

Year 4

- Dams completed;
- PH civil works completed;
- River passes through spillway (no rollways);
- Impound reservoir at year's end.

Year 5

- Rollways completed sequentially;
- Unit 1 commissioning completed in July (55 months).

Year 6

- Full commercial operation in February (61.5 months).

6.4 COMPARATIVE COST ESTIMATE

The detailed comparative cost estimate is included in Appendix D.

7 VARIANT 11

7.1 DESIGN CONCEPT

7.1.1 General Description

As presented in the 1999 Report, this layout is similar to Variant 7 except that diversion of the river would be via an open channel, excavated through the point of land on the north bank, adjacent to the lower falls. One (1) drawing is included in Appendix E to illustrate the three-stage cofferdam configuration. The layout of the permanent structures is the same as for Variant 7.

The layout would comprise an integrated intake/powerhouse and a spillway located in the river channel, above the lower falls. The spillway would be located to the north of the intake/powerhouse. Closure of the river valley would be provided by an RCC overflow dam on the south bank and an RCC dam on the north bank with a rubber dam. The RCC dam on the north bank would be constructed across the diversion channel.

Access for construction would be from the north, via a road around the rock knoll. Access across the river would be via a construction road on the south shore from the existing highway bridge 18 km downstream of the site, and later a construction bridge over the diversion channel and over the upstream cofferdam.

7.1.2 Diversion Concept and Facilities

Work would start with the excavation of a diversion channel on the north bank of the river, behind a small upstream cofferdam. The lower levels of the gravity dam monoliths and training walls would be concreted.

Once the spring flood has passed, the cofferdam would be removed and the first stage of the main upstream cofferdam would be constructed to close the river. In parallel with the second stage of the cofferdam construction, a rock spur would be constructed at the upper falls to partially control the upstream water level and the generation of frazil ice.

With the river flowing through the diversion channel, and upon completion of the downstream cofferdam, work would start on the powerhouse and spillway. The structures would be completed and the gates installed in the powerhouse and spillway. Stoplogs would be installed in the draft tubes.

In order to remove the upstream cofferdam the rollways of the spillway would be left out to enable passage of the river flow through the spillway.

After the spring flood has past, the upstream cofferdam would be lowered and as much of the fill as possible would be removed in the dry. Once this has been completed, the remaining cofferdam would be breached and a closure cofferdam would be constructed in the diversion channel.

Once the cofferdams are removed and the gravity dam constructed in the diversion channel, work would start on the spillway rollways. Each rollway would be dewatered by installing temporary upstream and downstream stoplogs. The river would continue to flow through the remaining spillway openings and slowly rise as the number of openings is reduced. When the final opening is closed for concreting the river will impound to a high enough level to allow the first unit to be wet tested.

In this layout, there would be limited control of the upstream water level, provided by constriction of the river flow by an armored rockfill spur constructed above the upper falls. The limited control of the upstream water level would not eliminate frazil ice generation and, therefore, frazil ice would still be a concern. The risk of flooding of the construction areas from downstream, due to ice damming in the downstream river channel would necessitate a downstream cofferdam, which for Variant 7 was set at an elevation of at 7 m. Of concern to this variant are recent water level readings during the winter season which show levels up to 11 m elevation downstream of the lower rapids. This means that the downstream cofferdam as previously designed for Variant 7 will be insufficient to retain these levels.

With the diversion channel 45 m wide, and a sill at elevation 5.0 m, the upstream water level during a design construction flood event would be at elevation 24 m. The main upstream cofferdam would be constructed to elevation 25 m.

7.1.3 Reservoir Control

As for Variant 7, the maximum operating level of the forebay would be at elevation 39.0 m, the 1000-year flood level would be at elevation 40 m, and the maximum flood level (PMF) would be at elevation 44.0 m.

Winter control of the water level above the upper falls to facilitate the formation of an ice cover for frazil ice mitigation by means of the rock groin was not considered reliable, and would constitute a major risk to this variant.

The completed spillway facilities would be the same as for Variant 7.

7.1.4 Power Facilities

The powerhouse would be the same for all variants. As the powerhouse would be located in the same location as for Variant 7, the approach and tailrace channels would also be the same.

7.2 SPILLWAY HYDRAULICS

The Variant 11 spillway facilities on completion would be identical to that of Variant 7.

During construction, the gated spillway would be constructed initially without rollways to allow diversion flows to pass through the spillway during stage 3 of the diversion sequence, when the diversion channel is cofferdammed off for completion of the north dam.

The hydraulic characteristics of the spillway during this period would be identical to that of Variant 10 during its diversion period.

7.3 CONSTRUCTION PLANNING AND SCHEDULE

7.3.1 General

The critical path of the construction is through the diversion channel, stage 2 diversion cofferdams, spillway and intake, powerhouse civil works, stage 3 diversion through the spillway for completion of the north dam and the erection of the turbine/generators.

7.3.2 Schedule

Year 1

- Access roads on north shore complete;
- Diversion channel and concrete abutment dam completed behind Stage 1 cofferdam.

Year 2

- Stage 2 cofferdams completed and river diverted through diversion channel;
- Foundation excavations for the spillway and powerhouse in progress.

Year 3

- Foundation excavations complete;
- Spillway structure complete, except for rollways;
- PH/Intake structures in progress;
- South dam construction begun;
- River flows through diversion channel.

Year 4

- Intake compete;
- PH in progress;
- Stage 3 cofferdams constructed to divert river through spillway after summer flood;
- North dam begun;
- South dam completed.

<u>Year 5</u>

- Dams complete;
- PH structure complete;
- River flows through spillway.

<u>Year 6</u>

- Unit 1 ready for commercial operation in May (64 months);
- Full commercial operation near end of November (70.5 months).

7.4 COMPARATIVE COST ESTIMATE

The detailed comparative cost estimate for Variant 11 is included in Appendix D.

8 REVIEW AND COMPARISON OF VARIANTS

8.1 RISKS

A subjective evaluation of the relative schedule risks of Variants 7, 10 and 11 was made, and the results are shown on the following Table 8-1. Each of the identified risks were evaluated for the potential impact on the schedule and the probability of occurrence, resulting in a relative risk for each in terms of months delay to the project schedule.

Table 8-1: Comparison of Relative Risk (Months)

Var.	Description	Coffer dams	Tunnels	PH	S'way	Dams	Rock Spur	Total
7	Diversion through tunnels		2.0					2.0
	Single stage u/s cofferdam	1.0						1.0
	Cofferdams in place for two flood seasons	1.0						1.0
	North dam constructed on dewatered riverbed					0.0		0.0
	PH & Spillway in riverbed above lower falls			2.0				2.0
	Reservoir control during diversion by gates				0.0			0.0
	Rollways to be completed before impounding				0.0			0.0
		2.0	2.0	2.0	0.0	0.0	0.0	6.0
10	Diversion through spillway, without rollways				0.5			0.5
	Single stage u/s cofferdam	1.0						1.0
	Cofferdams in place for one flood season	0.5						0.5
	North dam constructed on dewatered riverbed					0.0		0.0
	PH & Spillway on south abutment above lower falls			1.0				1.0
	Reservoir control during diversion by spillway gates	0.0			0.0			0.0
	Rollways to be completed after impounding				0.5			0.5
		1.5	0.0	1.0	1.0	0.0	0.0	3.5
11	Diversion through diversion canal				0.0			0.0
	Three stage cofferdam arrangement	2.0						2.0
	Cofferdams in place for two flood seasons	1.0						1.0
	North dam constructed on dewatered riverbed in 2 parts					0.5		0.5
	PH & Spillway in riverbed above lower falls			2.0				2.0
	Reservoir control during diversion by u/s groin						1.0	1.0
	Rollways to be completed after impounding				0.5			0.5
		3.0	0.0	2.0	0.5	0.5	1.0	7.0

From the above table of relative schedule risks, Variant 10 is assessed as having the least risk, followed by Variant 7 then Variant 11 with the highest risk. The total relative risk, or delay to the project, was included as a cost item in Table 8-5, Comparison of Relative and Total Project Costs.

8.2 SCHEDULE

The comparative implementation schedules included in Appendix C for the three variants, all beginning with a January 1 of Year 1 project release date, show the duration to first power as follows:

- Variant 7 65 months
- Variant 10 55 months
- Variant 11 64 months

While all of these schedules could be optimized to achieve a shorter duration to first power, particularly if winter work was employed, they have been developed on the same basis with a similar degree of float in each.

8.3 ADVANTAGES/DISADVANTAGES

Tables 8-2, 8-3 and 8-4 follow showing the identified advantages and disadvantages of Variants 7, 10 and 11 respectively.

Table 8-2: Variant 7 - Advantages and Disadvantages

Description	Advantages	Disadvantages	Risk	Schedule
- Close-coupled intake and powerhouse, three bay gated spillway located in the riverbed above lower falls.	-Frazil ice control with tunnel diversion and high water levels above U/S cofferdam. U/S water level higher than minimum recommended for frazil ice control.	- Requires tunnel diversion conduit	- Key risk factor to cost and schedule are the underground works, which are on the critical path.	65 months to first power.
- Diversion cofferdam upstream of upper falls with flows via tunnels through rock knoll.	- Simple diversion cofferdam arrangement. - PH and spillway structures remote from diversion discharge.	- Cannot begin to work on PH/Spillway until tunnel diversion and cofferdams complete.	- Higher risk than Variant 10, but slightly less than Variant 11.	72 months to full commercial operation.
- North dam of RCC has overflow rubber dam, constructed on north abutment.	- Single unwatering closure required for PH/Spillway construction.	- Rock plug required at tunnel outlet during construction.	- D/S cofferdam may be overtopped in winter.	
- South dam of RCC with overflow crest for >1:1,000 yr floods	- Complete unwatering of PH/Spillway during construction.	- Requires removal of a portion of the U/S cofferdam and all of D/S cofferdam.		
	- Concurrent construction of PH and Spillway - Spillway rollways constructed at the same time as base slab	- South dam is an overflow dam, so access road could be overtopped and switchyard has to go on powerhouse roof.		
	- Low hydraulic headloss during operation	- D/S cofferdam would have to be higher than originally designed.		
	- Access to PH/Spillway works across riverbed	- Requires rock quarry, as rock from excavations insufficient.		
	- Possibility of rock quarry in riverbed			

Table 8-3: Variant 10 - Advantages and Disadvantages

Description	Advantages	Disadvantages	Risk	Schedule
- Close-coupled intake and powerhouse, three bay gated spillway located on the south abutment above lower falls.	- Winter U/S water level controlled to recommended level for Frazil ice control.	- Diversion discharge adjacent to powerhouse and dam construction.	- No underground works and single stage cofferdam arrangement for only one flood season contributes to least risk of all Variants.	55 months to first power.
- Diversion cofferdam upstream of structures with flows passing through spillway chute without rollways.	- During PH/Spillway excavations and first stage concrete work, river flows in natural channel. - PH/Spillway excavated in the dry on right bank	- Large quantity of rock excavation required for approach channel, PH/Spillway and discharge channels .		61.5 months to full commercial operation.
- North dam of RCC has overflow rubber dam and fixed crest weir, constructed on riverbed and north abutment.	- Rock from excavations is good quality and may be used in fills and as aggregate, reducing the cost of these elements. Possibly very little wasted.			
- South dam of RCC, no overflow.	- Low hydraulic headloss during operation. - Switchyard could be relocated from PH roof to area near PH access on south bank. - Concentration of all overflow spillage on north dam allows option of eliminating rubber dam and overhead access road bridge, and the permanent road around rock knoll. - New highway bridge allows early access to south bank. Excavation of PH/Spillway begins much earlier than other Variants.			

Table 8-4: Variant 11 - Advantages and Disadvantages

Description	Advantages	Disadvantages	Risk	Schedule
- Close-coupled intake and powerhouse, three bay gated spillway located in riverbed above lower falls as in Variant 7. - Three stage cofferdam arrangement.	- Diversion channel excavated in the dry on north abutment. - Lowest capital cost.	- Complicated three stage cofferdam required: For diversion canal excavation and abutment dam construction; Main river channel for PH/Spillway and portion of north dam; Diversion channel for completion of north dam	- Main risks relates to complicated cofferdam requirements over two seasons and risk of winter flooding due to poor frazil ice control.	64 months to first power. 70.5 months to full commercial operation.
- Diversion through excavated canal in north abutment. - Reservoir control by overflow groin.		- Rock from excavations insufficient for rock fills and aggregates. Rock quarry required.	- Risk slightly more than that of Variant 7 and higher than Variant 10.	
- North dam of RCC has overflow rubber dam, constructed on north abutment.		- Poor reservoir control using overflow groin above upper falls. - River diversion through canal required over two flood seasons.	- May overtop cofferdam due to ice damming.	
- South closure dam of RCC with overflow crest for >1:1000 yr floods.		- South dam is an overflow dam, so access road could be overtopped and switchyard has to go on powerhouse roof.		
		- Cannot begin to work on PH/Spillway until diversion canal and cofferdams complete.		
		- Temporary access must be from the north. Added south shore access of limited benefit.		
		- D/S cofferdam to be higher than originally designed.		

8.4 COMPARATIVE COST ESTIMATES

The estimated total cost of the three (3) variants is shown in Table 8-5 below.

The direct costs are from the comparative cost estimate included in Appendix D.

Assumptions made are as follows:

- The rate of interest for IDC calculations is 8%;
- IDC calculation is based on the total construction cost at the half-way point;
- Generation variations are based on an annual generation of 5.53 TWh, from Section 5.5 “Final Capacity and Energy Determination” of the 1999 Report.
- Calculations for IDC and energy costs follow the same methods used in the 1999 Report.

Table 8-5: Comparison of Total Relative Project Costs

			Variant 7		Variant 10		Variant 11	
Description	**Unit**	**Unit$**	**Qty**	**Cost**	**Qty**	**Cost**	**Qty**	**Cost**
Total Direct cost	L.S.			1,258,996,000		1,246,925,000		1,199,811,000
Schedule costs								
Duration	Mon.		65		55		64	
IDC		8%		292,712,000		241,425,000		274,286,000
Lost Energy Sales								
Due to increased schedule	Mon		10	253,458,000	0	0	9	228,113,000
Relative Risks								
Potential delay	Mon		2.5		0		3.5	
IDC				20,983,000		0		27,996,000
Lost Energy Sales				63,365,000		0		88,710,000
Total Relative Project Costs				**1,889,514,000**		**1,488,350,000**		**1,818,916,000**
% variation				**0%**		**-21%**		**-4%**

Notes:

1. IDC is calculated as the accumulated compounded interest on a monthly basis using the full value of the construction cost for a period equal to half of the construction schedule.

2. Lost Energy Sales is calculated from the average monthly generation of 460,833 GWh by the number of months of schedule variation.

8.5 RECOMMENDATION

Based on the comparative costs, risks, schedule and other advantages, it is recommended that Variant 10 be selected as the basis for further development of this project.

9 FURTHER CONSIDERATIONS

If Variant 10 is accepted as the most suitable choice of the variants identified, there are a few alternatives, which may have economic, or operations benefits.

9.1 SPILLWAY GATE ALTERNATIVES

For the optimization of the spillway facilities, to be undertaken in WTO MF1050, Spillway Design Review, some different gate arrangements will be studied, including:

- Verification of the maximum size of vertical gate that is practical for this application;
- Review of alternative gate sizes and number to handle an increased flow so that the rubber dam may be eliminated;
- Review of alternative gate arrangements such as submerged gates on a flat base slab capable of passing an increased flow so that the rubber dam may be eliminated;
- Review of the practicality of utilizing a radial submerged gate in this application.

The use of the rubber dam was recommended in the 1999 study by SNC-AGRA due to its low initial cost, but there is concern over its long-term cost including possible plant shutdown costs during a future bladder replacement. Appendix F includes some recent literature on the use of rubber dams, however, we believe there would be long-term advantages for the Muskrat Falls plant to avoid their use.

9.2 ACCESS ALTERNATIVES

If the permanent access to the plant is to be from the north, then a permanent high-level road will have to be excavated around the rock knoll, connected to a bridge across the RCC dams and then to the powerhouse. In this case, a temporary access to the south shore could be:

- by a temporary bridge crossing just upstream of the upper falls and then by access road to the construction site, or

- by a temporary south shore access road from the existing bridge 18 km downstream, plus a temporary low-level access road around the rock knoll connecting the upstream cofferdam and a bridge across the spillway diversion channel.

If the permanent access road were to be from the south, along an access road on the south shore from the existing highway bridge, then that would favour making the temporary access also from the south, along the same route, and which would later be upgraded to the permanent road. No high level access road would be required to be excavated along the south side of the rock knoll. Later, after diversion, another temporary road would be required across the upstream cofferdam and on a rockfill berm around the base of the rock knoll to transport rock to the north spur, and to transport heavy equipment to the powerhouse. In the case where a rubber dam is incorporated into the north dam, only a light service bridge would be required across the north RCC dam to service the rubber dam. In the case where the rubber dam was not required, the overhead bridge could be eliminated completely.

The removal of the overhead bridge affects the spillway design in that it removes all of the required piers, increasing the length of overflow weir that could be available for spillage.

The only downside to having the permanent access from the south is a limited future capacity to transport the heaviest loads for replacement equipment across the existing bridge downstream of the project. Alternative means such as barge transport or alternate routes from the port of Cartwright may need to be considered.

APPENDIX A

RESERVOIR AND FLOOD CRITERIA

RESERVOIR AND FLOOD CRITERIA

Diversion Criteria

<u>Construction flood</u>

From Section 4.2 of 1999 Report:

- Canadian practice is to accept 5% risk per year for temporary works;
- Probability P = 1/T;
  Where T = recurrence interval;
- For a probability of 5% for a two (2) year diversion period, T= 40 years, which is a 1 in 40 year return period flood (Q_{40});
- Assuming modified flood handling procedures at Upper Churchill, the magnitude of this flood was determined to be **5,300 m^3/s**.
- Period during which this flood may occur is from 25 May to end of June.
- From about 15 May to 25 May, and from about the end of June to end of July, Q_{40} = **3600 m^3/s**.
- Balance of year, Q_{40} = **2600 m^3/s**.
- These "off-peak" values of the Q_{40} floods are useful in determining the expected flood levels for staging cofferdam construction or the construction of the rollways.
- For a one-year diversion period and 5% risk per year, the design flood would have a 1 in 20 year return period (Q_{20}). At the time of preparation of this report, no information is available on the magnitude of this flood, so for cases having a one-year diversion period, the Q_{40} flood values will be used.

Frazil Ice Control

From Section 4.4 of the 1999 Report:

- It was concluded that ice control could be reliably obtained by maintaining the upstream water level at a minimum of El. 23.0 m regardless of flow, however it was recommended that a minimum level of 24 m be adopted.

From Table 4.2 Summary of Ice Observations of the 1999 Report:

- For the period 1974 to 1992, the average maximum stage above the upper falls was observed to be **17.59 m**, and the maximum elevation was **20.13 m**.

Tailrace Rating curve

From Figure 5.2 of the 1999 Report:

- From the data on the curve, the stage/discharge relationship may be approximated by the second-degree polynomial:

- $Elev = -2.5062E\text{-}08Q^2 + 1.065E\text{-}03Q + 0.874$

Rating Curve Upstream of Upper Falls

From LaSalle Ice Study:

- $Elev = Q^{.429}/3.6808 + 10.177$

Spillway Flood Criteria

Maximum Design Flood

From Section 3.5 of Appendix B of the 1999 Report:

- Use PMF;

- Flood routing effect is negligible;

- Spillway design flow (PMF) = 22,100 m^3/s;
- Maximum design flood level at PMF = 44 m.

<u>Maximum Design Flood without Auxiliary Spill Facilities</u>

From Section 6.7.2 Design Criteria of the 1999 Report:

- 5% lifetime risk of occurrence based on a 50 year life = 1 in 1,000 year flood;
- Q_{1000} = 10,100 m^3/s;
- Maximum flood level = 40 m.

<u>Maximum Plant Flow Handling</u>

From Section 6.7.2 of the 1999 Report, noted to apply where economically feasible:

- Capability to discharge the maximum plant flow (Q_p = 2,667 m^3/s) over closed spillway gates plus auxiliary spill facilities without excessive rise in reservoir operating level in an emergency situation.

