Department of Fisheries and Oceans

ISSUED FOR USE

FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

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EXECUTIVE SUMMARY

Fulton Dam is a 17 m high and 55 m long concrete gravity structure, located about nine kilometres south of Granisle, B.C. and about six kilometres upstream of Babine Lake on the Fulton River. The storage provided by the dam, constructed in 1968, has been used to supply water to the Fulton River Spawning Channel facility operated by the Department of Fisheries and Oceans (DFO). To address the hydrotechnical issues as part of the Fulton Dam Safety review, Hay & Company Consultants (Hayco), was retained by DFO to carry out a hydrotechnical study involving a hydrologic assessment to determine the appropriate Inflow Design Flood (IDF), a flood routing and hydraulic analysis to assess the capacity of the dam to pass the IDF, and a dam break analysis to determine the impacts of a potential dam breach.

A site visit was conducted to gain familiarity of the project site and to gather background information. This was followed by a hydrologic analysis to determine the characteristics of the Fulton River watershed, to compile regional climatic and hydrometric data, and to derive the appropriate IDF. Based on previous dam safety review, it is understood that Fulton Dam is currently classified as a High Consequence dam. Accordingly, the IDF for Fulton Dam was considered to be 1/3 between an annual exceedance probability of 1 in 1000 and the Probable Maximum Flood (PMF). As recommended by the 2007 CDA Dam Safety Guidelines, both the IDF for the summer-autumn season and that for the spring season were determined.

Flood routing analyses were performed to assess the capacity of the dam outlet works to pass the IDFs. Two rule curves were investigated: Case 1 with the capacity of the regulating tunnel and the overflow spillway and Case 2 with the combined capacity of all available outlet works. Results of the analysis indicate that there would be sufficient freeboard available at the dam.

An assessment of the throw distance of the flip buckets for the overflow spillway was also conducted. Results show that the bucket jet would land beyond the falls downstream of the dam during the spring IDF. Therefore, the bucket lip would be much higher than the tailwater level, meeting the requirement stipulated by the CDA guidelines.

A reservoir drawdown analysis was also carried out to determine the amount of time required to drain the reservoir through the outlet works during an emergency. When only the regulating tunnel and the overflow spillway are operated, it would take approximately 29 days to drain the entire reservoir. When all the outlet works are operated, it would take approximately 24 days to drain the reservoir.

An updated dam break analysis was performed to determine the impacts of a hypothetical dam breach. It was assumed that the dam would breach during the spring IDF. To complete the analysis, available topographic maps provided by DFO were digitized. Such information was used to extract cross sections along the reach between the dam and Babine Lake. Dam breach results from the SMPDBK model were further refined by taking into account the backwater effects as a result of the highway bridge crossing and the control at Babine Lake. An inundation map was prepared based on the resulting flood profile. Results of the dam break analysis indicate that part of the Spawning Channel No. 1, part of the Spawning Channel No. 2, the highway bridge, and the Indian settlement near the outlet into Babine Lake would be inundated. The DFO facility buildings located southeast of the highway bridge would only have minor flooding concerns.

TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS

TABLES

FIGURES

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

Fulton Dam is located about nine kilometres south of Granisle, B.C. and about six kilometres upstream of Babine Lake on the Fulton River (Figure 1). The dam is a concrete gravity structure, approximately 17 m high and 55 m long, which is used to supply water to the Fulton River Spawning Channel facility operated by the Department of Fisheries and Oceans (DFO). The dam was constructed in 1968.

Dam safety reviews are required every ten years to satisfy Canadian Dam Safety regulations. The 1997 Dam Safety Review (DSR) was carried out by UMA Engineering Ltd. (UMA) with geotechnical input from EBA Engineering Consultants Ltd. (EBA). The 1997 DSR identified several hydrotechnical issues related to dam safety that required attention.

EBA completed the next 10-year DSF for Fulton Dam in February 2008 and this report addressed the geotechnical issues relating to dam safety. Their review report included the inspection of the dam, the low level outlet sluiceway, the regulating outlet structure, two concrete tunnel bulkheads accessed from Beaver Creek near the Beaver Creek valve house, the downstream portal of the eastern tunnel, and a section of the pipeline that crosses a landslide initiated during construction. This report did not address hydrotechnical issues related to dam safety. The hydrotechnical issues include a hydrologic assessment to determine the appropriate Inflow Design Flood (IDF), a flood routing and hydraulic analysis to assess the capacity of the dam to pass the IDF, and a dam break analysis and inundation mapping to evaluate impacts of a potential dam breach.