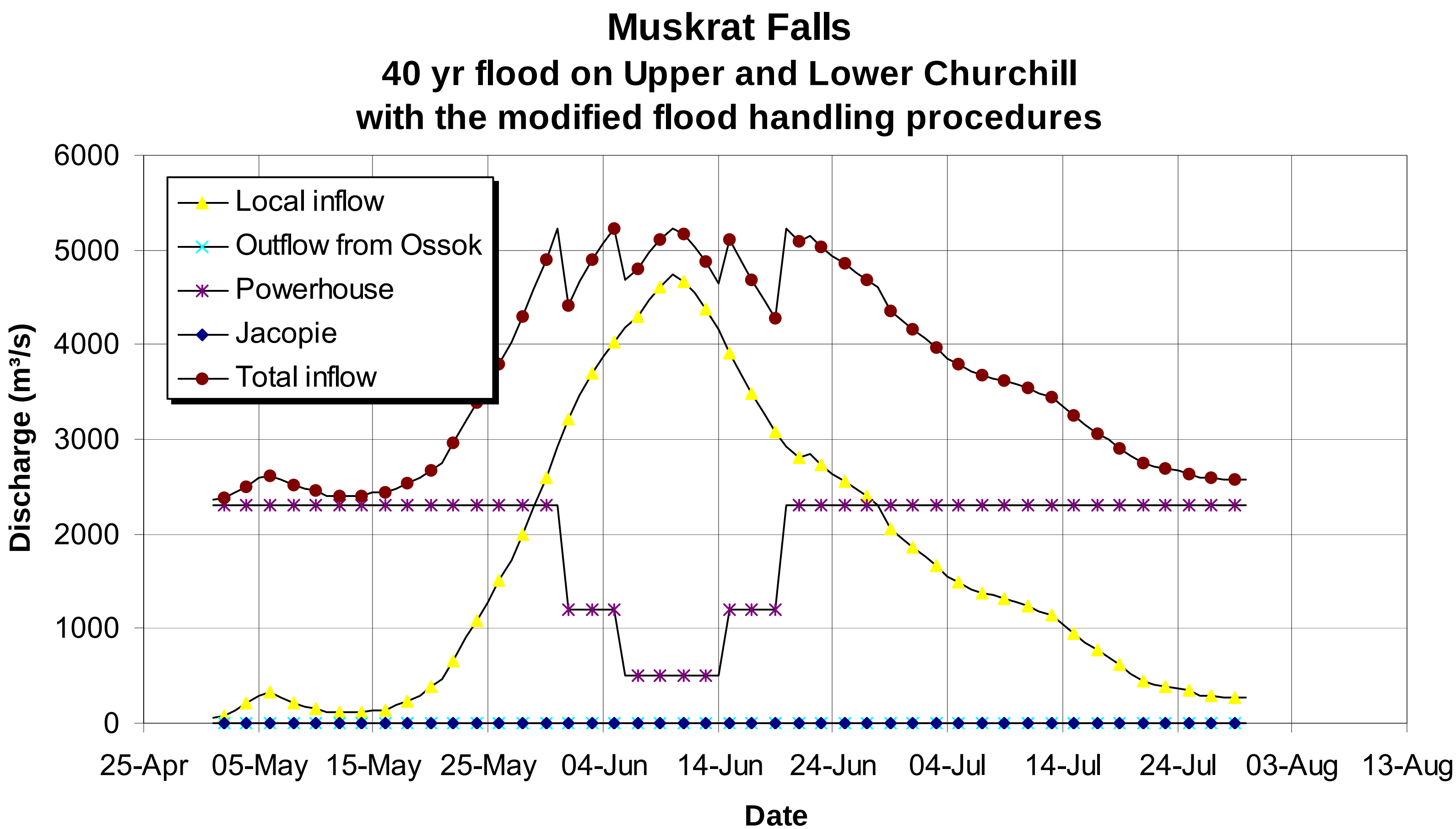


Figure A-1: 40 Year Return Period Flood Levels at Muskrat Falls

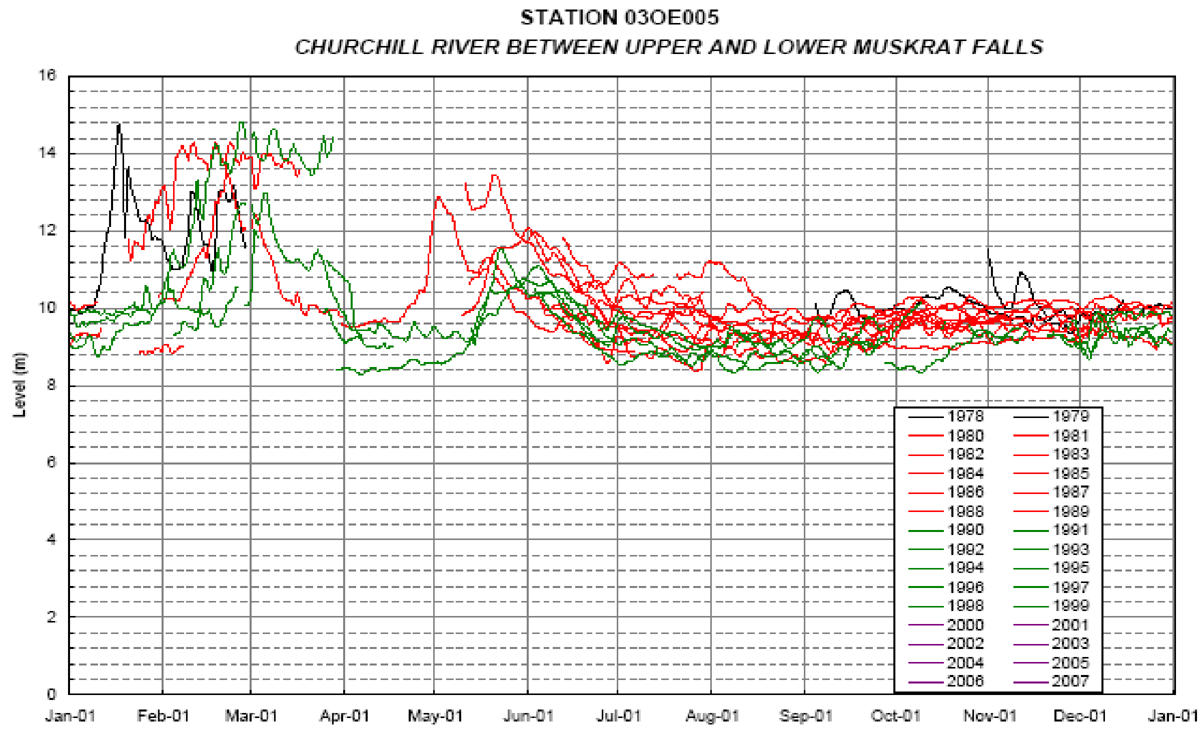


Figure A-2: Water and Ice Levels at Muskrat Falls between the Upper and Lower Falls

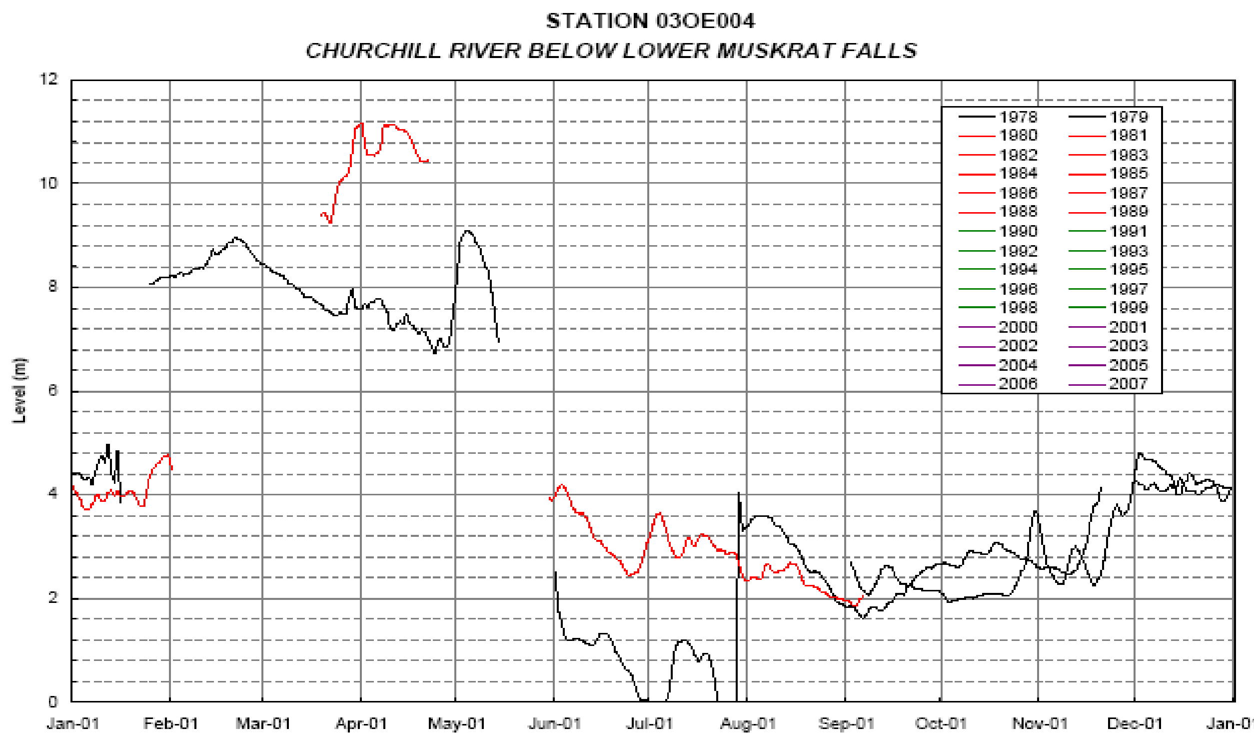


Figure A-3: Water and Ice Levels at Muskrat Falls below the Lower Falls

APPENDIX B

SPILLWAY HYDRAULICS

Maximum Flood Level (MFL) shown as dashed red line. Full Storage Level (FSL) and Low Storage Level (LSL) shown as dashed green lines.

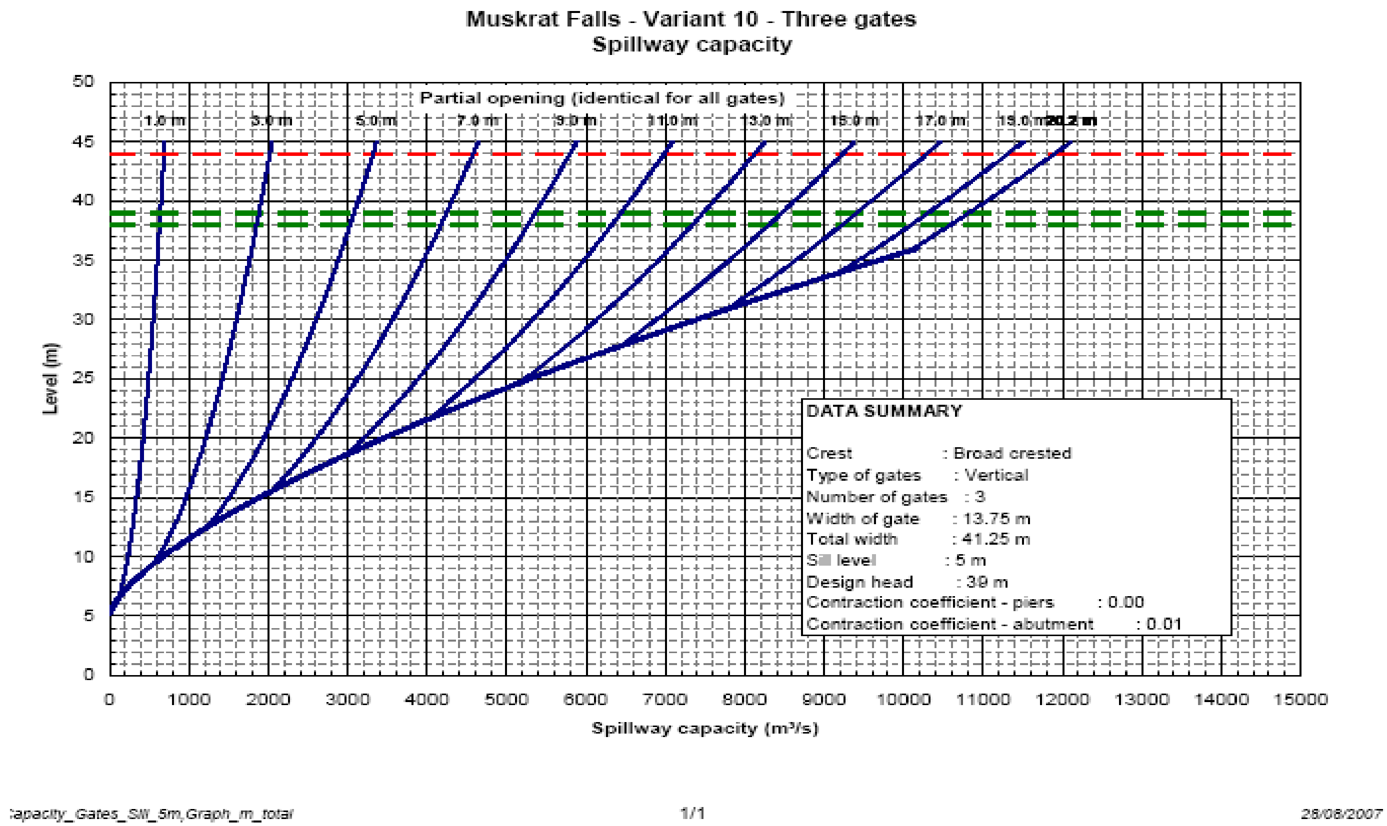


Figure B-1: Variant 10 – Three Gated Spillway Rating Curve, No Rollways

Maximum Flood Level (MFL) shown as dashed red line. Full Storage Level (FSL) and Low Storage Level (LSL) shown as dashed green lines.

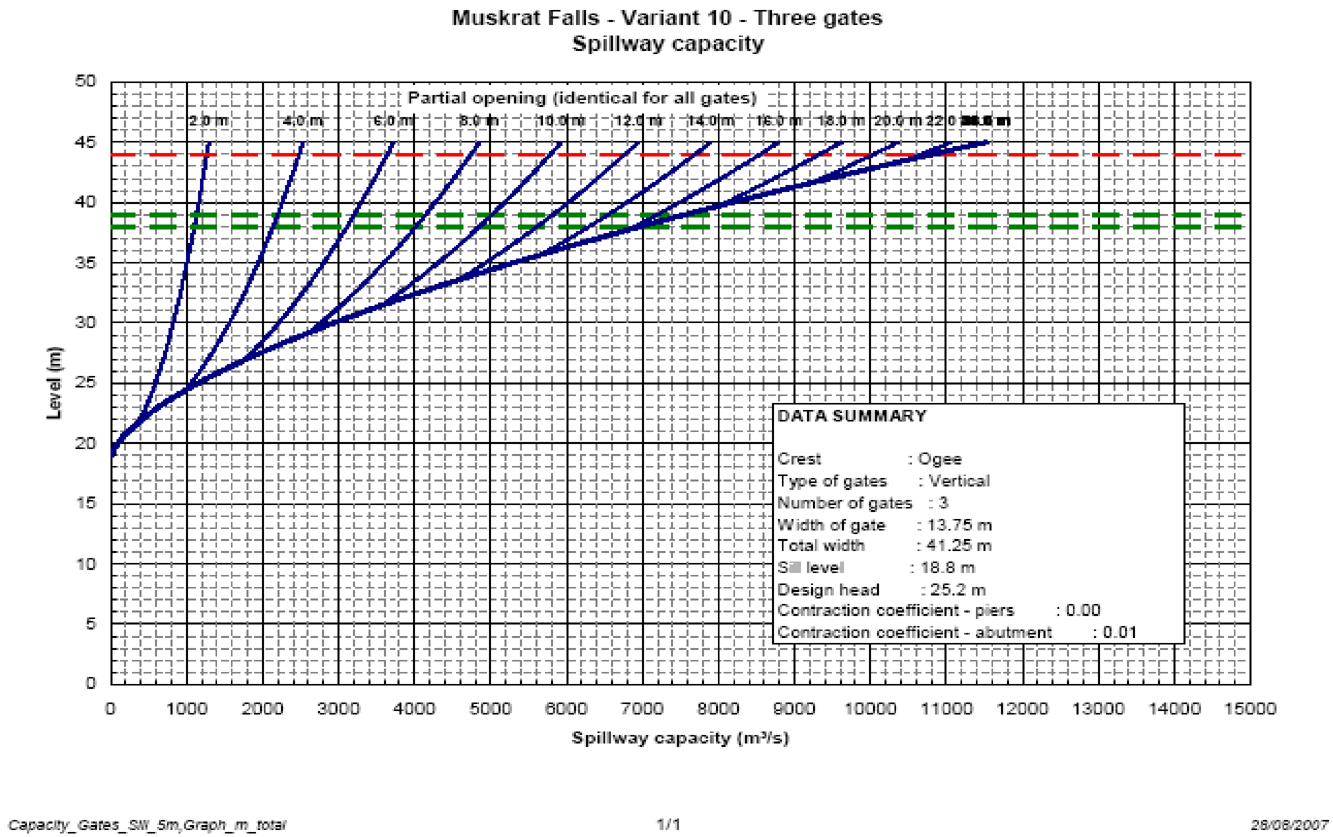


Figure B-2: Variant 10 – Three Gated Spillway Rating Curve, with Rollways

Maximum Flood Level (MFL) shown as dashed red line. Full Storage Level (FSL) and Low Storage Level (LSL) shown as dashed green lines.

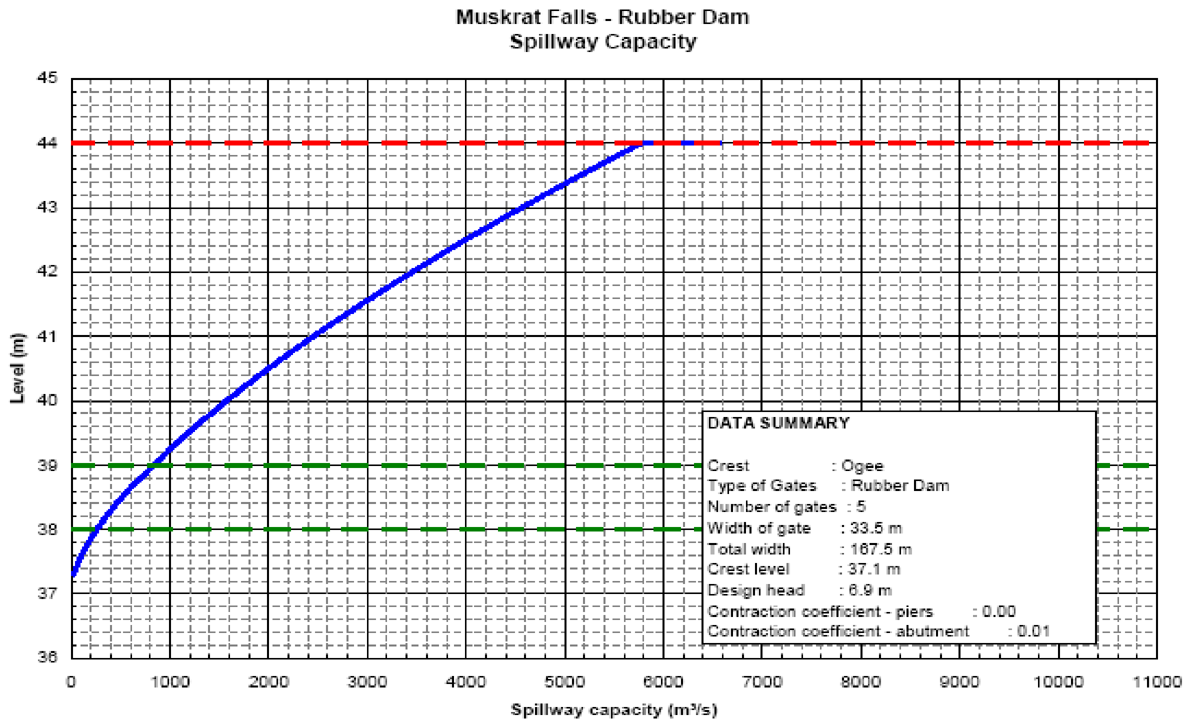


Figure B-3: Variant 10 – Rubber Dam Rating Curve

Maximum Flood Level (MFL) shown as dashed red line. Full Storage Level (FSL) and Low Storage Level (LSL) shown as dashed green lines.

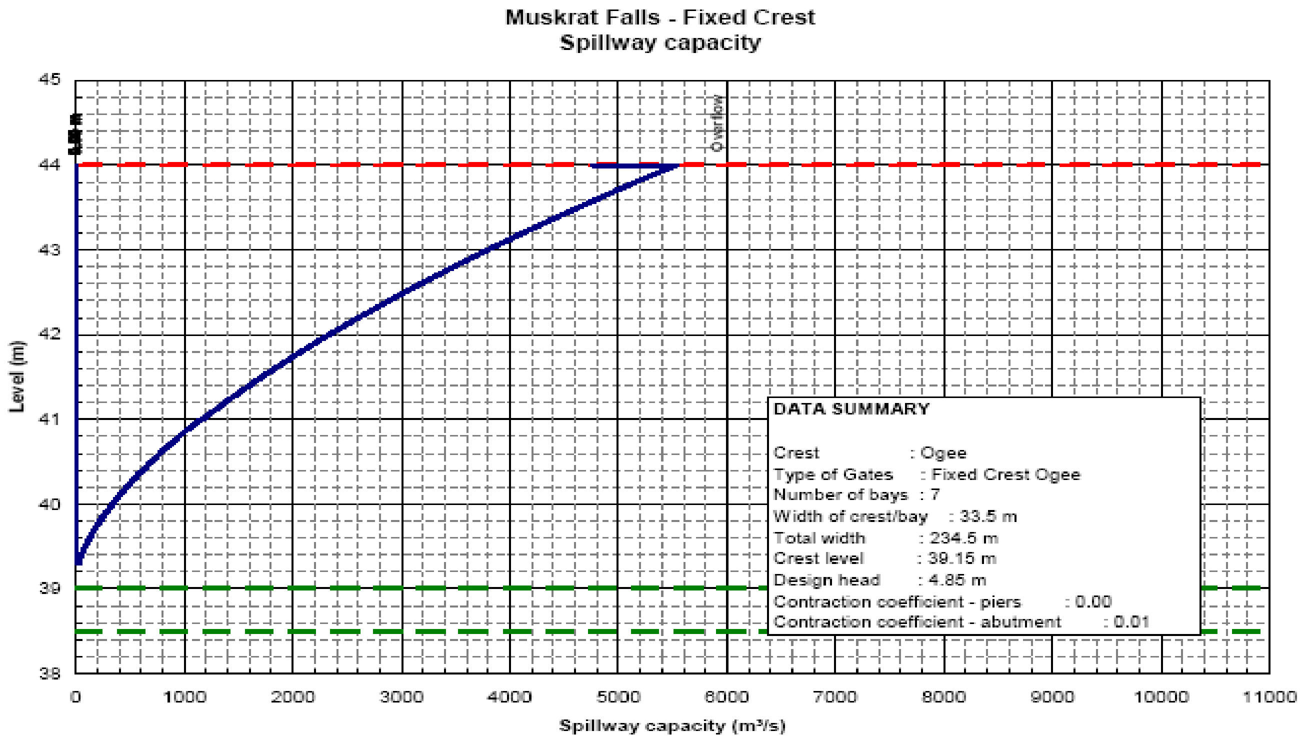


Figure B-4: Variant 10 – Fixed Crest Spillway Rating Curve

Maximum Flood Level (MFL) shown as dashed red line. Full Storage Level (FSL) and Low Storage Level (LSL) shown as dashed green lines.

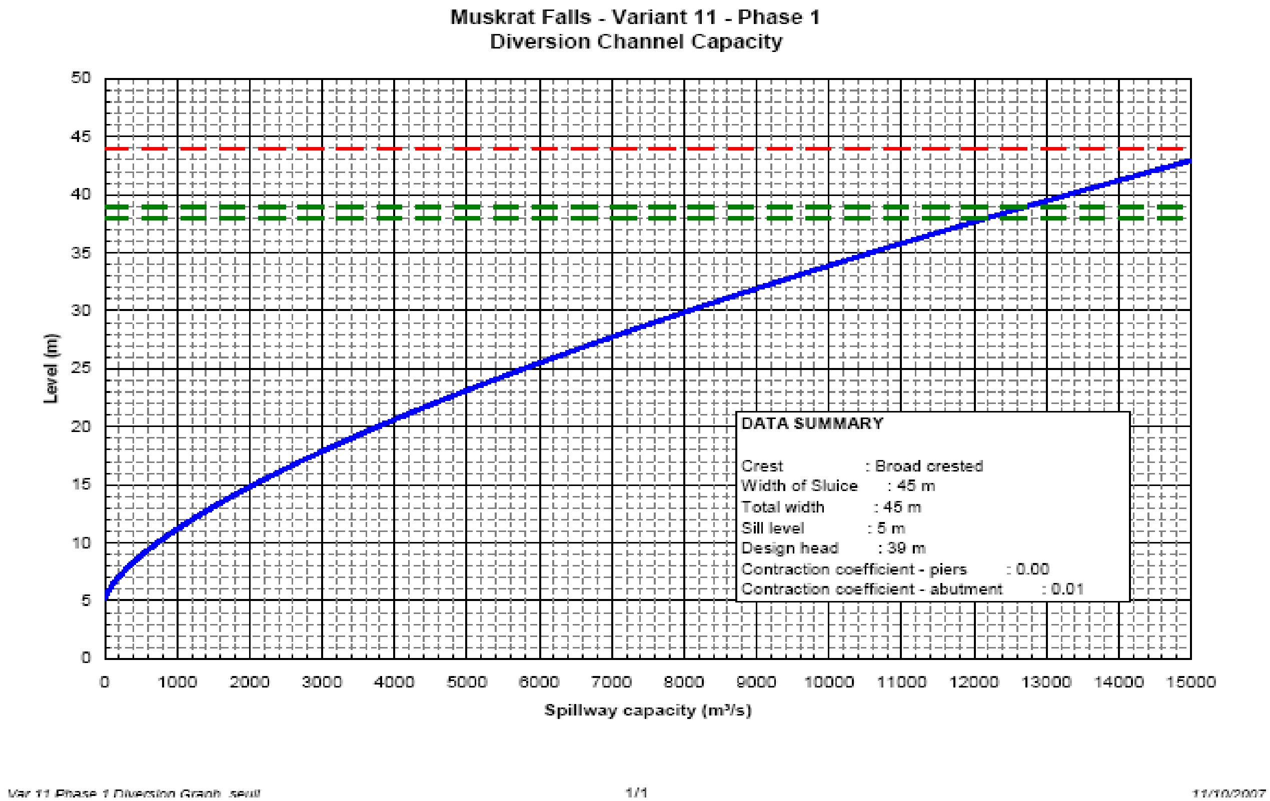


Figure B-5: Variant 11 – Diversion Channel Rating Curve

APPENDIX C

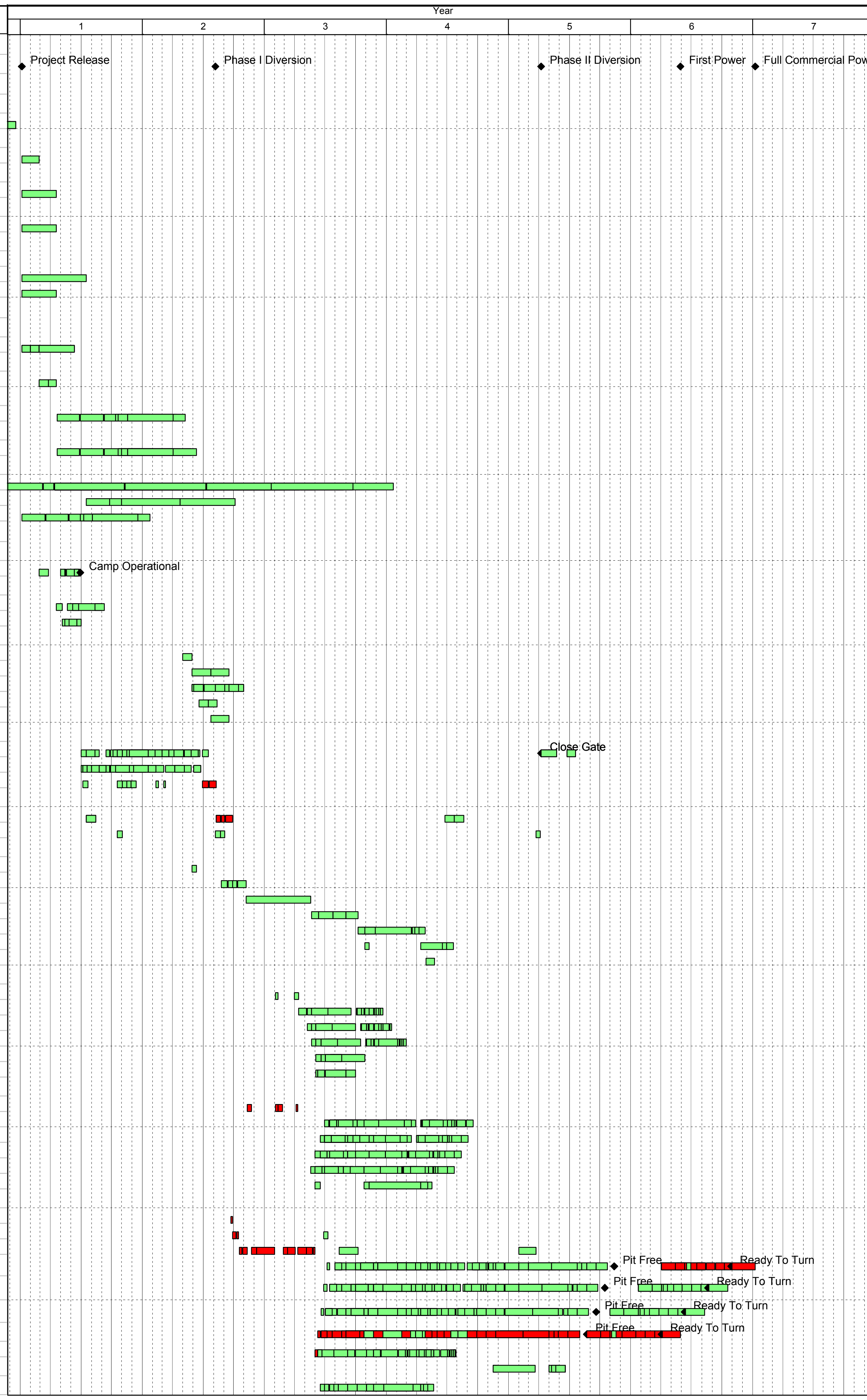
IMPLEMENTATION SCHEDULES

22-Nov-07 10:16

Muskrat Falls Hydroelectric Project

Variant 7 - Summary Schedule

Activity ID	Activity Name
General	
Milestones	
Engineering	
Site Facilities	
	Construction Camp
Site Access	
	Temporary Site Roads
Spillway	
	Gates & Stoplogs
Intake	
	Traskracks, Bulkhead Gates & Service Gates
Powerhouse	
	Turbine Generator
	Overhead Crane
	Draft Tube Gates
Procurement	
Site Facilities	
	Construction Camp
Site Access	
	Temporary Site Roads
Spillway	
	Gates & Stoplogs
Intake	
	Traskracks, Bulkhead Gates & Service Gates
Powerhouse	
	Turbine Generator
	Overhead Crane
	Draft Tube Gates
Construction	
Site Facilities	
	Construction Camp
Site Access	
	Permanent Site Roads
	Temporary Site Roads
North Spur	
	Clearing
	Overburden Excavation
	Fill
	Relief Wells
	Pump Wells
Diversion Facilities	
	Upstream Diversion Tunnel Portal
	Diversion Tunnel
	Downstream Diversion Tunnel Portal
Cofferdams	
	Upstream Cofferdam
	Downstream Cofferdam
Dam	
	Clearing
	Overburden Excavation
	Foundations
	RCC
	Concrete
	Rubber Dam
	Miscellaneous
Spillway	
	Rock Excavation
	Left Abutment
	Pier 1
	Pier 2
	Right Abutment
	Gravity Section
Intake	
	Rock Excavation
	Unit 4
	Unit 3
	Unit 2
	Unit 1
	Gravity Section
Powerhouse	
	Clearing
	Overburden Excavation
	Rock Excavation
	Unit 4
	Unit 3
	Unit 2
	Unit 1
	Erection Bay
Substation	
South Dam	



Remaining Work
Critical Remaining Work
Milestone

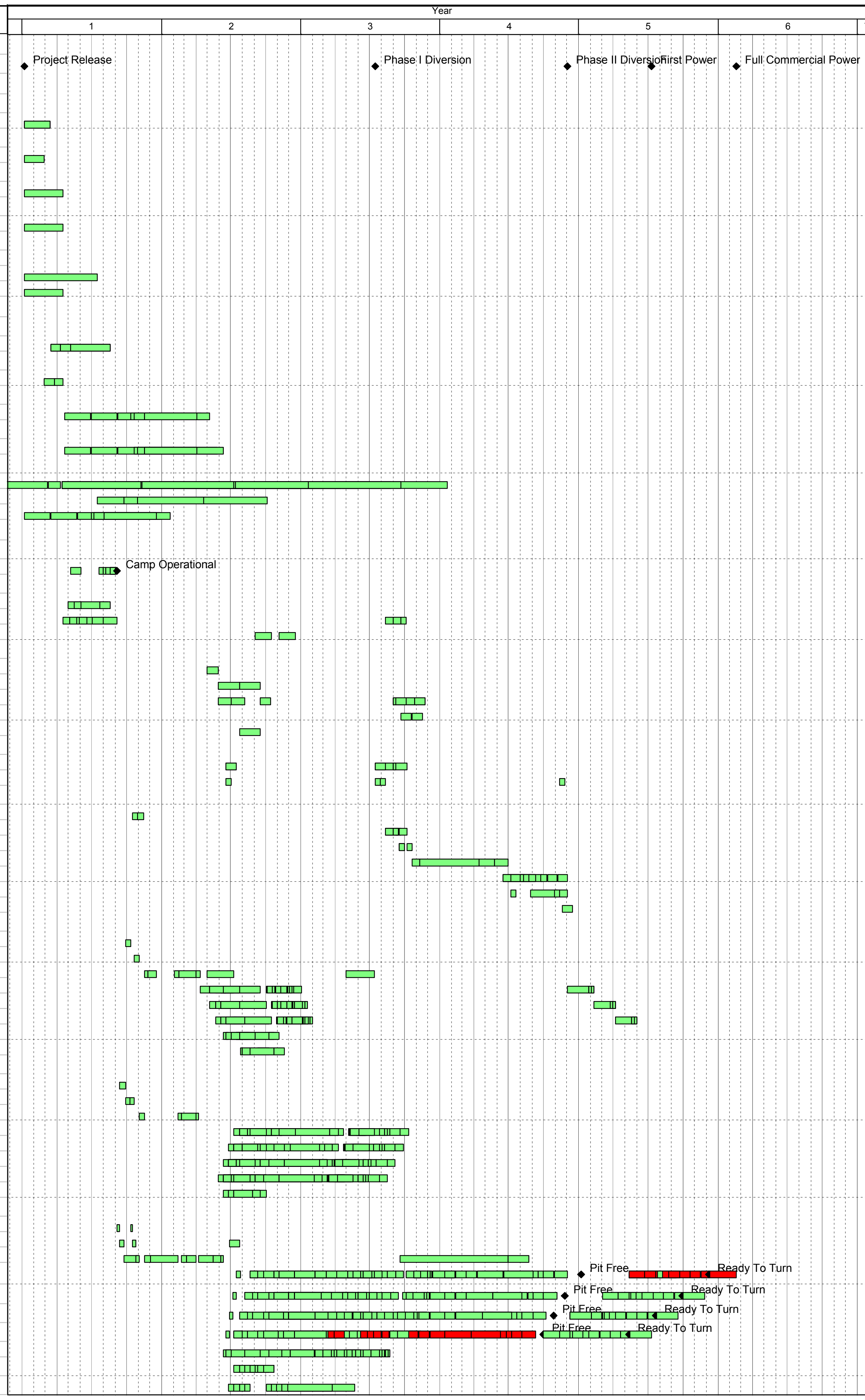
Figure C1

22-Nov-07 10:14

Muskrat Falls Hydroelectric Project

Variant 10 - Summary Schedule

Activity ID	Activity Name
General	
Milestones	
Engineering	
Site Facilities	
	Construction Camp
Site Access	
	Temporary Site Roads
Spillway	
	Gates & Stoplogs
Intake	
	Traskracks, Bulkhead Gates & Service Gates
Powerhouse	
	Turbine Generator
	Overhead Crane
	Draft Tube Gates
Procurement	
Site Facilities	
	Construction Camp
Site Access	
	Temporary Site Roads
Spillway	
	Gates & Stoplogs
Intake	
	Traskracks, Bulkhead Gates & Service Gates
Powerhouse	
	Turbine Generator
	Overhead Crane
	Draft Tube Gates
Construction	
Site Facilities	
	Construction Camp
Site Access	
	Permanent Site Roads
	Temporary Site Roads
	Temporary Bridges
North Spur	
	Clearing
	Overburden Excavation
	Fill
	Relief Wells
	Pump Wells
Cofferdams	
	Upstream Cofferdam
	Downstream Cofferdam
Dam	
	Clearing
	Overburden Excavation
	Foundations
	RCC
	Concrete
	Rubber Dam
	Miscellaneous
Spillway	
	Clearing
	Overburden Excavation
	Rock Excavation
	Left Abutment
	Pier 1
	Pier 2
	Right Abutment
	Gravity Section
Intake	
	Clearing
	Overburden Excavation
	Rock Excavation
	Unit 4
	Unit 3
	Unit 2
	Unit 1
	Gravity Section
Powerhouse	
	Clearing
	Overburden Excavation
	Rock Excavation
	Unit 4
	Unit 3
	Unit 2
	Unit 1
	Erection Bay
Substation	
South Dam	



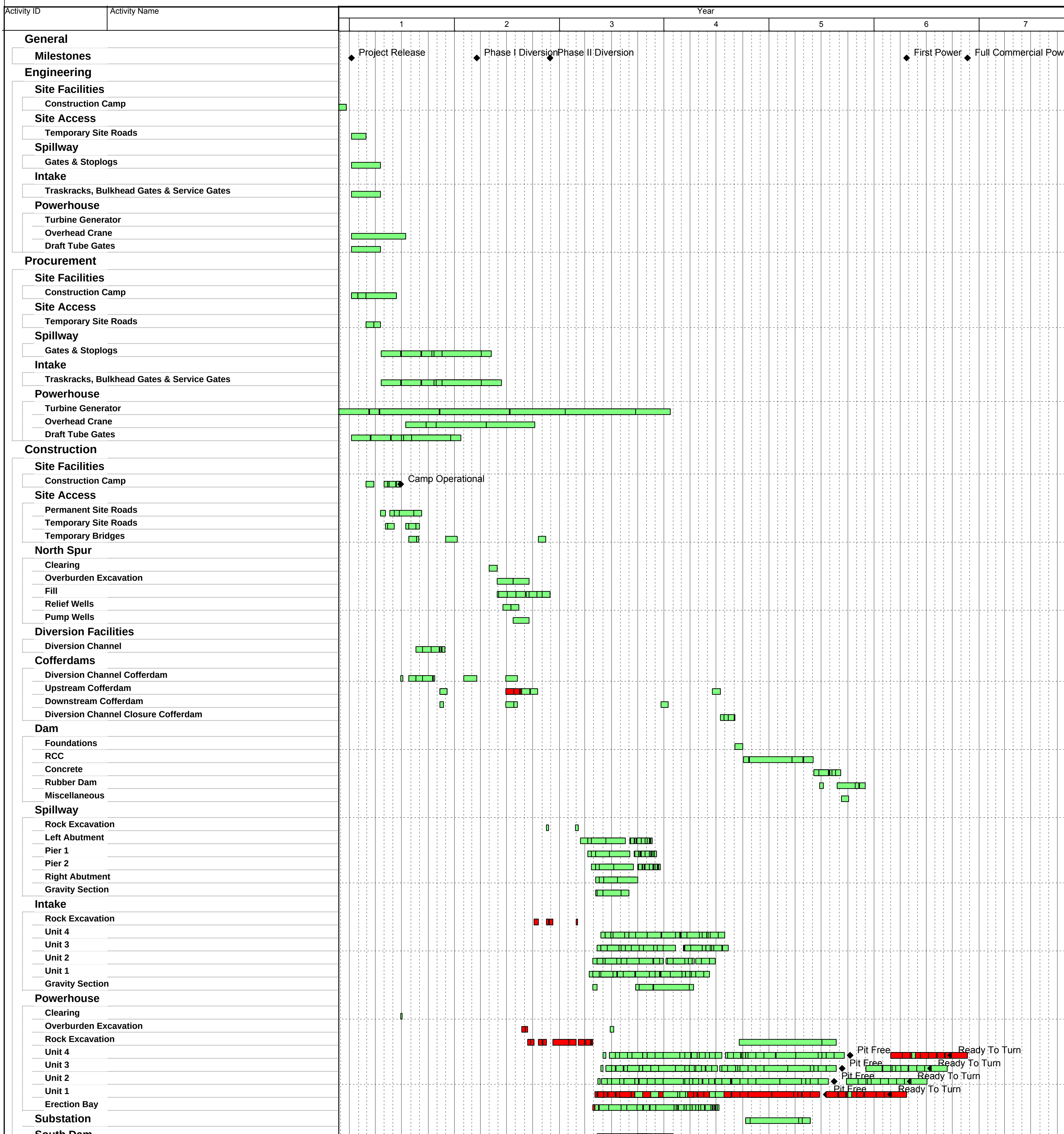
Remaining Work ◆ Milestone
Critical Remaining Work