Hay & Company Consultants (Hayco), a division of EBA, was requested by DFO to undertake a hydrotechnical assessment of the dam as part of a Dam Safety Review. Hayco was awarded the work based on a proposal submitted on July 16, 2008. The study was awarded to Hayco on July 29, 2008.

This report addresses the hydrotechnical issues pertaining to dam safety as outlined above.

2.0 REVIEW OF BACKGROUND INFORMATION

The following documentation provided by DFO was reviewed to obtain relevant project background information:

<u>Previous Reports and Manuals</u>

- Department of Fisheries and Oceans, March 1988. Fulton River Dam and Flow Control Works Operation and Maintenance Manual.
- Department of Fisheries and Oceans, June 2008. Fulton River Dam and Flow Control Works Operation and Maintenance & Surveillance Manual.
- EBA Engineering Consultants Ltd., December 2002. Re-Inspection of Water Supply Tunnels to Fulton River and Pinkut Creek Spawning Channels Babine Lake, B.C.

- EBA Engineering Consultants Ltd., February 2007. Fulton Dam 2007 Inspection.
- EBA Engineering Consultants Ltd., January 2008. Re-Inspection of Water Supply Tunnels & Related Facilities for the Fulton River Spawning Channel, Babine Lake, B.C.
- Patrick Fawkes & Associates, July 1986. Inspection Report of Fulton Lake Dam.
- Patrick Fawkes & Associates, September 1986. Fulton River Project Dam Breach Inundation Studies.
- UMA Engineering Ltd., August 1997. Fulton Dam – Dam Safety Review.

Drawings and Mapping

- Miscellaneous topographic maps and drawings of the Fulton River projects.

Records and Reporting

- Log sheet records of Fulton Lake elevation, valve opening percentage, gauged flow in the river, gauged flow in Spawning Channel No. 2, and estimated daily inflow based on the record of outflow and change in storage. Records available from 1973 to 2008 with some data gaps.

3.0 PROJECT SITE DESCRIPTION

The Fulton River Facility consists of a dam, an intake structure on Fulton Lake, a regulating outlet structure near the toe of the dam, a supply tunnel and pipeline system, and two spawning channels located approximately 2 to 3 km downstream of the dam. Spawning Channel No. 1 located more upstream is fed by river water while Spawning Channel No. 2 located approximately 800 m downstream of Channel No. 1 is fed by the supply tunnel and pipeline system.

Fulton Dam is a 17 m high and 55 m long concrete gravity dam with a 12 m high and 32 m long free overflow section. The overflow section or the spillway, curved in plan, has a crest elevation of 776.33 m (2547 ft), which is 5.18 m (17 ft) lower than the dam crest elevation of 781.51 m (2564 ft). A flip bucket is used as the terminal structure for the spillway. A gatehouse, located near the north abutment of the dam, contains a pair of gates to allow the reservoir to be controlled during periods of high flow. The discharge from both gates passes through a rectangular low level portal that exits at the toe of the north abutment.

The regulating intake structure for the pipeline, that supplies water to one of the two spawning channels, is located approximately 200 m north of the dam. An inlet channel protected by a log boom was constructed upstream of the structure, which contains a three level gated concrete intake tower equipped with trashracks. The intake structure also includes a main gate that controls water flow into the receiving tunnel and a flooding gate that can be used to fill the tunnel before the main gate is opened.

The intake tower can convey flows either to the regulating outlet structure near the toe of the dam through a 3.7 m diameter, 150 m long tunnel; or to Spawning Channel No. 2 through the 2.3 m diameter water supply tunnel and pipeline system that branches off the tunnel leading to the regulating outlet structure. The regulating outlet structure has three hollow cone discharge valves and is operated from a valve house above the outlets.

An overall plan of the Fulton River Facility is shown on Figure 2. Additional plans and drawings related to the dam structure can be found in Appendix A.