Figure C2

22-Nov-07 10:12

Muskrat Falls Hydroelectric Project

Variant 11 - Summary Schedule



Remaining Work ◆ ◆ Milestone Critical Remaining Work	Page 1 of 1	**Figure C3**

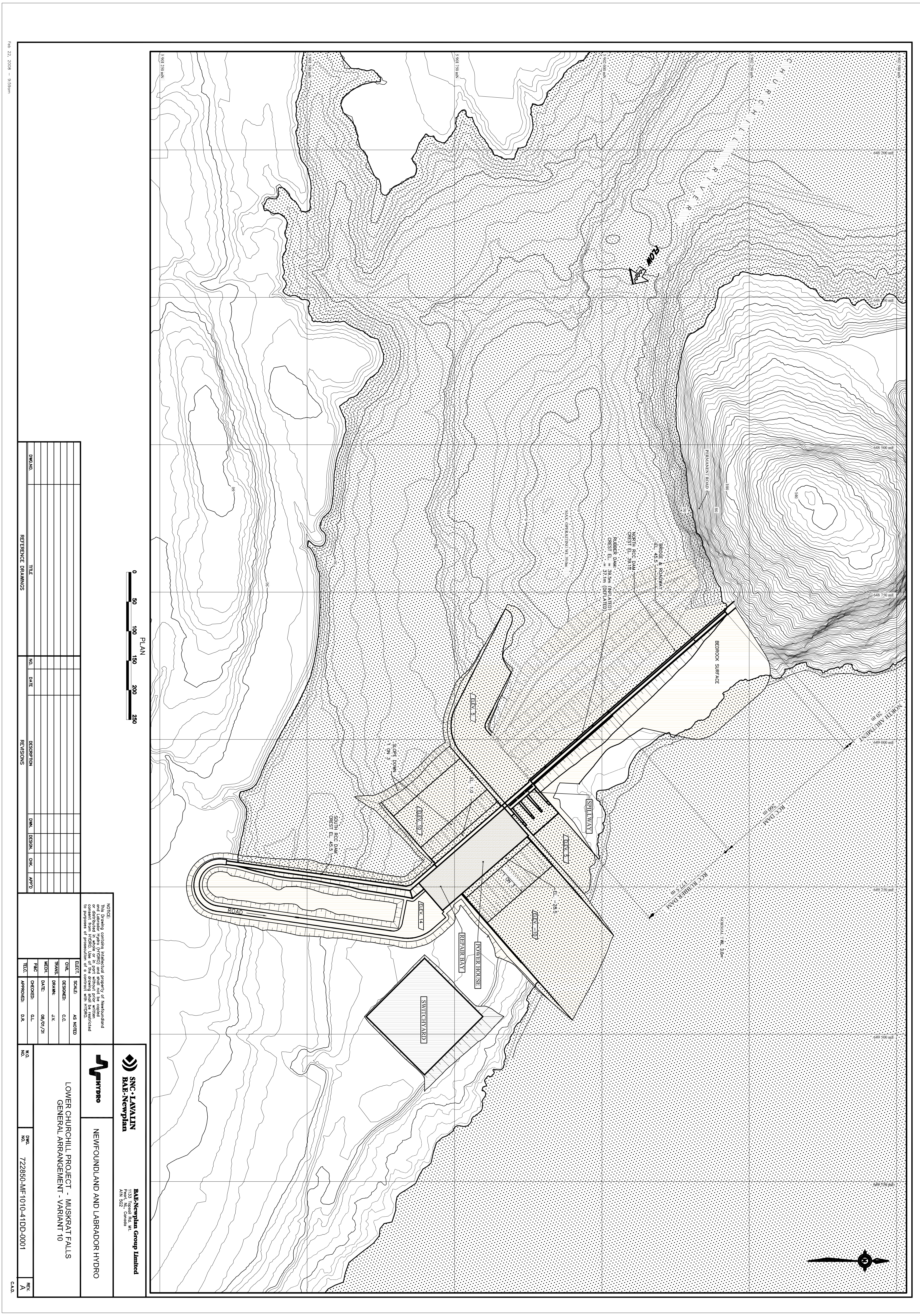
APPENDIX D

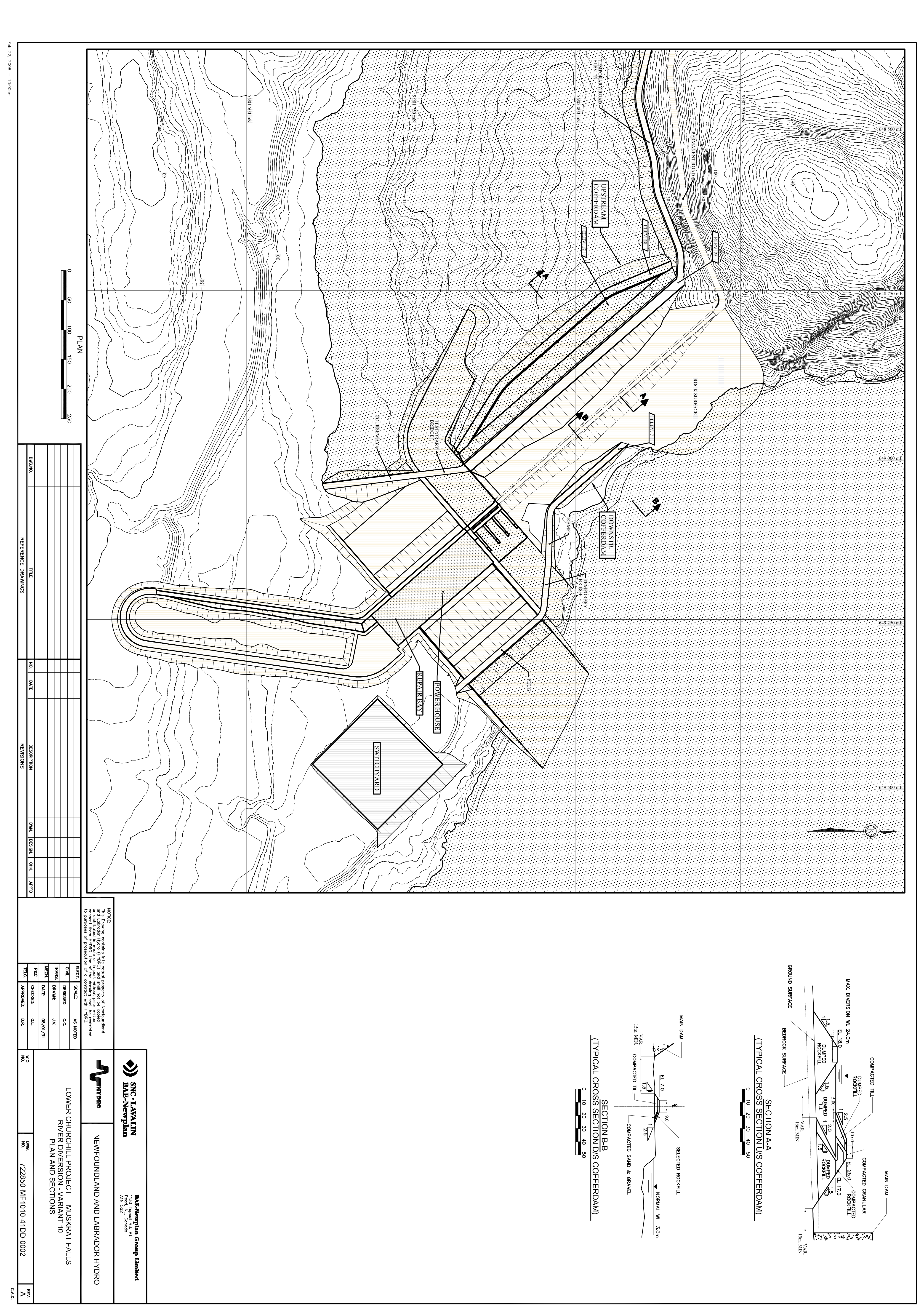
COST ESTIMATES

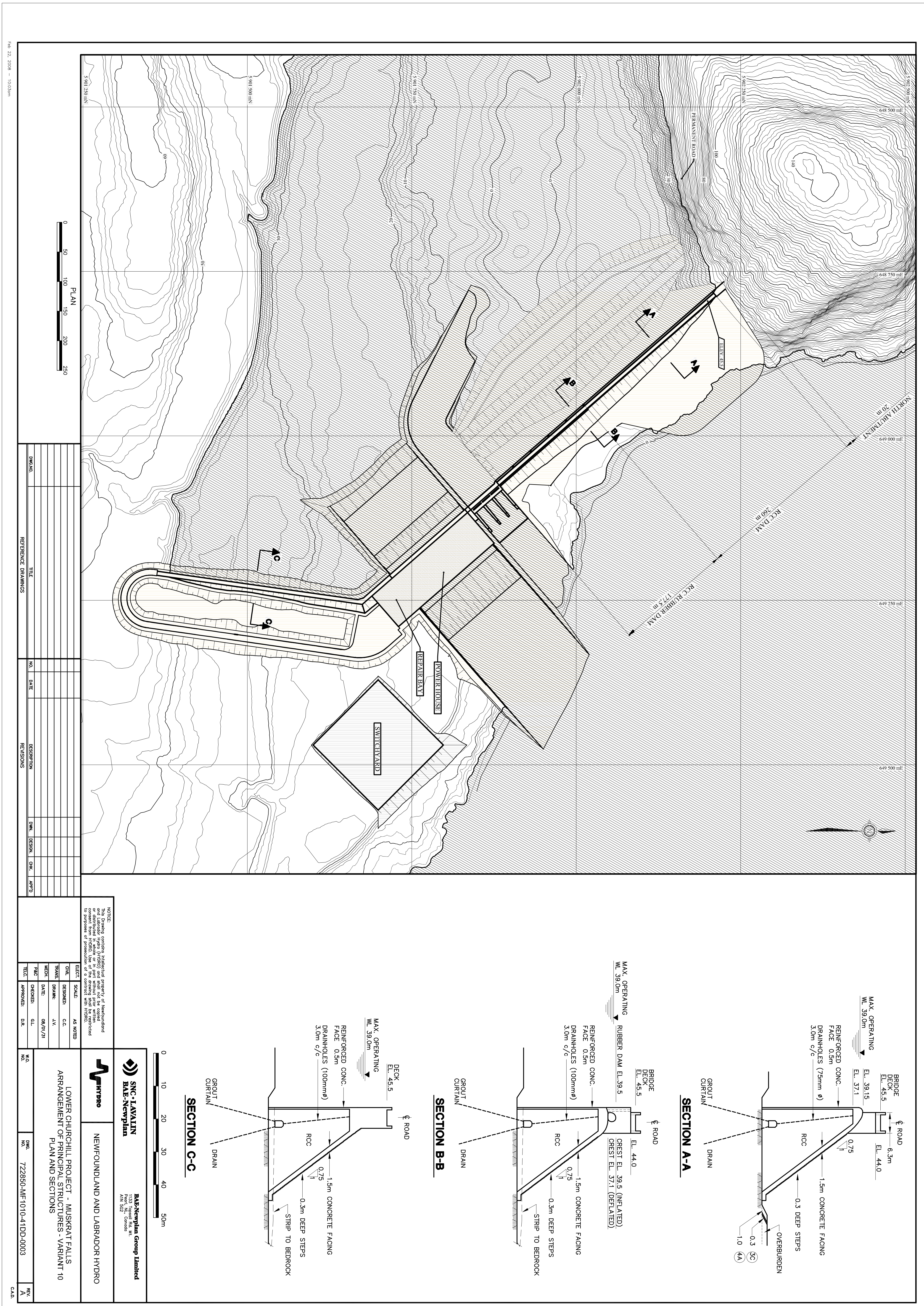
Cost estimates not included
in Public version

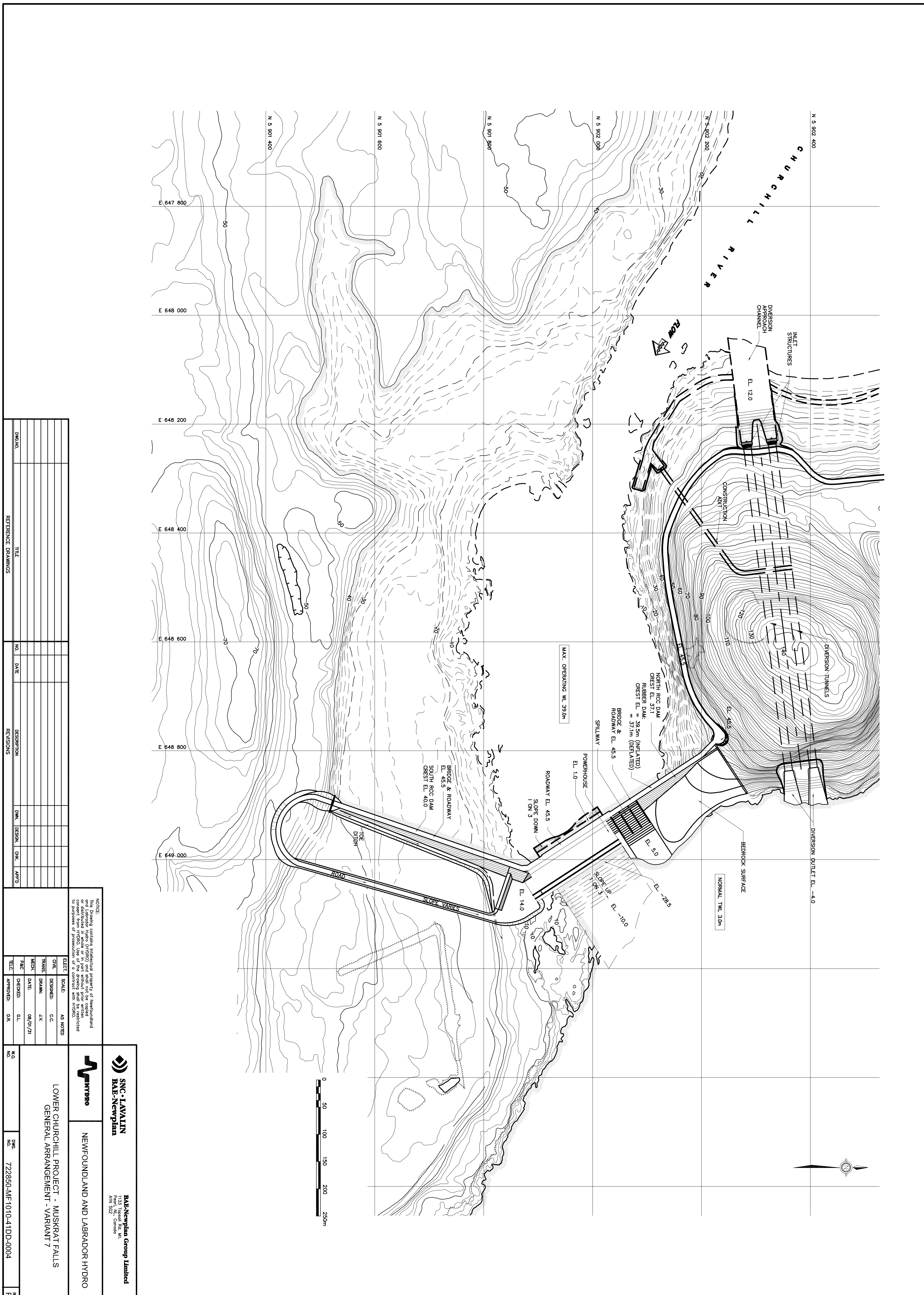
APPENDIX E

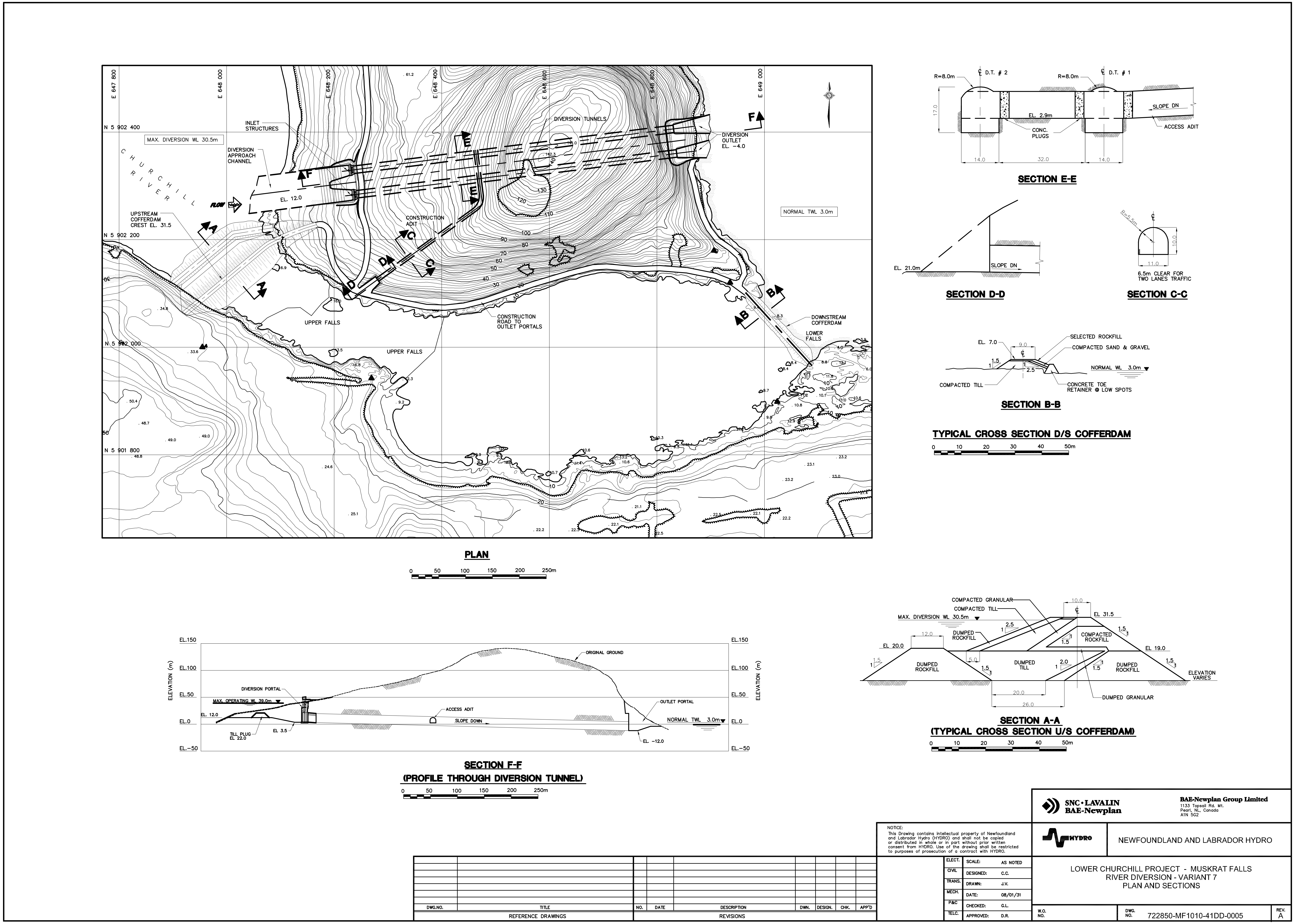
DRAWINGS

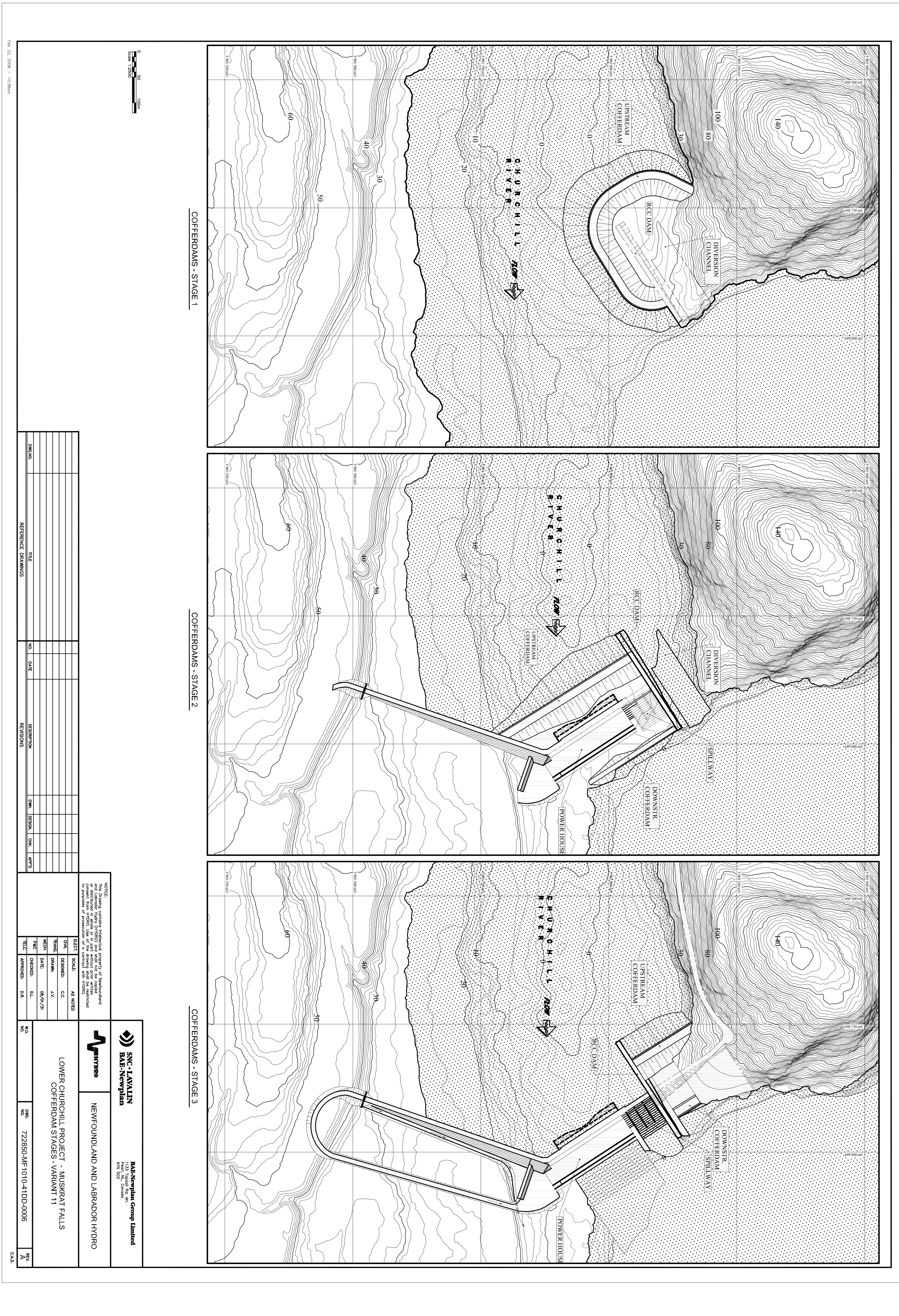

PLAN
NORTH RCC DAM
BEDROCK SURFACE
SPILLWAY
POWER HOUSE
REPAIR BAY
SWITCHYARD
ROAD
SOUTH RCC DAM
PERMANENT ROAD
CHURCHILL RIVER
FLOW
REFERENCE DRAWINGS
REVISIONS
SNC-LAVALIN
BAE-Newplan
BAE-Newplan Group Limited
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
GENERAL ARRANGEMENT - VARIANT 10
722850-MF1010-41DD-0001
REV. A


PLAN
UPSTREAM COFFERDAM
DOWNSTR. COFFERDAM
TEMPORARY ROAD
PERMANENT ROAD
ROCK SURFACE
TEMPORARY BRIDGE
CAUSEWAY
RAMP
POWERHOUSE
REPAIR BAY
SWITCHYARD
SECTION A-A
(TYPICAL CROSS SECTION U/S COFFERDAM)
MAX. DIVERSION WL 24.0m
GROUND SURFACE
BEDROCK SURFACE
COMPACTED TILL
DUMPED ROCKFILL
DUMPED TILL
COMPACTED GRANULAR
COMPACTED ROCKFILL
MAIN DAM
EL 18.0
EL 25.0
EL 17.0
SECTION B-B
(TYPICAL CROSS SECTION D/S COFFERDAM)
EL 7.0
SELECTED ROCKFILL
COMPACTED SAND & GRAVEL
NORMAL WL 3.0m
NOTICE:
This Drawing contains intellectual property of Newfoundland and Labrador Hydro (HYDRO) and shall not be copied or distributed in whole or in part without prior written consent from HYDRO. Use of the drawing shall be restricted to purposes of prosecution of a contract with HYDRO.
REFERENCE DRAWINGS
DWG.NO.
TITLE
REVISIONS
NO.
DATE
DESCRIPTION
DWN.
DESIGN.
CHK.
APP'D
ELECT.
CIVIL
TRANS.
MECH.
P&C
TELC.
SCALE: AS NOTED
DESIGNED: C.C.
DRAWN: J.V.
DATE: 08/01/31
CHECKED: G.L.
APPROVED: D.R.
SNC·LAVALIN
BAE-Newplan
BAE-Newplan Group Limited
1133 Topsail Rd. Mt. Pearl, NL, Canada
A1N 5G2
HYDRO
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
RIVER DIVERSION - VARIANT 10
PLAN AND SECTIONS
W.O. NO.
DWG. NO.
722850-MF1010-41DD-0002
REV. A
C.A.D.


PLAN
RCC DAM
260 m
RCC RUBBER DAM
177.5 m
NORTH ABUTMENT
20 m
POWER HOUSE
REPAIR BAY
SWITCHYARD
PERMANENT ROAD
SECTION A-A
SECTION B-B
SECTION C-C
MAX. OPERATING
WL 39.0m
BRIDGE
DECK
EL. 45.5
EL. 39.15
EL. 37.1
EL. 44.0
6.3m
ROAD
REINFORCED CONC.
FACE 0.5m
DRAINHOLES (75mm ø)
3.0m c/c
DRAINHOLES (100mmø)
GROUT
CURTAIN
RCC
DRAIN
1.5m CONCRETE FACING
0.3 DEEP STEPS
0.3m DEEP STEPS
OVERBURDEN
STRIP TO BEDROCK
RUBBER DAM EL. 39.5
CREST EL. 39.5 (INFLATED)
CREST EL. 37.1 (DEFLATED)
DECK
EL. 45.5
0.75
REFERENCE DRAWINGS
REVISIONS
SNC-LAVALIN
BAE-Newplan
BAE-Newplan Group Limited
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
ARRANGEMENT OF PRINCIPAL STRUCTURES - VARIANT 10
PLAN AND SECTIONS
722850-MF1010-41DD-0003
REV. A
SCALE: AS NOTED
DESIGNED: C.C.
DRAWN: J.V.
DATE: 08/01/31
CHECKED: G.L.
APPROVED: D.R.