4.0 SITE VISIT

Mr. Robert J. Wallwork P.Eng. and Ms. Maria Lau, P.Eng. (Hayco) visited the site on September 17, 2008 accompanied by Mr. Tim Renaud (DFO). A meeting was held at the Fulton River Fish Hatchery office prior to visiting the dam and related facilities. The meeting was attended by:

- Bob Wallwork (Hayco – Project Manager)
- Maria Lau (Hayco – Project Engineer)
- Tim Renaud (DFO – Project Engineer)
- Brad Thompson (DFO – Fulton Facility Manager)
- Dennis Merkley (DFO – Fulton Facility Maintenance Superintendent).

Dam operations and data records were discussed together with weather station and snow course records.

The above personnel, with the exception of Brad Thompson, then drove to the dam site to view the dam, spillway, intake structure, and low level outlet facilities. This was a general site orientation to familiarize Hayco staff with the dam and its appurtenances.

The dam spillway was in operation during the site visit with a forebay reading of 776.46 m (2547.45 ft) representing 0.14 m (0.45 ft) head on the spillway crest. The impoundment reservoir at the dam is narrow and highly contorted such that the effective fetch length upstream of the dam would be limited to a few hundred metres at most. Wind generated waves are expected to be relatively minor based on the limited fetch.

A brief inspection was made of the gatehouse where flow conditions over the spillway crest were viewed through the side window. The path and stairs at the left abutment were used to gain access to the Regulating Intake Structure which houses the 30-inch (0.762 m) and twin 84–inch (2.134 m) Howell-Bunger (hollow-cone) valves. Only the 30-inch valve was in operation during the site visit.

The intake structure, located a couple hundred metres to the north of the dam, was inspected next. Gates were open but not visible as floor grates covered the portal openings. At this location the reservoir has a longer fetch distance and consequently wind/wave interactions would result in higher waves.

The river channel was inspected near the first and second spawning channels and later further downstream at the counting fence. Lastly, the Fulton River outlet to Babine Lake was viewed from the shoreline at the right bank. Site photos were taken to document the inspection sites together with GPS coordinates for each.

5.0 HYDROLOGIC ANALYSIS

5.1 WATERSHED CHARACTERISTICS

The Fulton River watershed has an area of approximately 1400 km^2, and it is bounded by Skeena Mountains in the northwest. Headwaters of Fulton River drain first to Bristol Lake, near the northern boundary of the watershed, then south to Chapman Lake. Approximately 15 km downstream of Chapman Lake is Fulton Lake, which in turn drains to Babine Lake through a short steep river channel. The watershed elevation varies from approximately 1980 m at the Skeena Mountains to 760 m at the project site. While 12% of the watershed has an average slope of 13.8%, the remaining part of the watershed has an average slope of 0.6%. The median basin elevation is approximately 900 m. The watershed boundary of Fulton River at the dam is shown on Figure 3.

5.2 CLIMATIC AND SNOW COURSE DATA

A number of climate stations operated by the Meteorological Service of Canada (MSC) are located within the region. In view of its close proximity to the project site, elevation, and relatively long period of record, the Topley Landing station was considered to have climate data that is the most representative of the climate conditions at the project site. A summary of information for this station is shown in Table 1.

TABLE 1: REGIONAL CLIMATE STATIONS

Station Name	Station No.	Elevation	Period of Record	Distance to Dam
Topley Landing	1078209	722 m	1962 – 2007	5 km

According to the 1971 – 2000 climate normals, the mean annual precipitation at Topley Landing is 533 mm. Most rainfall occurs in early summer and fall (May – October) while most snowfall occurs in winter (November – February). Figure 4 illustrates the monthly precipitation totals at Topley Landing.

Monthly temperature from the Topley Landing station is shown on Figure 5. Mean annual temperature is 3°C for the period from 1971 to 2000. Mean daily temperatures are generally above freezing from April to October, and it ranges from about -10°C in January to 14.5°C in July.

Daily precipitation totals are available at the Topley Landing station, but rainfall intensity frequency data at this station has not been published by MSC. Published rainfall intensity frequency data within the region is available at the Smithers Airport (#1077500 at elevation of 521 m) and Quick (#1076638 at elevation of 533 m) climate stations, which are located west of the Skeena Mountains. A frequency analysis (Gumbel distribution) was performed with the annual maximum daily rainfall totals at Topley Landing. Results of the analysis were increased by 13% to account for the fact that daily (calendar day) totals under-predict the 24-hour windows of maximum precipitation. By comparing the adjusted values to the published data, it was determined that the rainfall intensities at Topley Landing are slightly lower than those at Smithers Airport and Quick, likely because Topley Landing is located east of the Skeena Mountains with a relatively dryer climate. As a conservative approach, the average rainfall intensities from these three stations were used in the hydrologic analysis. Table 2 shows the rainfall intensity frequency data for all three stations for various return periods.

TABLE 2: RAINFALL INTENSITY FREQUENCY DATA

Return Period (Years)	24-Hour Rainfall Total (mm)			
	Topley Landing	Smithers A	Quick	Average
2	25.5	29.9	27.0	28
5	34.1	39.0	36.7	37
10	39.7	45.1	43.2	43
50	52.1	58.5	57.3	56
100	57.3	64.2	63.3	62
1000	74.7	82.8	83.0	80

The Water Stewardship Division of the BC Ministry of Environment has conducted manual snow surveys in the region since 1969. Five survey sites were considered to be useful to the study, and they are listed in Table 3.

TABLE 3: REGIONAL SNOW COURSE SITES

Station Name	Station No.	Elevation	Period of Record	Distance to Dam
Hudson Bay Mtn.	4B03A	1452 m	1972 – 2007	67 km
Chapman Lake	4B04	1485 m	1965 – 2007	34 km
Tachek Creek	4B06	1133 m	1968 – 2007	20 km
McKendrick Creek	4B07	1048 m	1968 – 2007	37 km
Mount Cronin	4B08	1491 m	1969 – 2007	40 km

In general, manual measurements of snow depth were made on the first of the month in March, April, May, and June. Average snow depth and water equivalent for the period of record is summarized in Table 4 for each station.

TABLE 4: AVERAGE SNOWPACK DATA										
	Snowpack Depth (cm)					Snow Water Equivalent (mm)				
Date	4B03A	4B04	4B06	4B07	4B08	4B03A	4B04	4B06	4B07	4B08
1-Mar	144	134	82	95	161	455	408	195	257	509
1-Apr	150	143	84	94	173	516	468	222	287	597
1-May	132	131	56	66	165	525	489	173	235	645
15-May	105	115	28	34	164	441	506	92	133	715
1-Jun	68	113	N/A	4	159	302	532	N/A	15	736
15-Jun	25	N/A	N/A	N/A	115	122	N/A	N/A	N/A	593

The data shows that the average maximum snowpack depth in the region usually occurs in late March, and the average maximum snow water equivalent usually occurs in May for stations situated at higher elevations (i.e. Hudson Bay Mtn., Chapman Lake, and Mount Cronin) and in early April for stations situated at lower elevations (i.e. Tachek Creek and McKendrick Creek).

5.3 HYDROMETRIC DATA

Estimated daily inflow data is available for Fulton Lake based on the record of outflow and change in storage. This information was provided by DFO, and the period of record is from 1973 to 2008, with intermittent data gaps. Additional streamflow data was obtained from Environment Canada for the hydrometric station Fulton River at the Mouth (08EC002). This station was active from 1963 to 1970 (8 years), and it had been regulated since 1968. Unregulated peak flow data at this station is available from 1964 to 1967.

By analyzing available streamflow data at the lake, it was determined that peak flows generally occur during the snowmelt season (April to June). Annual maximum daily inflows and annual snowmelt runoff volumes (April 1 to June 30) are presented in Table 5. It should be noted that the highest annual maximum daily inflow and annual snowmelt runoff volume occurred in 2002.

TABLE 5: MAXIMUM DAILY INFLOW AND SNOWMELT RUNOFF VOLUME					
	Max Daily Inflow	Snowmelt Runoff Volume		Max Daily Inflow	Snowmelt Runoff Volume
Year	(m^3/s)	(m^3-days)	Year	(m^3/s)	(m^3-days)
1964	190	4594	1987	107	3434
1965	139	4366	1988	121	4112
1966	140	3761	1989	142	3304
1967	123	4559	1990	105	4177
1968	N/A	N/A	1991	89	3156
1969	N/A	N/A	1992	104	4818
1970	N/A	N/A	1993	152	3998