CHURCHILL RIVER
FLOW
N 5 901 400
N 5 901 600
N 5 901 800
N 5 902 000
N 5 902 200
N 5 902 400
E 647 800
E 648 000
E 648 200
E 648 400
E 648 600
E 648 800
E 649 000
DIVERSION APPROACH CHANNEL
INLET STRUCTURES
EL. 12.0
CONSTRUCTION ADIT
DIVERSION TUNNELS
DIVERSION OUTLET EL. -4.0
BEDROCK SURFACE
NORMAL TWL 3.0m
MAX. OPERATING WL 39.0m
NORTH RCC DAM CREST EL. 37.1
RUBBER DAM: CREST EL. = 39.5m (INFLATED) = 37.1m (DEFLATED)
BRIDGE & ROADWAY EL. 45.5
SPILLWAY
POWERHOUSE EL. 1.0
ROADWAY EL. 45.5
SLOPE DOWN 1 ON 3
BRIDGE & ROADWAY EL. 45.5
SOUTH RCC DAM CREST EL. 40.0
TOE DITCH
ROAD
SLOPE VARIES
EL. 14.0
EL. 5.0
EL. -28.5
EL. -10.0
SLOPE UP 1 ON 3
EL. 45.5
0 50 100 150 200 250m
REFERENCE DRAWINGS
DWG.NO. TITLE
REVISIONS
NO. DATE DESCRIPTION DWN. DESIGN. CHK. APP'D
NOTICE:
This Drawing contains intellectual property of Newfoundland and Labrador Hydro (HYDRO) and shall not be copied or distributed in whole or in part without prior written consent from HYDRO. Use of this drawing shall be restricted to purposes of prosecution of a contract with HYDRO.
ELECT. SCALE: AS NOTED
CIVIL DESIGNED: C.C.
TRANS. DRAWN: J.V.
MECH. DATE: 08/01/31
P&C CHECKED: G.L.
TELEC. APPROVED: D.R.
SNC·LAVALIN BAE-Newplan
HYDRO
BAE-Newplan Group Limited
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
GENERAL ARRANGEMENT - VARIANT 7
W.O. NO.
DWG. NO. 722850-MF1010-41DD-0004
REV. PA
C.A.D.


PLAN
0 50 100 150 200 250m
MAX. DIVERSION WL 30.5m
INLET STRUCTURES
DIVERSION APPROACH CHANNEL
CHURCHILL RIVER
FLOW
EL. 12.0
UPSTREAM COFFERDAM CREST EL. 31.5
CONSTRUCTION ADIT
DIVERSION TUNNELS
DIVERSION OUTLET EL. -4.0
NORMAL TWL 3.0m
UPPER FALLS
CONSTRUCTION ROAD TO OUTLET PORTALS
DOWNSTREAM COFFERDAM
LOWER FALLS
E 647 800
E 648 000
E 648 200
E 648 400
E 648 600
E 648 800
E 649 000
N 5 902 400
N 5 902 200
N 5 902 000
N 5 901 800
SECTION E-E
R=8.0m
D.T. # 2
D.T. # 1
EL. 2.9m
CONC. PLUGS
SLOPE DN
ACCESS ADIT
17.0
14.0
32.0
SECTION D-D
EL. 21.0m
SECTION C-C
R=5.5m
10.0
11.0
6.5m CLEAR FOR TWO LANES TRAFFIC
SECTION B-B
EL. 7.0
9.0
SELECTED ROCKFILL
COMPACTED SAND & GRAVEL
NORMAL WL 3.0m
COMPACTED TILL
CONCRETE TOE RETAINER @ LOW SPOTS
TYPICAL CROSS SECTION D/S COFFERDAM
0 10 20 30 40 50m
SECTION A-A
(TYPICAL CROSS SECTION U/S COFFERDAM)
0 10 20 30 40 50m
COMPACTED GRANULAR
COMPACTED TILL
MAX. DIVERSION WL 30.5m
EL 31.5
EL 20.0
EL 19.0
DUMPED ROCKFILL
DUMPED TILL
COMPACTED ROCKFILL
DUMPED GRANULAR
ELEVATION VARIES
12.0
5.0
20.0
26.0
SECTION F-F
(PROFILE THROUGH DIVERSION TUNNEL)
0 50 100 150 200 250m
ELEVATION (m)
EL.150
EL.100
EL.50
EL.0
EL.-50
ORIGINAL GROUND
DIVERSION PORTAL
MAX. OPERATING WL 39.0m
EL. 12.0
TILL PLUG EL 22.0
EL 3.5
ACCESS ADIT
SLOPE DOWN
OUTLET PORTAL
NORMAL TWL 3.0m
EL. -12.0
NOTICE:
This Drawing contains intellectual property of Newfoundland and Labrador Hydro (HYDRO) and shall not be copied or distributed in whole or in part without prior written consent from HYDRO. Use of the drawing shall be restricted to purposes of prosecution of a contract with HYDRO.
SNC·LAVALIN BAE-Newplan
BAE-Newplan Group Limited
1133 Topsail Rd. Mt. Pearl, NL, Canada A1N 5G2
HYDRO
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
RIVER DIVERSION - VARIANT 7
PLAN AND SECTIONS
SCALE: AS NOTED
DESIGNED: C.C.
DRAWN: J.V.
DATE: 08/01/31
CHECKED: G.L.
APPROVED: D.R.
ELECT.
CIVIL
TRANS.
MECH.
P&C
TELC.
DWG.NO.
TITLE
REFERENCE DRAWINGS
NO.
DATE
DESCRIPTION
DWN.
DESIGN.
CHK.
APP'D
REVISIONS
W.O. NO.
DWG. NO. 722850-MF1010-41DD-0005
REV. A
C.A.D.
Feb 22, 2008 - 10:38am


COFFERDAMS - STAGE 1
UPSTREAM COFFERDAM
RCC DAM
DIVERSION CHANNEL
CHURCHILL RIVER
FLOW
COFFERDAMS - STAGE 2
SPILLWAY
DOWNSTR. COFFERDAM
POWER HOUSE
COFFERDAMS - STAGE 3
Scale 1:2500
REFERENCE DRAWINGS
REVISIONS
SNC-LAVALIN
BAE-Newplan
BAE-Newplan Group Limited
NEWFOUNDLAND AND LABRADOR HYDRO
LOWER CHURCHILL PROJECT - MUSKRAT FALLS
COFFERDAM STAGES - VARIANT 11
722850-MF1010-41DD-0006
REV. A

APPENDIX F

RUBBER DAM DATA

Construction, operation, and maintenance of rubber dams

X.Q. Zhang, P.W.M. Tam, and W. Zheng

Abstract: Rubber dams are inflatable and deflatable hydraulic structures. Thousands of rubber dams have been installed worldwide for various purposes: irrigation, water supply, power generation, tidal barrier, flood control, environmental improvement, and recreation. Furthermore, rubber dams have been used in cold areas where the temperature is as low as –40°C. The simplicity and flexibility of the rubber dam structure and its proven reliability are key considerations in its wide scope of applications. Based on the management practices of 20 rubber dams in Hong Kong in the past 35 years, interviews with rubber dam experts and practitioners, and the investigation to the construction of a recent rubber dam, this paper provides a detailed discussion on various issues related to the construction, operation, maintenance, and repair of rubber dams.

Key words: construction, hydraulic structure, maintenance, operation, repair, rubber dam.

Résumé : Les barrages en caoutchouc sont des structures hydrauliques gonflables et dégonflables. Des milliers de barrages en caoutchouc ont été installés à travers le monde avec différents buts : irrigation, approvisionnement en eau, production d'électricité, coupe-marée, contrôle des inondations, améliorations environementales et loisirs. De plus, les barrages en caoutchouc ont été utilisés dans des régions froides où la température est aussi basse que –40°C. La simplicité et la flexibilité du barrage en caoutchouc et sa fiabilité prouvée sont des considérations clés dans son vaste champ d'utilisation. Cet article donne une discussion détaillée sur différentes questions reliées à la construction, l'opération, le maintient et la réparation des barrages en caoutchouc, celle-ci basée sur les pratiques de gestion de 20 barrages en caoutchouc à Hong Kong durant les 35 dernières années, sur des entretiens avec des experts et des praticiens en barrage en caoutchouc, ainsi que sur l'investigation de la construction d'un récent barrage.

Mots clés : construction, structure hydraulique, maintient, opération, réparation, barrage en caoutchouc.

[Traduit par la Rédaction]

Introduction

Rubber dams were developed in the 1950s and the first one was installed on the Los Angeles River, California, for the purpose of groundwater recharge and flood mitigation. This early dam was inflated with a combination of water and air (Plaut et al. 1998; Kahl and Ruell 1989). Since then, rubber dams have experienced continuous technological improvement. Simplicity of the rubber dam concept, flexibility, and proven reliability are key considerations in the use of rubber dams for multiple purposes. To date, thousands of rubber dams have been installed in different parts of the world, serving various functions such as irrigation, water supply, power generation, tidal barrier, water treatment, flood control, environment improvement, and recreation.

Received 18 June 2001. Revision accepted 21 February 2002. Published on the NRC Research Press Web site at http://cjce.nrc.ca on 30 May 2002.

X.Q. Zhang.[1,2] Department of Civil Engineering, The University of Hong Kong, Hong Kong, China.
P.W.M. Tam. Parsons Brinckerhoff (Asia) Ltd., Hong Kong, China.
W. Zheng. Guangdong Institute of Water Conservancy and Hydropower, Guangzhou, China.

Written discussion of this article is welcomed and will be received by the Editor until 31 October 2002.

[1]Corresponding author (e-mail: eagle_xqzhang@hotmail.com).
[2]Present address: 2861 Revelstoke Court, Vancouver, BC V6T 1N8, Canada.

As an innovative hydraulic structure, the rubber dam project mainly consists of four parts: (*i*) a rubberized fabric dam body; (*ii*) a concrete foundation; (*iii*) a control room housing mechanical and electrical equipment, such as air blower/water pump, automatic inflation and deflation mechanisms; and (*iv*) an inlet/outlet piping system. The dam body is fixed onto the concrete foundation and abutments by a single-line or double-line anchoring system.

A unique characteristic of the rubber dam is its ability to function as a reliable crest-adjustable water gate. When inflated by a medium (air, water, or combination of both), it rises to retain water. When deflated by releasing the medium, it collapses flatly down to the foundation, completely opening the channel for free flow of water. The rubber dam can also be adjusted to operate at intermediate heights to meet the needs for different upstream and downstream water levels at different time. Air is used more often than water as the filling medium for the following reasons (Bridgestone 1994):

1. Water and water-borne debris can corrode and clog pipes.
2. The design and construction of air-filled dams are simpler.
3. Water-filled dams require a more complex piping system and often need a pond to store water for filling the dams when river water levels are low.

 DOI: 10.1139/L02-016

4. The inflation or deflation time of an air-filled dam is much shorter than that of a water-filled dam of the same size.
5. Because of the weight of water, a water-filled dam has a squat shape, requiring more rubber material than an air-filled dam of the same height.
6. The circumference of a water-filled dam is about 4.8 times of its height, compared with 3.5 times for an air-filled dam. To accommodate the dam body, the foundation of a water-filled dam must be wider than that of an air-filled dam of the same height.

However, air-filled dams are less stable and suffer more from vibration than water-filled dams, which are preferable when hydraulic conditions are more demanding (Markus et al. 1995).

Research methodology

In this paper, a thorough review of the limited literature on rubber dams is carried out. This is followed by extensive investigations on the construction, operation, maintenance, and repair of the 20 rubber dams that have been installed in Hong Kong. At the same time, interviews/correspondences with experts and practitioners in the management of rubber dams (especially those from the Agriculture and Fisheries Department, Drainage Service Department, and Water Supply Department of Hong Kong, China) and professionals from rubber dam manufacturers are conducted. In addition, a number of field trips to some dam sites and the different phases in the construction of a new rubber dam in Hong Kong have been made.

Rubber dam construction

Selection of dam site

In the selection of a suitable dam site, geological, geomorphic, hydrological, meteorological, and hydraulic factors as well as construction methods should be taken into consideration. Although the body of a rubber dam is light and the load is uniform, the dam site should always be on a solid ground and a concrete foundation should be placed. When a rubber dam is installed on a high sediment-laden river, to avoid silt deposition closely behind the dam, the dam site should not be on a section where the longitudinal gradient abruptly becomes gentle. The dam foundation level should be higher than that of the downstream riverbed to prevent silt and gravel from getting directly under the dam body and increasing abrasion. Usually, the dam foundation is connected to the downstream riverbed by a slope of 1:4–1:2 (vertical:horizontal).

The dam site should be in a straight river section where the river flow is smooth and the riverbed and bank slopes are stable. The site should not be in a section where the hydraulic conditions can change abruptly. Adverse hydraulic conditions are the major cause of vibration of rubber dams. Vibration causes abrasion and tear of the dam body and can severely damage it. As a result of inappropriate dam site selection, some rubber dams have been unable to function properly and have had to be demolished. The Mahuang Weir Rubber Dam in the middle section of the Jiangan River in China was an example of unsuitable dam site selection that has led to final failure of the rubber dam. The dam was installed in the lower part of a curved section in March 1967. The dam body, made from canvas and chloroprene rubber, measured 2.5 m high and 33 m long and used a single line anchoring system. Owing to the effect of the curved section of the river, river flows were deflected upstream of the dam. The deflected flows caused strong vibration of the dam body. In 1968, the dam body was inspected and repaired. More than 20 abraded areas were found on the dam body, with some of those areas measuring about 400 cm^2 and the canvas exposed. Because flows were deflected to the left bank, abrasions on the left dam body near the bending left bank were the most serious. Many inspections and repairs were carried out in 1970, 1972, and 1973. Finally, the dam had to be demolished because of vibration damages.

Construction of civil works

The construction of river works is generally carried out in the dry season, as conditions prevailing during the wet season are not favorable for such work. To facilitate the excavation of earthworks and the construction of the dam foundation, river diversion is made. The sequence and program of works should be carefully planned. An ideal measure is to use existing channel to divert river water into adjacent lakes or ponds. If such ideal conditions do not exist, a temporary cofferdam is built to divert water downstream of the dam site. River diversion can also be realized by placing earthfill upstream of the dam site and installing a water pipe under the earthfill. River water bypasses the dam site through the water pipe into the downstream section. In addition, in a broad river, the construction of the dam foundation can begin from one side of the river by diverting water to the other side. Upon completion of the dam foundation on one side, part of the dam body can be placed on the completed dam foundation. Then, river water is diverted into this side so that construction of the dam foundation on the other side can begin. Figure 1 shows the construction site of a new rubber dam in Hong Kong, where river water flows over the dam body placed on the completed dam foundation on one side of the river (while construction of the dam foundation on the other side is in progress).

Before the placement of the dam foundation, loose material should be removed so that it is placed on sound founding material. Foundation settlement affects the crest level of the rubber dam, which is an important parameter affecting the proper functioning of the dam. In addition, reinforced concrete is used to avoid possible differential settlement that could cause tearing of the dam body. A loading test or other testing methods should be used to check the bearing force of the dam foundation. A smooth foundation surface is desirable for reducing abrasion of the dam body when it vibrates and rubs itself against the foundation. The anchoring bolts should be accurately positioned to ensure level placement of the dam body. Otherwise, improperly formed bolts may have a disastrous impact on the whole rubber dam project.

A strong and durable concrete bedplate and apron are installed to prevent wash-cut damage to the riverbed and banks by water flows. In addition, the accumulation of sediment on top of, or adjacent to, the dam will not only adversely affect

Fig. 1. Water flowing over fixed rubber dam body.

the inflation of the dam, but may also damage it. Thus, desilting traps are essential to the proper functioning of the dam. To facilitate regular desilting, vehicular access is usually provided from the riverbank to the bottom of the desilting trap.

Dam body installation

The rubber dam body is a membrane of rubberized fabric, which is composed of layers of synthetic rubber and layers of synthetic fiber reinforcement that are firmly bonded together by vulcanization. When a rubber bladder is manufactured, quality control is maintained throughout the processes of rubberizing, putting together and gluing. Unevenness or local imperfections can lead to bladder breakage. With a large bladder, truck or container transportation may pose restrictions on packing dimensions. Therefore, the adhesion of the bladder has to be carried out on site. Special attention should be given to quality management, since conditions (e.g., dust and humidity) at the dam site may adversely affect the adhesion process.

Before the transportation from the factory to the dam site, the rubber bladder is wound into roll and carefully packaged to avoid any deformation and damage. The bladder and its related parts should be protected against corrosion, mechanical damage, and deterioration during transportation and on-site storage before installation.

The installation of the rubber bladder is simple and can be finished within a short period. Easy and quick installation is one of the advantages of rubber dams compared with other hydraulic structures, such as steel gates, concrete dam, and earth and stone dams. For an experienced crew, the installation of the rubber bladder can be completed in one day. Table 1 shows the installation sequence and time duration of Dam No. TKL 12, Hong Kong, China. The installation crew comprised four members (a supervising engineer from the manufacturer, an interpreter, and two labourers).

Construction costs

Compared with steel gates, the rubber dam becomes more cost-effective with the increase in the length of its span(s). Steel gates are expensive and require intermediate structures (such as concrete piers) that collect debris during floods. Although flashboards are inexpensive to construct, they are unreliable and expensive from an operation and maintenance point of view. The rubber dam is almost maintenance free compared with a steel gate, which requires periodic maintenance such as repairs to the lifting mechanisms, sandblasting, and painting. In the case of raising the crest of an existing dam, the rubber dam is much cheaper to install than any of the various types of steel gates; in fact, given the design of an existing structure, many types of steel gates may be non-viable alternatives. Furthermore, the increased water storage capacity for additional power generation and water supply can result in rapid payback of the construction cost of the rubber dam.

Operation mechanisms

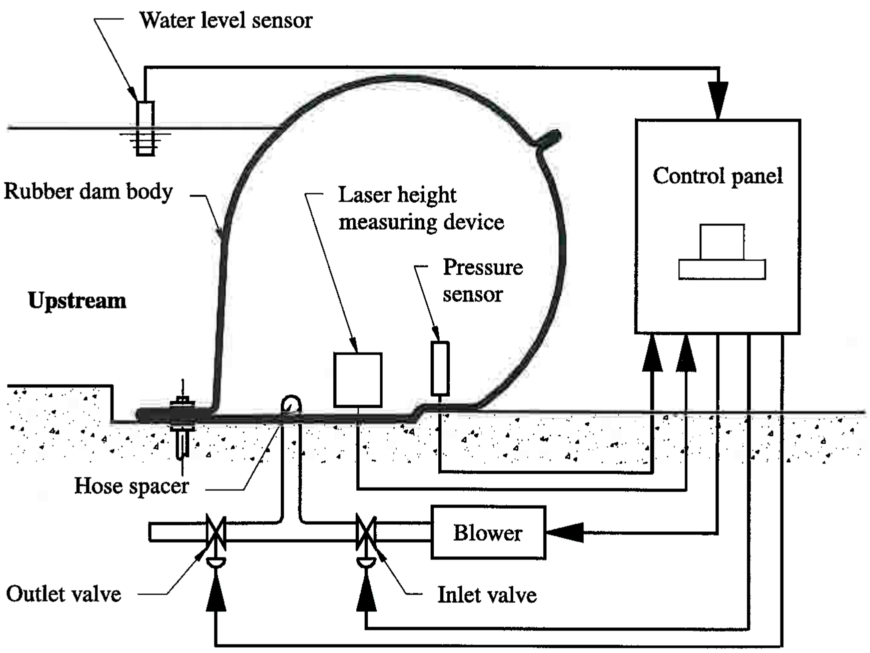
Inflation, deflation, and safety systems

Inflation and deflation devices are installed in the control room. An air blower or water pump and ancillary devices such as valves are used to inflate the air- or water-filled dam respectively. There are three types of deflation systems: bucket, float, and electrical. For the bucket type, water flows into the bucket causing it to fall when the upstream water rises to the deflation level. As it falls, the exhaust valve is opened, deflating the dam body. For the electrical type, a sensor monitors water level. The exhaust valve is opened automatically when the upstream water level reaches the deflation level. For the float type, a float fluctuates up and down with the water level. The outlet valve is opened automatically when the deflation level is reached.

Table 1. Installation sequence and time duration of rubber dam No. TKL 12.

Date and time	Duration (h)	Sequence	Work
20 January 1997, 09:00–12:30	3.5	1	Cleaning of dam foundation
		2	Checking for possible mismatch of the anchor bolts and anchor plates
		3	Application of sealant along the line of anchor bolts
20 January 1997, l4:00–17:00	3	4	Placing of rubber dam body over dam foundation
		5	Drilling of holes on dam body for anchorage (bottom layer; river bed)
		6	Marking the exact position of the pipe opening (bottom layer; river bed)
		7	Drilling of holes on dam body for anchorage (bottom layer; river bank)
		8	Making the exact position of the pipe opening (bottom layer; river bank)
21 January 1997, 14:00–17:00	3	9	Installation of pipe flanges for the air inlet and release system
		10	Drilling of holes on dam body for anchorage (top layer; river bed)
22 January 1997, 10:00–12:00	2	11	Drilling of holes on dam body for anchorage (bottom layer; river bank)
		12	Trimming of dam body
22 January 1997, 13:30–16:00	2.5	13	Clamping of dam body

Fig. 2. Automatic control mechanism.



An air blow-off tank can be used as a safety device for the air-filled dam. It ensures that air is released from the dam body when the principal deflation mechanism malfunctions or the inner air pressure exceeds the maximum design pressure. For a water-filled dam, a siphon pipe can be used for safety purposes. It operates when the principal deflation mechanism fails or when inner water pressure rises above the maximum operating pressure.

Monitoring of inner pressure and dam height

A pressure sensor (Fig. 2) is used to monitor the inner pressure of the dam body. The inlet of air/water to the dam body is cut off automatically when inner pressure reaches the preset level. This prevents damage to the dam body due to excessive pressure. The inner pressure of an air-filled dam can be maintained at a constant value even though the dam is installed in a variable exterior environment.

A laser height measuring device (Fig. 2) can be installed inside the dam body to continuously measure its height when inflating or deflating.

Inflation and deflation procedures

Based on Bridgestone (1994) and experiences in Hong Kong, the following procedures are recommended for the inflation or deflation of an air-filled dam. The procedures for inflating and deflating a water-filled dam are similar to those for an air-filled dam.

Inflation

1. Inspect the upstream side, ensuring that neither people nor property will be adversely affected by raising the upstream water level.
2. Remove debris (especially sharp objects) adjacent to the rubber dam.
3. Close the air exhaust valve.
4. Open the air supply valve.
5. Start the air compressor.
6. Monitor the air pressure gauge while inflating the dam. Adjust the air pressure according to the headwater level. If the headwater level is less than 1/2 of the dam height, the air pressure should be approximately 60% of the de-

sign pressure. The air pressure should increase in proportion to the headwater level. Increase the air pressure to the design level after the headwater reaches the design level. Then turn off the air compressor and close the air supply valve.