TABLE 5: MAXIMUM DAILY INFLOW AND SNOWMELT RUNOFF VOLUME					
Year	Max Daily Inflow (m³/s)	Snowmelt Runoff Volume (m³-days)	Year	Max Daily Inflow (m³/s)	Snowmelt Runoff Volume (m³-days)
1971	N/A	N/A	1994	111	4555
1972	N/A	N/A	1995	96	2816
1973	N/A	N/A	1996	97	4287
1974	109	4381	1997	242	6377
1975	103	2820	1998	120	3196
1976	170	5666	1999	105	4603
1977	108	3116	2000	61	2858
1978	80	3145	2001	67	2954
1979	95	3104	2002	302	6842
1980	85	2660	2003	103	3459
1981	132	4233	2004	90	2868
1982	169	4700	2005	141	3570
1983	99	3207	2006	104	2596
1984	94	3164	2007	200	6069
1985	116	3464	2008	167	3834
1986	103	3543	-	-	-

Historical daily lake elevations from 1973 to 2008 with intermittent data gaps were also provided by DFO. In general, the annual minimum lake elevation usually occurs in April prior to the snowmelt season, and the annual maximum lake elevation usually coincides with the snowmelt runoff peak in May or June. Lake elevations generally remain above the spillway crest elevation throughout the summer-autumn period (July to October) and drop below the spillway crest elevation during the winter period (November to March). It should be noted that lake elevations are related to the operation of the reservoir, which could be different each year due to the forecasted inflow volumes. Annual maximum and minimum daily lake levels are listed in Table 6.

TABLE 6: MAXIMUM AND MINIMUM DAILY LAKE ELEVATIONS					
Year	Lake Elevation (m)		Year	Lake Elevation (m)	
	Annual Max	Annual Min		Annual Max	Annual Min
1964	N/A	N/A	1987	776.67	772.26
1965	N/A	N/A	1988	777.19	771.84
1966	N/A	N/A	1989	776.56	771.00
1967	N/A	N/A	1990	776.98	772.24
1968	N/A	N/A	1991	777.05	771.20
1969	N/A	N/A	1992	777.21	773.19

TABLE 6: MAXIMUM AND MINIMUM DAILY LAKE ELEVATIONS					
Year	Lake Elevation (m)		Year	Lake Elevation (m)	
	Annual Max	Annual Min		Annual Max	Annual Min
1970	N/A	N/A	1993	776.97	772.66
1971	N/A	N/A	1994	777.35	772.94
1972	N/A	N/A	1995	776.45	770.22
1973	775.57	774.31	1996	777.04	769.83
1974	776.96	769.99	1997	777.15	770.02
1975	776.33	770.26	1998	776.85	769.92
1976	777.44	769.49	1999	777.44	769.72
1977	776.61	771.08	2000	776.66	770.44
1978	776.79	771.10	2001	777.18	769.76
1979	776.72	770.78	2002	777.31	770.27
1980	776.76	771.03	2003	776.88	770.38
1981	776.94	773.08	2004	776.93	770.99
1982	776.80	770.60	2005	777.33	773.67
1983	777.01	771.23	2006	776.87	770.53
1984	776.81	772.24	2007	777.26	770.61
1985	776.95	772.28	2008	776.69	771.07
1986	776.96	771.33	-	-	-

Figure 6 shows the annual hydrograph and lake elevations of Fulton Lake in 2002 with the largest peak flow and snowmelt runoff volume. Graphs of the estimated daily inflow data and daily lake elevation data from 1973 to 2008 can be found in Appendix B.

5.4 DAM CLASSIFICATION

Based on previous dam safety reviews, it is understood that Fulton Dam is currently classified as a High Consequence dam. The usual standard for Inflow Design Floods (IDFs) for High Consequence dams is 1/3 between an annual exceedance probability of 1 in 1000 and the Probable Maximum Flood (PMF). In general, the PMF is defined as the most severe flood that may reasonably be expected to occur at a particular location and is generated by the Probable Maximum Precipitation (PMP). Two PMFs were considered in the current study: summer-autumn PMF and spring PMF. The summer-autumn PMF is generated by the summer-autumn PMP. The spring PMF is defined as the maximum of the PMF computed with spring PMP and snow accumulation with frequency of 1/100 year and the PMF computed with the Probable Maximum Snow Accumulation (PMSA) and rainstorm with frequency of 1/100 year. The reason for computing two separate PMFs for the spring season is that it would not be reasonable to assume that snow accumulation and a spring rainstorm, which are two independent phenomena, are simultaneously extreme.

5.5 INFLOW DESIGN FLOOD

Both the IDF for the summer-autumn season and that for the spring season were determined in accordance with the 2007 CDA guidelines. The summer-autumn IDF was determined to be a flood as a result of a rainstorm with rainfall total being 1/3 between the 1000-year 24-hour rainfall total and the 24-hour summer-autumn PMP. The spring IDF was determined to be a flood with a snowmelt runoff volume (April 1 to June 30) being 1/3 between the 1000-year snowmelt runoff volume and the spring PMF volume, which is the greater of the following two cases:

- Case 1: PMF volume computed with 24-hour spring PMP and 100-year snow accumulation.
- Case 2: PMF volume computed with PMSA and 100-year 24-hour rainstorm.

Summer-Autumn IDF

The average rainfall intensity frequency data from Topley Landing, Smithers Airport and Quick climate stations was used to determine the 1000-year rainfall depth for 24-hour duration. The 1000-year 24-hour rainfall was determined to be 80 mm.

The 24-hour summer-autumn PMP was estimated using the Hershfield method described in the Rainfall Frequency Analysis for Canada (Hogg and Carr, 1985). The Hershfield empirical relationship is as follows:

$$K_{M\,24} = 19 \times 10^{-0.000965 X_{24}}$$

$$X_{PMP} = X_{24} + K_{M\,24} \times S$$

Where: $K_{M\,24}$ is a frequency factor for a 24-hour duration rainfall;

X_{24} is the mean annual 24-hour extreme rainfall (mm);

X_{PMP} is the PMP for a 24-hour duration (mm);

S is the standard deviation for a 24-hour duration rainfall (mm).

The 24-hour summer-autumn PMP determined by this method is 153 mm.

The rainfall total used to determine the summer-autumn IDF was calculated as 1/3 of the way from the 1000-year quantity to the probable maximum precipitation, or 104 mm.

The US Soil Conservation Service (SCS) unit hydrograph method was then applied to determine the summer-autumn IDF flow hydrograph from the calculated 24-hour rainfall of 104 mm. The SCS Type 1A distribution was selected to define the distribution of rainfall over 24-hours. The hydrologic model used in the runoff analysis was HEC-HMS version 3.0.1, developed by U.S Army Corps of Engineers. The catchment area was determined to consist of mainly forested areas. Soil Type C, representing soil with a moderate infiltration rate, was chosen for the study area assuming Antecedent Moisture Condition III (rain and low temperatures over the last five days causing saturated conditions). Slopes, elevations and channel lengths were taken from topographic maps to estimate a combined time of concentration for the total basin area.

The peak inflow during the summer-autumn IDF was determined to be 1146 m^3/s. Figure 7 shows the summer-autumn IDF hydrograph.

Spring IDF

A total of 39 years of estimated daily inflows to Fulton Lake were used to calculate the annual snowmelt runoff volume (April 1 to June 30). The 1000-year snowmelt runoff volume was determined by carrying out a frequency analysis using Environment Canada's CFA 3.1 program. Distributions that provided a good fit to the data were used. Table 7 summarizes the results of the frequency analysis. The 1000-year snowmelt runoff volume was determined to be 11,950 m^3/s-days.

TABLE 7: SNOWMELT RUNOFF VOLUME FREQUENCY ANALYSIS

Return Period (Years)	Snowmelt Runoff Volume (m^3/s-days)
100	8,170
1000	11,950
10000	17,090

The Case 1 spring PMF was computed using the 24-hour spring PMP combined with 100-year snow accumulation. The ratio of average daily maximum rainfall in spring, to that in the summer-autumn season, was determined to be 0.58 at Topley Landing. This factor was applied to adjust the summer-autumn PMP to the spring PMP, resulting in a rainfall total of 89 mm or rainfall volume of 1438 m^3/s-days. In this study, it was assumed that the 100-year snow accumulation condition would cause the 100-year snowmelt runoff volume, which was determined to be 8170 m^3/s-days. The Case 1 spring PMF was therefore computed to have a total volume of 9608 m^3/s-days.

The Case 2 spring PMF was computed using a 100-year 24-hour rainstorm combined with the Probable Maximum Snow Accumulation (PMSA). The 100-year 24-hour rainfall total was obtained from the results of the rainfall frequency analysis at Topley Landing, Smithers Airport and Quick. The 100-year 24-hour rainfall depth was determined to be 62 mm and its volume was determined to be 998 m^3/s-days. It was assumed in this study that the PMSA condition would cause the 10000-year snowmelt runoff volume, which was determined to be 17,090 m^3/s-days. The Case 2 spring PMF was estimated to have a total volume of 18,088 m^3/s-days, which is the governing case with a greater value when compared to the Case 1 spring PMF.