Deflation

1. Inspect the downstream side, ensuring that neither people nor property will be adversely affected by raising the downstream water level.
2. Make sure the downstream side of the dam foundation is free of sharp objects and other obstructions that may damage the dam body during deflation.
3. Open the drainpipe to drain the condensed water.
4. Open the air exhaust valve to deflate the dam.

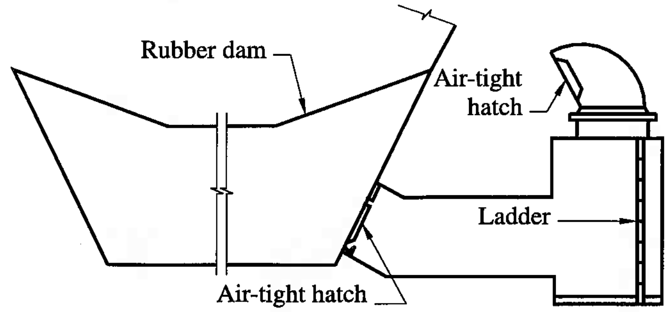
Inflation/deflation time

Inflation time is a function of three main factors: the inner volume of the dam body, the external pressure acting on the dam body, and the capacity of the air compressor or water pump. According to Bridgestone (1994), compressors ranging from 1.9 to 15 kW have been used. A 1.9 kW air compressor that produces air at a rate of 3.0 m^3/s can inflate a 0.8 m high and 10 m long dam in 3 min, while two 15 kW compressors can inflate a 2.6 m high and 60 m long dam in about 65 min. Deflation time is a function of the external pressure, the volume of the dam body, and the number of exhaust pipes. The dam deflates more quickly when there is a head of water acting on the dam body.

Inspection gallery

Some large rubber dams include an inspection gallery (Fig. 3), which permits regular inspection inside the dam body. Through this gallery, inspectors can identify problems earlier and take appropriate preventive and corrective actions. The provision of such a gallery improves the safety and security of the dam. The 1.1 m high and 147.7 m wide rubber dam on Naruse River, Miyagi Prefecture, Japan, is provided with such an inspection gallery. The gallery houses instruments for the observation of (*i*) dam shape, (*ii*) dam vibration, (*iii*) dam tension, (*iv*) dam abrasion, (*v*) river sediment flow, and (*vi*) river flow condition when deflated (Bridgestone 1992).

Fig. 3. Inspection gallery.



Management practices in Hong Kong

Rubber dams were introduced to Hong Kong in the 1960s. To date, a total of 20 rubber dams have been installed in Hong Kong, among which 15 by the Agriculture and Fisheries Department (AFD), 3 by the Water Supplies Department (WSD), and 2 by the Drainage Services Department (DSD). These dams are used for irrigation, pollution abatement, environment improvement, flood mitigation, and water supply. Tam (1998) provides some comparative information of these dams.

Hand-over test, operation manual, and spare parts

In Hong Kong, when the construction of a rubber dam is completed, a hand-over inspection and completion test is conducted before the dam is taken over by the personnel responsible for future daily operation and maintenance. This ensures that construction and installation have been properly carried out and that the dam will function properly. The contractor usually provides an instruction manual, which gives a detailed description of the whole rubber dam system, including the dam body, inflation/deflation procedures, power supply system, and other accessories. Personnel responsible for the operation, maintenance, and repair of the dam are required to use this manual in conducting their duties.

To facilitate maintenance and repair, it is advisable to have the supplier provide a set of repair kits and sufficient key spare parts to ensure satisfactory performance of the dam. A price list including the names of manufacturers of all necessary spare parts and tools for repair should also be provided for future reference. It is recognized that the rubber dam is a patented product. Therefore, the lack of repair tools and spare parts can cause great difficulty in repairing malfunctioning equipment or the dam body itself.

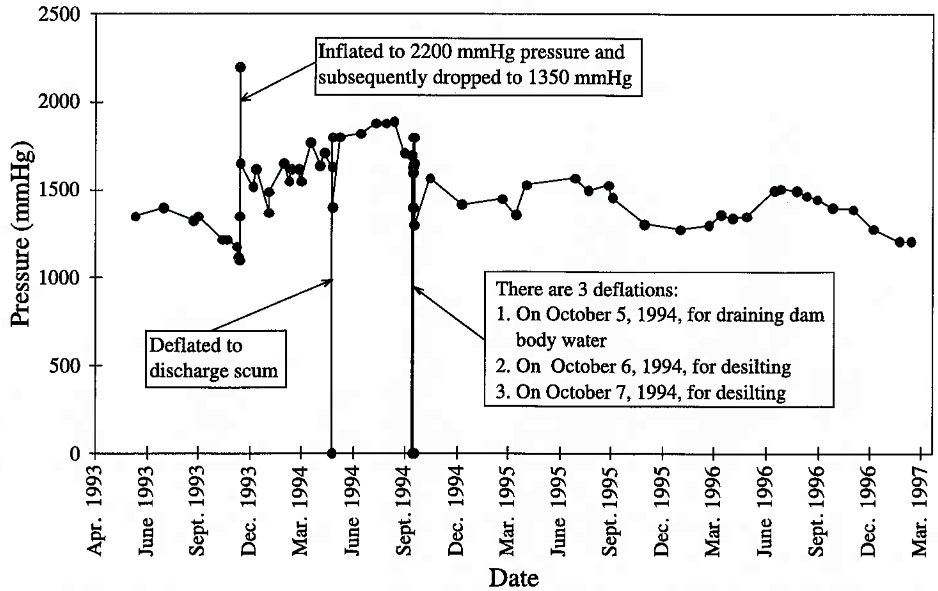
Operation mechanism

Rubber dams in Hong Kong have an electrical inflation/deflation system. Hydrostatic level sensors are erected both upstream and downstream of the dam. The sensors detect the water levels and transmit signals to the control unit, which is a programmable logic controller (PLC). The control unit determines whether the dam should be inflated, deflated, or kept at the present running mode. When the water level reaches a dangerous level, the deflation device is automatically triggered to deflate the dam and thus to mitigate flood.

Signals are also transmitted to the control centers in Ha Tsuen Pumping Station or Yuen Long Sewerage Treatment Plant. Emergency operation teams stationed there will monitor the situation at each rubber dam and take appropriate action as required. In the case of an emergency, such as the PLC not functioning properly, the control center can send signals to the dam control room to regulate its function. In the event that this step also fails, an emergency team will go to the dam site to carry out the necessary operations.

Dams installed by the Drainage Services Department can re-inflate automatically when the upstream water level drops to the designed inflation point. Although the Agriculture and Fisheries Department dams can automatically deflate during heavy rainstorms according to the predetermined upstream water level, they cannot re-inflate automatically. The dams are manually inflated, for example, when the rainstorms are over.

Fig. 4. Pressure record of rubber dam No. YLN 189.



Operation record

To monitor the performance of rubber dams and for future reference as well, important operation data have been recorded and stored in the Ha Tusen Pumping Station or Yuen Long Sewerage Treatment Plant. These data include inner air pressure of the dam body, upstream/downstream water levels, times of inflation and deflation, dates, and reasons for such operations. These data can be used by the operation team to investigate the reasons for a particular inflation or deflation and report to their senior management or to the public, if requested. This information is also valuable for the study of the river's hydrology, including the pattern of water level changes throughout the year. Figure 4 shows the pressure record of Dam No. YLN 189 from June 1993 to March 1997.

The Hong Kong Observatory will announce rainfall and flood warnings when the intensity of rainfall collected in the previous 2-h period exceeds 50 mm/h. In periods of excessive precipitation, river water levels are high. Some dams must be deflated to allow the passage of flood flows, thus avoiding flooding to adjacent areas. The AFD's rubber dam deflation record shows that most deflations occur during the wet season, especially in July and August. The average annual deflation (or inflation) frequency for rubber dams in Hong Kong is about 6 times per dam.

Safety considerations

When the rubber dam is inflated/deflated, water levels change suddenly and may create a hazard to adjacent people and properties. Information on dam inflation and deflation is provided in advance to the people concerned so that they can take proper protection measures. In addition, warning and notification signs are erected along riverbanks adjacent to rubber dams to draw attention to the danger of sudden water level change.

The installation of a rubber dam across a river may sometimes tempt people to use it as a footbridge. This not only may damage the dam but also pose a danger to the trespassers. Notices are posted as reminders not to use the dam to cross the river.

Coordinated operation in the future

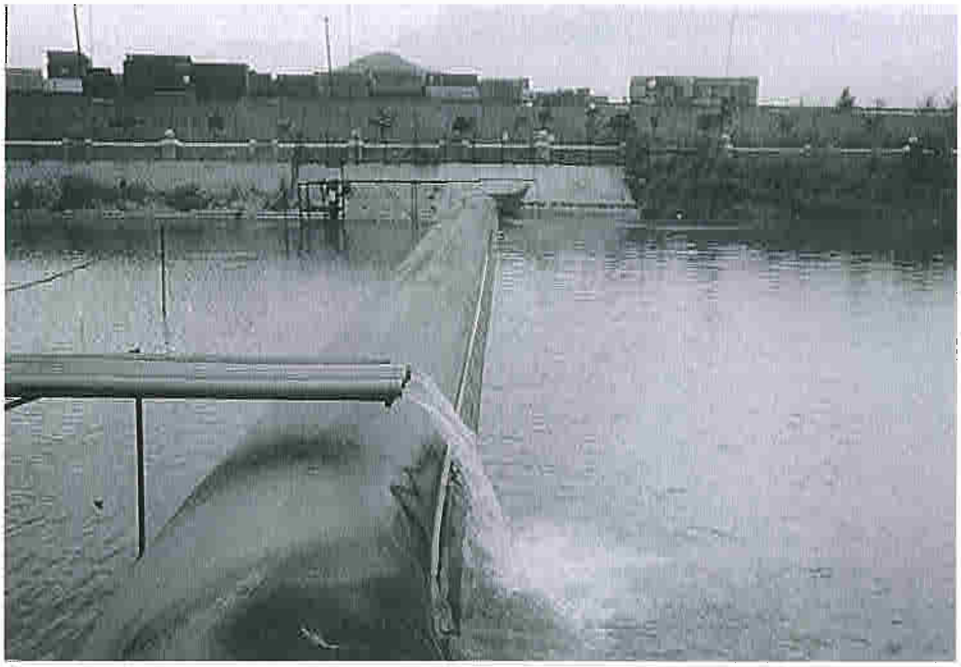
Successful management and public support encourage a new plan to install more rubber dams in river training projects in Hong Kong. For example, there are many potential sites where rubber dams can be installed to relieve flood at locations such as the lower reaches of the Indus River. Furthermore, since they have been proven to be more useful and reliable, rubber dams will gradually replace existing concrete irrigation weirs in the North and Yuen Long Districts. In addition, future dams will be multipurpose, combining flood control, irrigation, water supply, channel maintenance, and environmental improvement to achieve maximum benefits.

At present, rubber dams in Hong Kong are under the separate control of three departments (AFD, DSD, and WSD). In the long term, when more watercourses become engineered, there will be a need for the coordinated operation of rubber dams to meet various demands.

Operation example: Tin Shui Wai rubber dam

Until about 70 years ago, the area that now comprises the Tin Shui Wai new town, New Territories, Hong Kong, was a shallow bay. During development of the new town, extensive land reclamation was carried out, greatly reducing the flood storage capacity in the catchment and increasing the flood risk in low-lying flood-prone areas around the town. Streams through the town tend to be tidal and are prone to severe silting from muddy water in the shallow bay. Furthermore, these streams are heavily polluted by pig wastes and effluent from farms, villages, and towns in the upstream. In addition, the main catchment drainage channel (MCDC) is an important landscape and recreation feature in the town. However, it is broad (about 60 m wide) and could create a harsh feature in an otherwise pleasant environment. For mitigating

Fig. 5. Tin Shui Wai Rubber Dam.



flood and improving the environment, a rubber dam (Fig. 5) with a bypass was installed across the MCDC in 1991.

The dam serves three main functions: (*i*) to retain amenity water in the MCDC at a level of +2.0 m P.D. (principal datum); (*ii*) to allow the smooth passage of storm water without flooding the new town or exacerbating flooding in the low-lying areas; and (*iii*) to replace amenity water by partially deflating the dam when water becomes polluted or its quality deteriorates during prolonged dry spells. This operation ensures that water is maintained at a "passive amenity value" — visually pleasing and without offensive odor.

Operation of the dam is controlled by water level sensors and a preset logic. It deflates when the upstream water level is more than 2.5 m P.D., and higher than the downstream level. The dam also deflates when the upstream water level reaches 3.0 m P.D., irrespective of the downstream level. The dam remains in the deflated mode until the upstream water level drops below 2.0 m P.D., at which point the air blower starts and re-inflates the dam.

There is also a bypass to the dam. The bypass includes three penstocks, two of which can be manually adjusted to vary the level of amenity water, while the third one is normally closed but can be opened to drain the water as an alternative to deflating the dam.

Preventive maintenance

Innovative materials and designs have been applied to rubber dams to increase their life span and enhance their performance. However, the life and performance of a rubber dam project are dependent on many external factors. Tam and Zhang (1999) have discussed various factors that can negatively affect and even damage the rubber dam. It is concluded that rubber dams are most vulnerable to two types of damages, due respectively to (*i*) vandalism and (*ii*) flood-borne debris, especially sharp objects. The objectives of preventive maintenance are to increase the dam's reliability and reduce the time when the dam is inoperable.

Periodic inspection and routine maintenance

Based on the management practice of rubber dams in Hong Kong, an operation and maintenance check list (Fig. 6) is developed for this paper. Inspection and maintenance are carried out periodically. These include mainly (*i*) inspecting the dam body and surrounding environment to identify if there is any presence of potential damages, such as punctures and cuts; (*ii*) removing silt and debris from both upstream and downstream of the dam; and (*iii*) keeping the chamber that houses the level sensors and associated piping systems clean and thus ensuring that the electrical and mechanical control equipment receives true information. Table 2 shows the components to be inspected, contents of inspection, and inspection frequency.

The need for maintenance of rubber dams is minimal. They do not corrode or require painting. Table 3 shows the maintenance actions for Dam No. YLN 189 from June 1993 to September 1995. The average maintenance time was about 3 h in each inspection. The total maintenance time (129 h) was only about 0.6% of the total operation time (approximately 19 992 h). Table 4 provides a comparison of the construction, operation, and maintenance costs of the Yuen Long Inflatable Dam.

Sediment removal

Sediment moving at high velocities in a hyper-sediment-laden river has a scouring effect on the dam body. Since rubber is less susceptible to scour than concrete, this does not pose a major problem to the dam body. In addition, if the sediment problem is endemic to a particular channel, an abrasion-resistant layer can be incorporated onto the outer layer of the dam body. However, sediment accumulation behind a rubber dam may affect the normal operation of the

Fig. 6. Rubber dam operation and maintenance check list.

Operation and Maintenance Check List

Name of rubber dam: ________________________ **Date:** __________a

I. Operation

1. Purpose of Operation: ☐ Scheduled ☐ At request ☐ After rainfall on: ______a
2. Operation Record:
   a. Deflated from _____ A.M./P.M. within _____ minutes
   b. Inflated from _____ A.M./P.M. within _____ minutes

II. Maintenance and repair

1. Inflation/deflation System: ☐ All in Order
(1) Auto-deflation device: ☐ In order ☐ Repaired
(2) Intake for inlet pipe: ☐ In order ☐ Cleansed/Desilted
(3) Inlet/outlet pipe: ☐ In order ☐ Cleansed/Desilted
(4) Auto-exhaust valve: ☐ In order ☐ Restored
(5) Auto-exhaust float: ☐ In order ☐ Repaired ☐ Replaced
(6) Float setting: ☐ Unchanged ☐ Reset to: _____ mm
(7) Wire for float: ☐ In order ☐ Tightened ☐ Replaced

2. Control Room: ☐ All in order
(1) Door: ☐ In order ☐ Repaired ☐ Replaced
(2) Pad lock: ☐ In order ☐ Lubricated ☐ Replaced
(3) Louvre: ☐ In order ☐ Repaired ☐ Replaced
(4) Pressure meter: ☐ In order ☐ Out of order ☐ Repaired/Replaced
(5) Access track/foot path: ☐ In order ☐ Cleansed ☐ Repaired

3. Dam Body Condition: ☐ In order ☐ Repaired (cut wound ___ cm^2/punch hole)
(1) Discharge of dam body water: ______ minutes
(2) Deflation/re-inflation time: _____ minutes
(3) Pressure reading:
   a. Just before blower shutting off: ______ mm Hg
   b. After blower shutting off: ______ mm Hg

4. Dam Cleansing:
(1) Scum discharging: _____ m^3
(2) Silt removal: _____ ton
(3) Vegetation removal: _____ ton
(4) Reduse removal:: _____ ton

Reported by: ___________ **Checked by:** ___________ **Approved by:** ___________

dam. For example, a large amount of sediment buildup on a deflated rubber dam may make it difficult to inflate it to the normal height. Silt and debris can be removed periodically using manual or mechanical tools.

Another way to solve sediment problem is to flush the sediment downstream by deflating the dam. In 1985, Sumitomo used a 5 m high and 10 m wide experiment dam to test the sediment removal capability of rubber dams through flushing. 160 m^3 of silt, sand, and rubble were piled to 3 m high upstream of the dam. Within the deflation time of 60 min, the material piled up was removed as a result of the flushing effect. An inflation test with silt deposited on the deflated rubber dam was also carried out. Test results indicated that if the depth of the deposit does not exceed 20% of the maximum height of the dam, the dam can be raised to its normal height when subject to the normal inflation pressure (Sumitomo 1985). It is acceptable to exceed the normal operating pressure by up to 1.5 times to lift a sediment-covered dam and then reduce the pressure once the sediment has been cleared (Bridgestone 1994). In the case of a large amount of sediment still remain after initial inflation, it is advisable to deflate the dam again after a water head has developed.

Prevention of vandalism

Vandalism is a major concern affecting the application of rubber dams. The most common types of vandalism are bullet holes and knife cuts. Bridgestone had endorsed bullet damage tests. Caliber rifles were fired at a 12.5 mm thick sample piece of their rubber dams from distances of 5–30 m. Most of the bullets passed through the sample. However, the bullet holes were much smaller than the projectiles due to the elasticity of rubber. A rubber dam does not burst like a balloon as a result of gun shots because of its reinforced fabric body. Knife cuts are a big concern because the area damaged in this way is usually larger and more difficult to repair than bullet holes.

Ceramic chips, steel mesh, and Kelvar have been used to armor, harden, and strengthen the dam body. For example, because ceramic chips are harder than steel, a knife cannot

Table 2. Periodic inspection of rubber dams.

Components	Contents of inspection	Frequency
Dam body	Whether there is any signs of damages or fatigue	Yearly
Anchoring system	Whether there is any sign of deformation of bolts and nuts or loosing	Yearly
Rubber dam surrounding areas	Whether there is drifting material	Monthly
	Whether there is accumulation of debris on top of or adjacent to the dam	Bimonthly
Water level sensor	Whether the sensor screen is blocked	Yearly
Silt trap	Whether there is accumulation of silt, rubbish, and other objects	Monthly
Pump storage pond	Whether there is siltation	Monthly
Low flow channel	Whether there is siltation	Monthly
Inlet structure	Whether there is siltation blocking the screen at inlet box	Monthly
	Whether there is siltation in the culvert	Yearly
Outlet structure	Whether the flap gate at outlet box functions properly	Yearly
	Whether there is siltation in the culvert	Yearly
Control room	Whether the electric and mechanic equipment functions properly	Monthly
	Whether there is water leakage	Monthly

cut or break them. Therefore a dam can be effectively protected from knife cuts by coating with a sufficient density of ceramic chips. Furthermore, the bond strength between the outer ceramic chips and the inner dam body layer is greater than the strength of the rubber itself, indicating that rubber tears before the adhesive bonding gives way. However, a coating of ceramic chips is not an effective countermeasure against bullet damage. Self-healing rubber has been used to prevent the loss of air from bullet holes. In addition, since the rate of air escape from bullet holes is slow at low inner pressure, an automatic air supply mechanism can be incorporated into a rubber dam. Upon detecting a drop of inner pressure below a preset trigger level, an air pressure sensor will start the air compressor to pump air into the dam. Once the inner pressure reaches the design operating level, the compressor turns off automatically.

Furthermore, a security fencing system can be used to prevent unauthorized people from entering the dam area, and a closed circuit television (CCTV) can be installed to deter vandals and to visually monitor dam operation as well. For example, 2.5 m high fences were erected in the Yeun Long Nullah Inflatable Dam in Hong Kong. CCTV systems have been provided for all DSD dams and this practice is increasingly used on other rubber dam sites.

Inspection and removal of sharp objects

Another major concern is that sharp objects carried down by fast-moving flow can puncture the upstream side of the dam body. Large moving objects such as rubble and stones also have a damaging impact on the dam body. To reduce the risk of damage by sharp objects during inflation and deflation, removal of silt and debris is carried out regularly on both the upstream and downstream sides of the dam. A resilient cushion can be incorporated onto the inner layer of the dam body to reduce the impacts of moving objects. In addition, trapping troughs can be constructed across the channel to trap sharp objects and debris and thus to keep potential damage to the minimum.

Rubber dams in cold area

Rubber dams are operable in very cold weather, and air-filled dams perform better than water-filled ones in such conditions. For example, two rubber dams (the Rainbow Dam and the Broadwater Dam) on the Missouri River, Montana, are in operation, where the temperature may be as low as –40°C. In a river prone to icing, upstream water surface adjacent to a rubber dam should be kept unfrozen to protect its body from being damaged by the great pressure of a large-area ice block. In case such a large block is formed, it should be broken into small pieces using manual or mechanical tools. Small pieces can pass over the dam to the downstream. The rubber dam absorbs impacts from drift ice by its capacity to undergo deformation during ice passage and regains its shape thereafter. The rubber dam at Highgate Falls, Vermont, is subject to heavy ice passage. In Canada, hinged steel plates are often used to protect the dam from ice damage.

Table 3. Maintenance actions at dam No. YLN 189.

Item	No. of times*
Inspection	43
Topping up of air pressure	5
Removal of drainage water	1
Desilting	2
Discharge of scum	1

*Number of times from June 1993 to September 1995.

Corrective maintenance

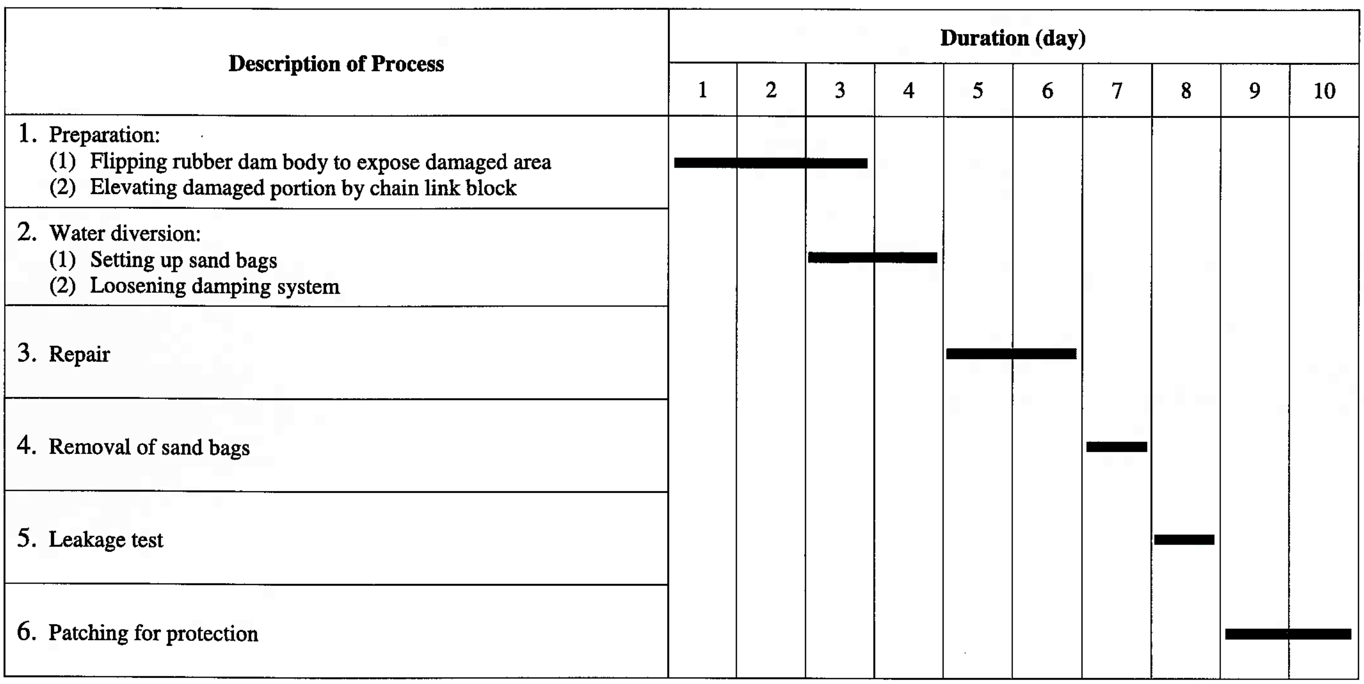
Although various preventive measures have been taken to reduce the incidence and extent of damage to rubber dams, these cannot be completely avoided. Corrective maintenance is performed to restore malfunctioning units to a satisfactory and safe condition within the shortest possible time.

Damage to the dam body can be classified into four types: (*i*) small puncture (e.g., bullet hole), (*ii*) surface damage, (*iii*) small area damage, and (*iv*) large area damage. The repair of small cuts/punctures would normally take half to one day. However, for a large area damage a longer repair period may be needed. For example, 10 days were spent to repair the damage caused to Dam No. YLN 189 in November 1997. Figure 7 shows the repair schedule of this dam. Table 5 provides a record of the AFD concerning puncture repairs of their dams. Repair techniques are discussed in the following sections and are shown in Fig. 8.

Table 4. Construction, operation, and maintenance costs for Yuen Long inflatable dam.

Description	Cost (HK$)	(US$)
Construction cost (not including foundation cost)	13 071 600	1 675 847
Manufacture and supply of rubber dam body	7 614 000	976 154
Fixing equipment	1 500 000	192 308
Pipes	147 600	18 923
Operating equipment	2 100 000	269 231
Installation, test, and commission	1 350 000	173 077
Independent checking of design and works	360 000	46 154
Annual operation and maintenance costs	130 000	16 667
Electrical and mechanical aspects	100 000	12 821
Civil aspects	30 000	3 846

Fig. 7. Repair schedule of dam No. YLN 189.



Small puncture (bullet hole)

The air pressure record can be used to analyze whether or not a dam is leaking. A suspected hole in the dam body can be found in two ways. The submerged portion can be checked for air bubbles coming from the dam body, while the portion above water can be coated with soapy water and inspected for bubbles caused by air leakage.

The simple method to repair automobile tires by inserting plugs can be deployed to repair small punctures (e.g., bullet holes) to a rubber dam. One advantage of this technique is that it does not require the damaged area to be dry. Therefore, dam operation is not interrupted during repair. The following procedures have been used in the repair of small puncture holes:

1. Remove the object that causes the puncture from the dam body. It is better to keep the damaged area clean and dry.
2. Insert a needle covered with cement into the puncture hole several times so that cement would fill in the hole.
3. Insert sealing plug into the puncture hole.
4. Test for air leakage by spreading soapy water over the repaired area. The repair is successful if no bubbles appear.
5. Cut off the end of the sealing plug.

Surface damage

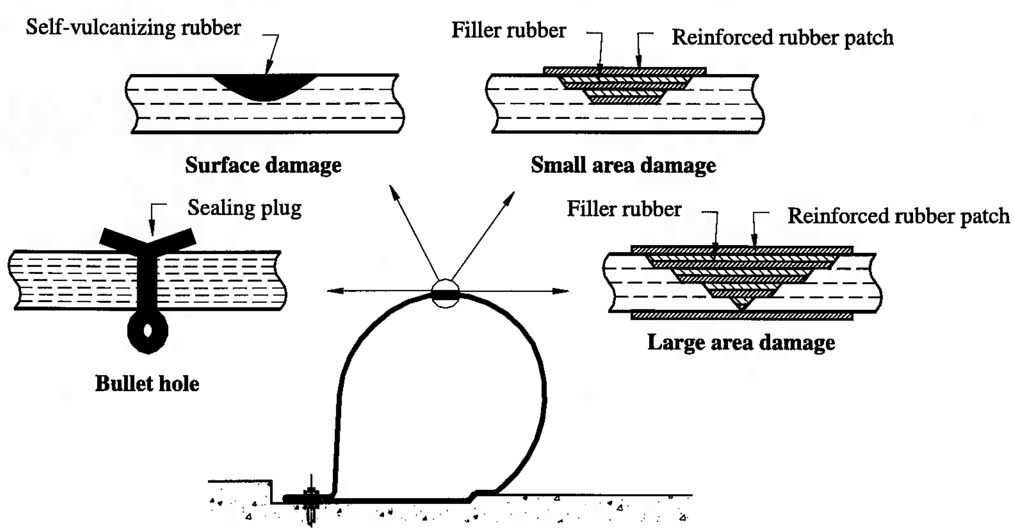
To repair surface damage, the following simple procedures are adopted:

1. Cut off the outer surface of the damaged area.
2. Fill self-vulcanizing rubber into the damaged area after cleaning and drying.
3. Smooth outer surface by removing protruding rubber materials.

Table 5. Puncture repair record (up to November 1997).

Dam No. (year installed)	Number of damages	Dates of repair	Repair duration (day)
BR9 (1991)	4	23 June 1993	0.5
		13 October 1993	0.5
		20 October 1993	0.5
		21 December 1994	0.5
BR11 (1991)	2	12 July 1993	0.5
		20 July 1993	0.5
BR16 (1992)	2	25 March 1996	1
		5 September 1996	1
YLN65 (1992)	1	11 December 1993	2
		12 December 1993	
YLN 189 (1992)	1	November 1997	10
BR8 (1991)	—	—	—
BR10 (1991)	—	—	—
TKL6 (1997)	—	—	—
TKL11 (1997)	—	—	—
TKL12 (1997)	—	—	—
TKL13 (1997)	—	—	—
YLN117 (1992)	—	—	—
YLN160 (1994)	—	—	—
YLN162 (1992)	—	—	—
YLN191 (1992)	—	—	—
	Total: 10		Total: 17

Fig. 8. Repair techniques for different types of damage to the rubber dam body.



Small-area damage

For small-area damage, the following procedures are adopted:

1. Cut off the outer rubber around the damaged area at approximately 45°.
2. Buff the cut surface, then clean and dry it.
3. Apply cement two times to the cut surface, the second coat applied after the first coat has dried.
4. Apply cement to a piece or several pieces of filler rubber and then patch the filler rubber onto the cut surface after the cement has dried.
5. Apply cement to a piece of reinforced fabric and patch it onto the filler rubber after the cement dries.
6. Level the applied piece of reinforced fabric by rolling it with a grinder.
7. Repeat steps 4, 5, and 6 until the last piece of reinforced fabric covers the outer surface of the dam body.
8. Patch the outer surface of the damaged area with a piece of reinforced fabric.
9. Carry out an air leakage test.

Large-area damage

The first 8 steps of the technique for repairing a large-area damage are similar to those for a small-area damage, plus the following two steps:

9. Patch the inner side of the damaged area with a piece of reinforced fabric.
10. Carry out an air leakage test.

Conclusions

Based on literature review, interviews/correspondence with professionals and practitioners, and case studies of the 20 rubber dams in Hong Kong, this paper discusses the various issues concerning the construction, operation, maintenance, and repair of rubber dams.

The rubber dam has been put into a wide scope of application (irrigation, water supply, power generation, flood control, environmental improvement, and recreation) due to its structural simplicity, being inflatable and deflatable, and proven reliability. The air-filled rubber dams are used more often than the water-filled ones for a number of reasons.

In the selection of a suitable dam site, geological, geomorphic, hydrologic, meteorological, and hydraulic conditions should be considered. The dam site should be in a straight section where river flow is smooth and riverbed and bank slopes are stable. It is better to carry out the construction of civil works related to the rubber dam in a dry season. The sequence and program of construction works should be carefully planned and an appropriate water diversion measure should be taken.

Compared with steel gates, the rubber dam becomes more cost-effective with the increase of the length of its span(s). The operation of rubber dams is easy and the operation and maintenance costs are minimal because of their simple structure and innovative inflation and deflation mechanisms. Nevertheless, proper operation procedures should be adopted to ensure safe operation.

Rubber dams are subject to external damages, especially vandalism and punctures by fast-flowing sharp objects. Therefore, both preventive and corrective maintenance measures should be carried out to keep rubber dams in effective operational condition as long as possible. Repair techniques for four types of damages to the dam body have been presented.

References

Bridgestone Corporation. 1992. Naruse River tidal barrier rubber dam. Tokyo, Japan.

Bridgestone Corporation. 1994. Questions and answers about the Bridgestone rubber dam. Tokyo, Japan.

Bridgestone Corporation. 1997. Rubber dam: inflatable rubber weir. Tokyo, Japan.

Hamed, G.R. 1992. Materials and compounds. *In* Engineering with rubber, how to design rubber components. *Edited by* A.N. Gent. Carl Hanser Verlag, Munich, Germany.

Kahl, T., and Ruell, S. 1989. Flashboard alternatives including rubber dams. Proceedings of Waterpower '89, Niagara Falls, New York, N.Y., pp. 447–456.

Lu, W.H. 1988. Rubber dams. China Agricultural Science and Technology Press, Beijing, China (in Chinese).

Markus, M.R., Thompson, C.A., and Ulukaya, M. 1995. Aquifer recharge enhanced with rubber dam installations. Water Engineering and Management, **142**(1): 37–40.

Moorthy, C.M.D., Reddy, J.N., and Plaut, R.H. 1995. Three-dimensional vibrations of inflatable dams. *In* Thin-walled structures. Elsevier Science, Ltd., Amsterdam, The Netherlands. Vol. 21, pp. 291–306.

Plaut, R.H., Liapis, S.I., and Telionis, D.P. 1998. When the levee inflates. Civil Engineering, January, pp. 62–64.

Sumitomo Electric Industries Ltd. 1985. Technical description of Sumigate: inflatable rubber dams. Osaka, Japan.

Sumitomo Electric Industries Ltd. 1990. Inflated ideas. New Civil Engineering, September, pp. 34–38.

Sumitomo Electric Europe SA. 1991–1992. The inflatable rubber dam: 30 years on. Kensington Publication Limited, London, U.K.

Sumitomo Electric Industries Ltd. 1997. Pioneering rubber gate technology: Sumigate. Osaka, Japan.

Tam, P.W.M. 1998. Use of inflatable dams as agricultural weirs in Hong Kong. Journal of Hydraulic Engineering, **124**(12): 1215–1226.

Tam, P.W.M., and Zhang, X.Q. 1999. Management of rubber dams in Hong Kong. Canadian Journal of Civil Engineering, **26**: 123–134.

Application of inflatable dam technology – problems and countermeasures

Paul W.M. Tam

Abstract: The history of the use of inflatable dams has been a long one in Hong Kong. The first three inflatable dams were constructed in the sixties as an integral part of the Plover Cove Water Supply Scheme. Up to December 1996, a total of 16 inflatable dams had been constructed. The use of rubber as a construction material has been subject to much skepticism. There are many reasons for this and one of them is clearly the problem of durability. Despite the many problems, rubber dams have been successfully implemented in Hong Kong. Most of the rubber dams in Hong Kong had been constructed by the Agriculture and Fisheries Department, Hong Kong Government, for the replacement of polluted agricultural weirs which were still in use. A rubber dam is inflatable and deflatable; when it is inflated, it serves as an agricultural weir (low-level dam) and when it is deflated it functions as a flood mitigation device. This note reviews the problems associated with the application of the technology in Hong Kong. Some countermeasures are suggested.

Key words: inflatable dams, flooding, flood mitigation, rubber, weathering, vandalism.

Résumé : L'utilisation des barrages gonflables a connu une longue histoire à Hong Kong. Les trois premiers barrages gonflables furent construits dans les années soixante comme partie intégrante du schéma d'alimentation en eau de Plover Cove. Jusqu'à décembre 1996, seize barrages gonflables au total furent construits. L'utilisation de caoutchouc comme matériau de construction fut sujet à beaucoup de scepticisme. Il y a plusieurs raisons à ceci, et l'une d'entre elles est clairement un problème de durabilité. Malgré leurs nombreux inconvénients, les barrages en caoutchouc furent implémentés avec succès à Hong Kong. La plupart des barrages en caoutchouc à Hong Kong furent construits par le département de l'agriculture et des pêches du gouvernement de Hong Kong, pour le remplacement des déversoirs agricoles pollués qui étaient encore utilisés. Un barrage en caoutchouc est gonflable et dégonflable; lorsqu'il est gonflé, il sert comme un déversoir agricole (barrage à faible hauteur) et lorsqu'il est dégonflé, il fonctionne comme un moyen d'atténuation de crue. Cet note révise les problèmes associés à l'application de cette technologie à Hong Kong. Quelques contremesures sont suggérées.

Mots clés : barrages gonflables, inondation, atténuation de crue, caoutchouc, dégradation, vandalisme.
[Traduit par la rédaction]

Introduction

Rubber dams are installed to function as weirs or barrages which are relatively low-level dams constructed across a river for the raising of river level for the diversion of flow in full, or in part, into a supply canal or conduit for irrigation, domestic, or industrial use.

The first inflatable dam was developed by an American engineer, Norman Imbertson, Chief Operations Engineer in the mid-1950s for the Los Angeles Department of Water and Power. The product was known as Fabridam and was first marketed by Firestone Tire and Rubber Co. This early dam relies on water and air for inflation (Kahl and Ruell 1989). In 1968, Sumitomo Electric Industries Inc. acquired the inflatable dam technology from Firestone Tire and Rubber Co. and their first overseas installation was in Taiwan in 1977 for irrigation. Their second installation was on Tai Po Tau River, Hong Kong, in 1978. In 1978, Bridgestone Corporation introduced an air-inflated rubber dam in Japan and in the international market in 1982. The rubber dam at Indus River, Hong Kong, was one of their first overseas installations.

Although the use of inflatable dams is not very widespread in Hong Kong, many inflatable dams have been installed in Japan. It has been reported that approximately 2000 inflatable dams have been installed worldwide since the first installation (Moorthy et al. 1995). These dams are mainly used as the dam body for reservoirs for the provision of irrigation water for rice fields. These dams have a number of advantages, including the fact that they could be provided over a much longer span than those provided by steel gates. Also, rubber dams can be laid flat on river bottom without causing any obstruction to river flow.

Dam material

Two types of elastomers are used for the inflatable dams: neoprene and EPDM-armored (ethylene propylene diene monomer) rubber. The former is used by Sumitomo Electric Industries Inc. and the latter is used by Bridgestone Corporation. Some important characteristics of the elastomers are discussed below.

Neoprene (chloroprene or polychloroprene)

The material used in the earliest types of inflatable dams is

Received April 14, 1997.
Revised manuscript accepted August 12, 1997.

P.W.M. Tam. Department of Civil and Structural Engineering, The University of Hong Kong, Pokfulam Road, Hong Kong.

Written discussion of this note is welcomed and will be received by the Editor until August 31, 1998 (address inside front cover).

Table 1. Rubber dams in Hong Kong.

No.	Dam No.	River	Height (m)	Width (m)	Year of first installation	Manufacturer[a]	Dam material
		Agriculture and Fisheries Department					
1	BR8	Beas	1.6	13.04	1991	BC	EPDM-R[b]
2	BR9	Beas	1.4	13	1991	BC	EPDM-R
3	BR10	Beas	1.24	16.76	1991	BC	EPDM-R
4	BR11	Beas	0.70	10.8	1991	BC	EPDM-R
5	BR16	Beas	2.0	32.0	1992	BC	EPDM-R
6	YLN 65	Kam Tin	1.8	12.4	1992	BC	EPDM-R
7	YLN 117	Kam Tin	1.4	22.2	1992	BC	EPDM-R
8	YLN 162	Yuen Long	2.0	7.0	1992	BC	EPDM-R
9	YLN 189	Yuen Long	1.7	9.0	1992	BC	EPDM-R
10	YLN 191	Yuen Long	1.2	12.0	1992	BC	EPDM-R
11	YLN 160	Yuen Long	1.5	11.0	1994	BC	EPDM-R
		Drainage Services Department					
12	TSWID	Tin Sui Wai Drainage Channel	2.2	52.5	1988	BC	EPDM-R
13	YLID	Yuen Long Nullah	3.0	59.0	1990	BC	EPDM-R
		Drainage Servces Department					
14	Tai Po Tau Fabridam	Tai Po	3.7	38.0	1964	FTRC	Neoprene
15	Indus River Fabridam	Indus River	2.7	32.5	1967	FTRC	Neoprene
16	Tau Pass Fabridam	Tau Pass	1.5	19.8	1965	FTRC	Neoprene

Notes: Rubber dam dimensions are based on data from Bridgestone Corporation and Water Supplies Department, Hong Kong Government. The width of a rubber dam corresponds to the width of the river.

[a]BC, Bridgestone Corporation; FTRC, Firestone Tire and Rubber Co.

[b]EPDM-R, ethylene propylene diene monomer armored rubber.

neoprene. Neoprene (polychloroprene) is the generic name for chloroprene polymers (2-chloro-1,3-butadiene) manufactured since 1931 by E.I. du Pont de Nemours & Company (Forman 1973). Neoprene has superior weatherability, heat resistance, and low permeability to air and water vapor.

EPDM-armored rubber

EPDM (ethylene propylene diene monomer) has excellent resistance to weathering and good heat stability (Hamed 1992). They are fully saturated elastomers and they are the ultimate in ozone resistance (Ellul 1992).

Inflatable dams in Hong Kong

Both types of the inflatable dams are in operation in Hong Kong and are supplied by the manufacturers mentioned above. Both of these dams are patented. Although both dam types are strongly resistant to weathering, the rubber dams manufactured by Bridgestone Corporation have a thicker dam body and are considered by the Agriculture and Fisheries Department to be more resistant to abrasion and cut damage. A list of the inflatable dams in Hong Kong is given in Table 1.

Most rubber dams installed in Hong Kong are for irrigation and are usually small; the maximum width of an agricultural dam ever constructed was 32 m and the maximum height was 2 m. The highest rubber dam installed in Hong Kong is 3.7 m high and it is used for water storage. Figure 1 shows the rubber dam locations in Hong Kong and Fig. 2 shows an agricultural rubber dam (BR16) in the inflated and the deflated modes.

Problems associated with the rubber dams

There are a number of problems associated with inflatable dams. The problems discussed below are a collection of the experience from the Water Supplies Department and the Agriculture and Fisheries Department for the inflatable dams.

Vandalism

A dam body is prone to damage by sharp objects. The vulnerability of the dams to vandalism suggests that it is an important consideration if a rubber dam should be constructed at a particular location at all. However, the experience gained from installation of rubber dams in Hong Kong indicates that the risk of vandalism is relatively small. Since the first agricultural rubber dam was in operation in 1992, the incidences of damage due to vandalism were relatively small. In one instance, a dam body was damaged. It was suspected that the dam was pierced by a sharp object by a person who was trying to deflate the dam for catching fish. In another instance, a control house was broken into through the louvre. It was found that the manhole cover inside the house was open but there was no damage to the instrumentation. There were no lightweight, valuable, detachable components in the control house.

Fencing of the dam site is an effective means for the avoidance of vandalism and is adopted by the Water Supplies Department. An alternative measure is the provision of a 24-hour close circuit television monitoring system. The provision of this system itself deters vandalism.

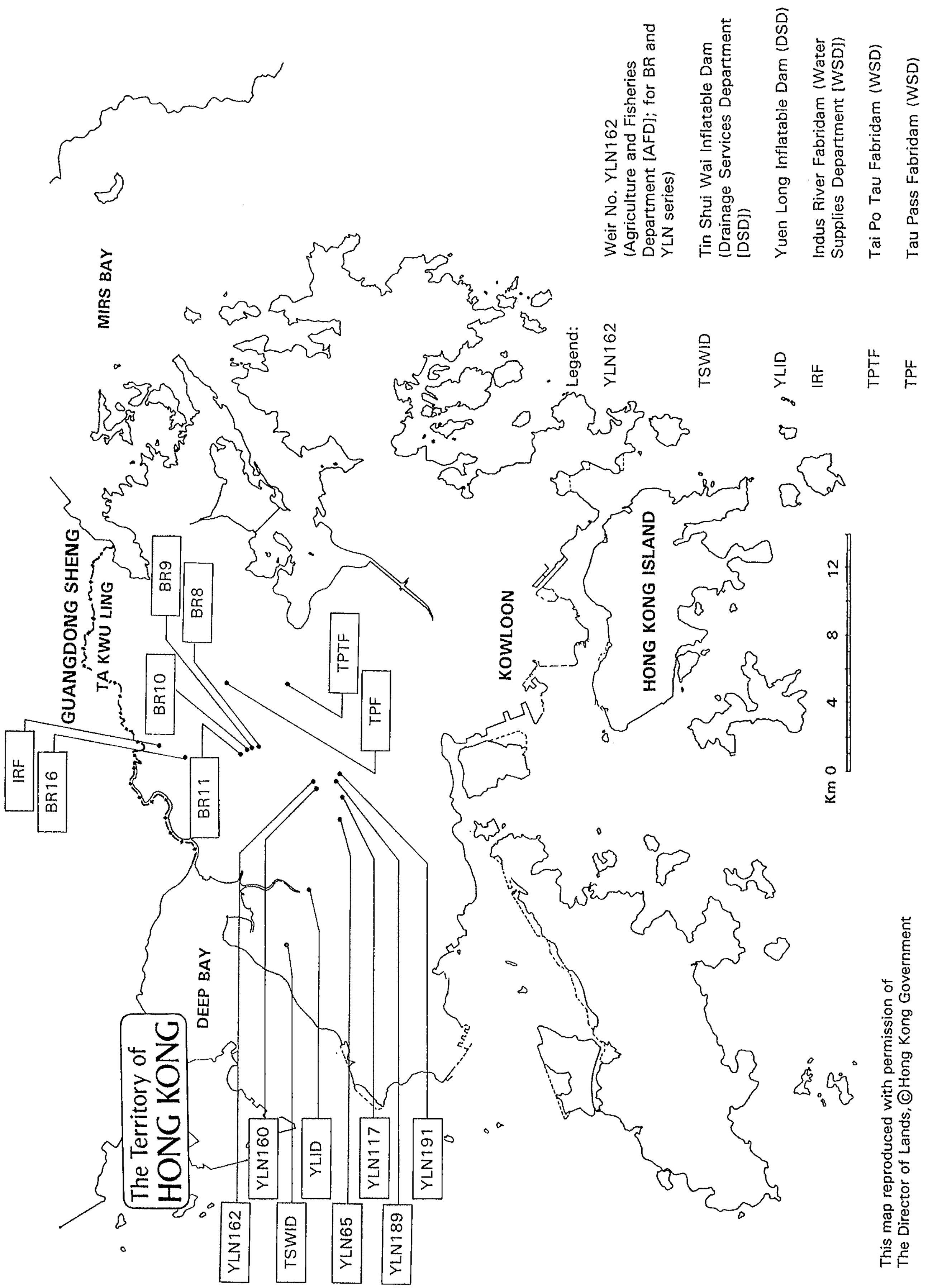


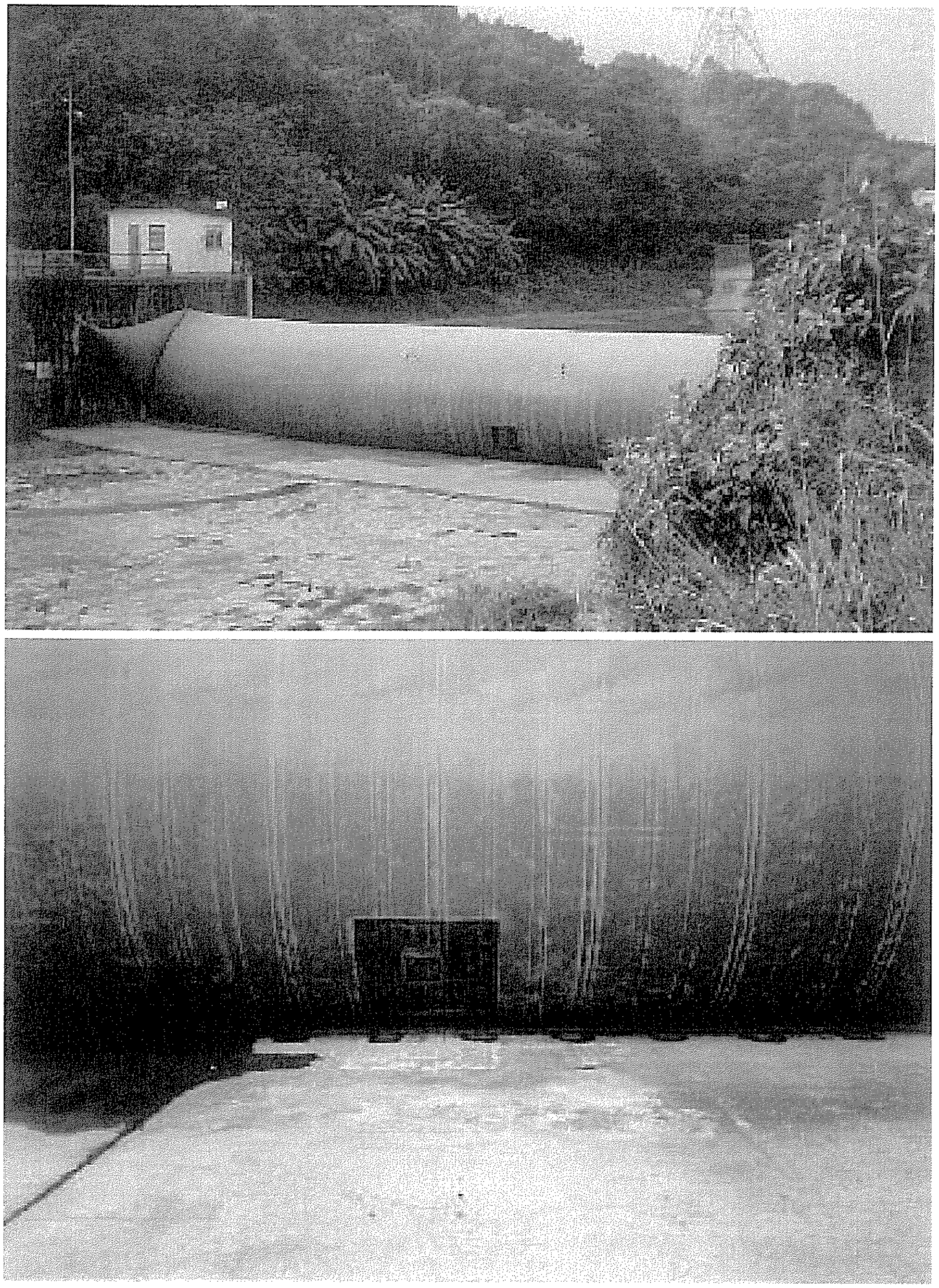
Fig. 1. Location of inflatable dams.

Fig. 2. Dam No. BR16 on Beas River.

Damage due to flow-induced vibration

The rubbing of the inflatable body against the foundation and the sides of the dam can lead to wearing of the dam body. Local wearing will be accelerated if the inflatable dam body is rubbing against a sharp object or even a relatively smooth object such as a cobble. The Tai Po Tau dam (built by the Water Supplies Department) had experienced such damage recently. The dam body was partially deflated for the regulation of water flow and part of the dam body was in contact with the foundation. It was suspected that the abrasion effect due to "flow-induced vibration" was most severe at that point and a small hole was found at the bottom of the inflatable dam (Fig. 3). This mechanism of damage was supported by the discovery of a depression on the concrete foundation. The location of the depression mirrors the damage on the inflatable dam body. The smoothness of the surface suggests that it was found by a slow wearing process. Both the shape and size of the depression match the damage of the inflatable dam.

Fig. 3. Tai Po Tau Dam on Tai Po River.



For the avoidance of flow-induced vibration, partial dam deflation should not be carried out unless it is absolutely necessary.

Damage due to dam inflation and deflation
During dam deflation, it is possible for the dam body be punctured by a sharp object immediately downstream of the dam. At Weir No. YLN 65, a puncture was discovered at the downstream side of the weir and a broken glass bottle was found to have caused the damage. Sharp objects deposited upstream of the the dam can also cause dam damage during dam inflation. Regular cleaning of the river beds both upstream and downstream of the dam is of help to avoid such damage.

Damage due to objects carried from upstream of the weir
During severe rainstorms, rivers can become very violent and debris is often carried by the flood water. Water-borne sharp objects such as steel bars or metal frames could cause damage to the dam. Due to the extensiveness of flooding in Hong Kong, there has not been any effective measures provided to prevent such damage.

Discussion

Repair of inflatable dams
The repairs on the rubber dams are normally associated with

punctures and cuts. The repair of punctures of inflatable dams is normally a very simple process. Dam repairs are infrequent. For example, only seven repairs were made during the period from March 1993 to June 1995.

Contractual arrangement

A special contractual arrangement is used by the Agriculture and Fisheries Department for the installation of the inflatable dams (386–392 1995). The important component of the specification is stated below:

> The existing weirs are to be demolished with a new weir designed as a full width barrage to replace them. The lower part of the weir will comprise a concrete weir with a low crest or sill. ...
>
> On top of the sill will be fitted a gate or barrage weir across the full width of the weir. The gate or barrage must be capable of opening, dropping or collapsing down flush with the sill level over the full width of the weir within one hour. No overhead mechanical equipment will be permitted over the full width of the weirs.

Contractual claims

Four contracts were awarded to four different contractors between April 1991 and December 1992. No contractual claims have arisen due to the contractual arrangement. It can be said that the contractual arrangement is reasonably well tested for further large-scale application. The average cost of a rubber dam is about HK$150 000 (CDN$27 000) per metre width for the rubber dams installed by the Agriculture and Fisheries Department.

Conclusions

The use of rubber dams as agricultural weirs is a relatively new concept in Hong Kong. Due to the rubber material itself, there is often a concern if the dam material may be subject to repeated damage due to vandalism. However, experience suggests that the installation of rubber dams is feasible in Hong Kong. It can be said that the use of a relatively innovative material in civil engineering is met with success in Hong Kong.

Acknowledgements

The author is grateful to the Director of Agriculture and Fisheries Department for his assistance and support rendered during the collection of information and data for this note. The views expressed in the note are those of the author's and do not represent the views of the Agriculture and Fisheries Department; and its director is not responsible for the accuracy of information contained in this note.

References

Ellul, M.D. 1992. Mechanical fatigue. *In* Engineering with rubber: how to design rubber components. *Edited by* A.N. Gent under the auspices of the Rubber Division of the American Chemical Society. Carl Hanser Verlag, Munich, Germany.

Forman, D.B. 1973. Rubber technology. 2nd ed. *Edited by* M. Morton. Sponsored by the Rubber Division of the American Chemical Society. Van Nostrand Reinhold Company, New York, N.Y.

Hamed, G.R. 1992. Materials and compounds. *In* Engineering with rubber: how to design rubber components. *Edited by* A.N. Gent under the auspices of the Rubber Division of the American Chemical Society. Carl Hanser Verlag, Munich, Germany.

Kahl, T., and Ruell, S. 1989. Flashboard alternatives including rubber dams. Proceedings, Waterpower '89, Niagara Falls, N.Y., pp. 447–456.

Moorthy, C.M.D., Reddy, J.N., and Plaut, R.H. 1995. Three-dimensional vibrations of inflatable dams. Thin-Walled Structures, **21**: 291–306.

Tam, P.W.M. 1995. Procurement of specialised civil engineering facility by design and build contract. Proceedings, International Congress on Construction, Singapore, pp. 204–210.

Swanton Village

About | Departments | Utilities | Businesses | History | Contact Us

Electric

Hydroelectric Power Plant Dam

News and Activites

Read about the latest news and events in the community.

Welcome to the Largest Inflatable Rubber Dam in North America, located at the Highgate Falls Dam Site in Highgate Falls, Vt.

Imagine a dam that goes up and down automatically to keep the proper water level in the reservoir. It never has to be painted or rustproofed. It has a smooth top surface so that, in a storm or flood, trees and debris slide over it instead of getting caught in the gates. In cold weather, ice won't jam it up. It's environmentally friendly, too: it has an inconspicuous profile and can be made to fold down on top of itselt now and then to let sediment pass downriver.

The dam is an inflatable dam or as it's called by it's manufacturer, Bridgstone of Tokyo, "Rubber Dam". The dam is 15 feet in diameter and 220 feet long, made of nylon re-enforced rubber, it is the largest inflatable rubber dam in North America. This dam is also one of the first in the world to have a pressurized hatchway allowing dam operators to walk right inside.

The operation of the dam is controlled by computerized control system which is programmed to monitor the pond level by inflating or deflating the dam by automatically turning on one of two inflation blowers or one of the two motorized deflation valves. The systems sensitivity is such that it keeps the pond level to within +/- 0.1 feet of the pond level setpoint. The blowers that fill the dam with nearly 40,000 cubic feet of air at 7 1/2 pounds per square inch are high volume, low pressure units that can inflate the dam from elevation 175 ft.asl to 190 ft. asl in about 1 hour.

Return to the previous ***Hydroelectric Power Plant*** section
Return to Utilities section

Residential Demand Rates | Residential Service Rates | Commercial Service Rates | Security Lighting Service Rates | Industrial Service Rates | Municipal Street Lighting Rates | Industrial Off-Peak Service Rates |Appliance energy consumption | Hydroelectric Powerplant | NEPPA.com

MASON BILAFER PARTNERSHIP

THE WORLD OF INFLATABLE RUBBER DAMS AND SPILLWAY GATES

Click on image to enlarge it

News: News of the latest projects from Europe and around the world.

Tenders: Tender announcements and contract news.

Featured Project: A description of a major project involving a rubber dam or spillway gate.

Rubber Dams: The inflatable rubber dam has been used to control flow in rivers for more than forty years. The basic characteristics of this unique structure, in terms of design, construction and operation, are quite well known to potential investors and/or operators. Less well known, perhaps, is the degree to which the technology has advanced over the past four decades and how many projects are actually being carried out every year. At the present time, three of the world's largest inflatable rubber dams are being installed as storm surge barriers at a site in the Netherlands. The fact that these dams are 8.0m high would hardly have been considered possible five years ago.

Spillway Gates: The pneumatically-operated spillway gate manufactured by Obermeyer Hydro Inc. is a much more recent innovation than an inflatable rubber dam but no less unique. It has been in use for about fifteen years and already gates with heights of approximately 5.5m (18ft) have been installed. Up to now, most spillway gates have been installed in the USA, but, with contracts in India, Peru and Germany under way, it is likely that its use will become as global as that of the inflatable rubber dam within a very short period.

Click on image to enlarge it

Mason Bilafer Partnership: Mason Bilafer Partnership (MBP) is an independent consulting engineer and a specialist in the field of inflatable rubber dams and pneumatically-operated spillway gates. Although based in the United Kingdom, MBP offers the following services to owners, operators, consultants and contractors around the world:

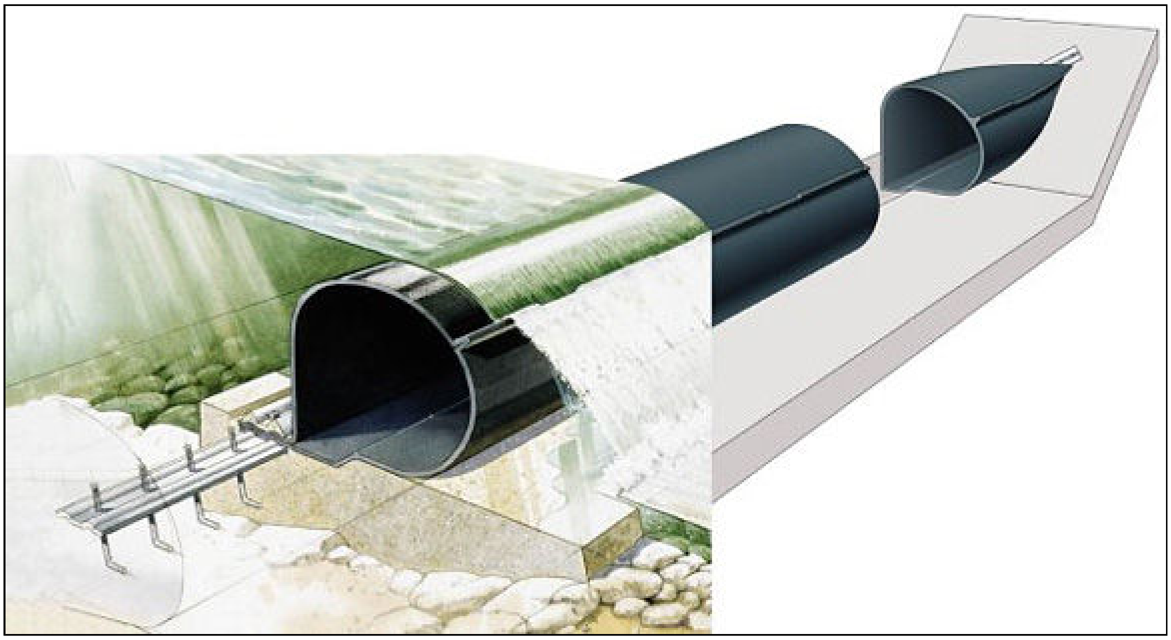
- Technical advice
- Budgetary estimates
- Preparation of tender documentation
- Contract administration
- Project management
- Installation supervision
- Troubleshooting
- Dispute resolution

Click on image to enlarge it

Don Mason of MBP has worked on rubber dam and spillway gate projects for more than ten years.

RUBBER DAM

Design & Features

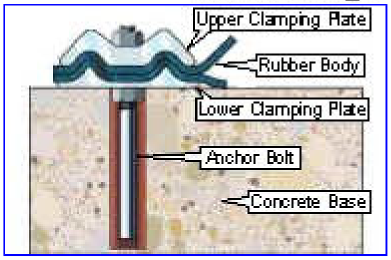


Click To Expand

Strong Body

An inflatable bladder made of heavy-duty, nylon-reinforced rubber, with EPDM cover to withstand ozone and ultraviolet light. Thickness of bladder ranges from 9.5 to 25mm, depending upon the dam height. The minimum safety factor for the bladder is 8.0.

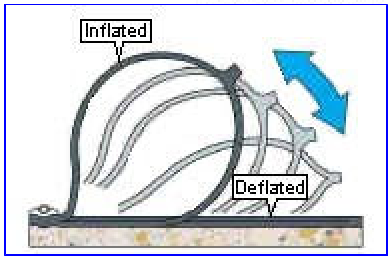
Click To Expand



Secure Anchor

The bladder is anchored to the foundation using a simple clamping system composed of anchor bolts and steel clamping plates. This simple design produces an extremely dependable air-tight seal. The system can be installed quickly but firmly with standard tools. The minimum design safety factor for the anchor

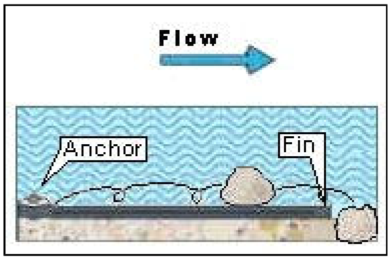
Click To Expand



Simple & Reliable

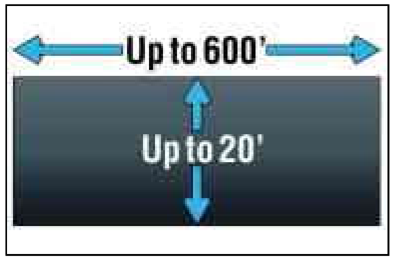
A low pressure system, usually between $0.05 kgf/cm^2$ to $0.6 kgf/cm^2$ depending on dam height. Air is supplied using air blowers. No overhead structure or hydraulic system are required. Inflation & deflation of rubber dams is highly reliable and little maintenance is required.

system is 3.0.



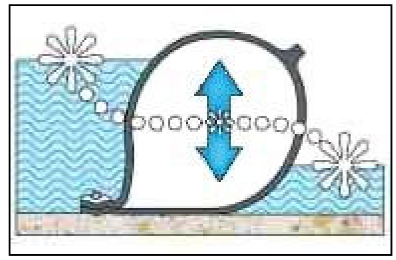
Flat On Foundation

The FIN structure allows the fabric to Lay-Flat when deflated. This prevents damage from debris or ice. The lay flat characteristic eliminates the bulge at the end of the deflated body, which is prone to serious vibration & abrasion. It also permits passage of debris.



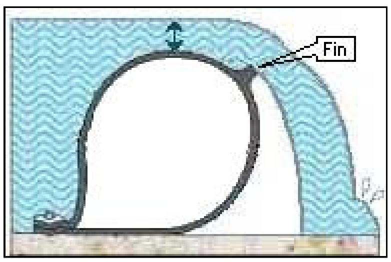
Longer Span

The Bridgestone Rubber Dam permits very long spans, thus reducing the need for intermediate piers necessary in steel gate installations. The long span of the rubber dams also maximizes discharge as there are few piers obstructing the water flow.



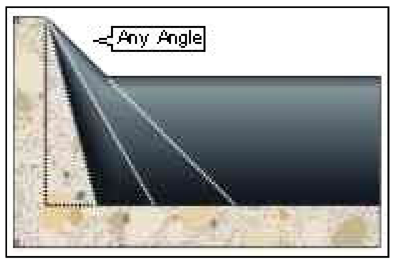
Flexible Control

Inflation & deflation can be manual or automatically controlled. The automatic control system can monitor the upstream water level and adjust the air pressure in the dam to maintain a prescribed water level in the upstream pool.



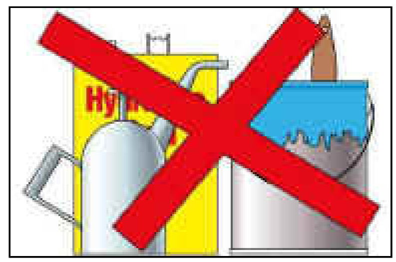
Oscillation Reduction

When inflated, the FIN structure works as a deflector to create aeration below the fin. This effectively reduces the phenomenon of oscillation up to a 50% overflow when compared to FIN-LESS bladders.



Variable Side Slope

Rubber Dams can be installed in rivers with any side slope angle, eliminating the necessity of modification to river bank, unlike steel gates which can only be installed on vertical side slopes.



Low Maintenance

Other than normal maintenance of control equipment, blowers and actuators, rubber dams are virtually maintenance free. This is a big advantage over steel gates, where removal of rust, painting and changing of hydraulic oil are required.

SNC • LAVALIN

www.snclavalin.com

SNC-LAVALIN Inc.
1133 Topsail Road
Mount Pearl, NL A1N 5G2
Canada
Tel.: (709) 368-0118
Fax: (709) 368-3541