The spring IDF was considered to be 1/3 of the way from the 1000-year snowmelt runoff volume to the Case 2 spring PMF volume, or 13,996 m^3/s-days. This April 1 to June 30 snowmelt runoff volume was distributed over a 65-day period (April 15 to June 30) based on the 2002 inflow hydrograph, which had the highest recorded peak flow and the greatest recorded snowmelt runoff volume over the entire period of record. The resulting peak inflow during the spring IDF was determined to be 621 m^3/s. Figure 8 shows the spring IDF hydrograph.

6.0 HYDRAULIC ANALYSIS

6.1 RESERVOIR OPERATION

The Fulton River Project comprises a dam with a free overflow spillway (overflow section), a bottom outlet, and a flow regulating system including an intake which provides regulated flow to the downstream Spawning Channel No. 2 via a supply tunnel and pipeline and a regulating tunnel which leads to a valve house containing three hollow cone valves.

The free overflow spillway is the main flood discharge outlet for the project. This overflow section of the dam has a crest length of 32 m (104 feet) and a crest elevation of 776.33 m (2547 ft). The rating curve for the spillway is shown on Figure 9 (Patrick Fawkes, 1988).

The bottom outlet has an upper gate and a lower gate both discharging into a common spillway channel. The upper gate (2.2 m by 3.9 m or 7 feet 4 inches by 12 feet 9 inches) with sill elevation of 771.98 m (2532.75 ft) is intended for supply releases to the river while the lower gate (2.2 m by 2.3 m or 7 feet 4 inches by 7 feet 6 inches) with sill elevation of 765.81 m (2512.5 ft) is intended for flood passage during the spring flood. The gates are vertical lift fixed-wheel design fabricated in welded steel. The lower gate should not be used when the reservoir elevation is at or below 769.62 m (2525 feet). The upstream construction cofferdam with a crest elevation of 768.71 m (2522 feet) has been left in place to enable the upstream portions of the outlet and the dam to be inspected at low reservoir elevations. Use of the dam outlet at reservoir elevations below 769.62 m (2525 feet) could erode the crest of the cofferdam. The rating curves for the two gates are provided on Figure 10 (Patrick Fawkes, 1988).

Flow releases to the Fulton River can also be made through the intake, regulating tunnel and three hollow cone valves (twin 2.1-m or 84-inch diameter and one 0.8-m or 30-inch diameter located below the twin valves). The operating mechanisms for the valves are housed in a concrete structure founded on the mass concrete surround to the valves.

The purpose of the Fulton River Project is to regulate the flow of the Fulton River to supply flows more suitable for spawning in the downstream artificial salmon spawning channels. Flows are maintained above a specified minimum limit, and flood flows are prevented from exceeding specified maximum limits. Rule curves providing guidance on the operation of the flow control equipment such as spillway, gates and valves have been developed and contained in the operational manual to ensure that the project will provide these regulated flows and flood relief. Rule curves representing the capacity of the regulating tunnel and the overflow spillway (Case 1) and representing the combined capacity of the regulating tunnel, the overflow spillway and dam outlet works (Case 2) for large forecasted inflow volumes are shown on Figure 11. It should be noted that when lake levels exceed the spillway crest elevation, the regulating tunnel and outlet works were assumed to be gradually closed. As a conservative approach, it was assumed that at high tailwater levels, only the overflow spillway would be fully operational.

Fulton Lake has a live storage of 93,745,000 m^3 (76,000 acre-feet or 1085 m^3/s-days) between elevations 766.57 m (2515 feet) and 776.33 m (2547 feet). The maximum and minimum operating elevations are 779.98 m (2559 feet) and 766.57 m (2515 feet), respectively. The lake storage-elevation relationship is shown on Figure 12.

6.2 FLOOD ROUTING

The flood routing was done using the HEC-HMS model, which includes a routing component for flows through reservoirs. The starting water surface elevation was assumed to be the mean annual maximum lake level (776.95 m), during the summer-autumn IDF; and, the mean annual minimum lake level (770.40 m), during the spring IDF. Both Case 1 and Case 2 rule curves, Figure 11, were considered in the analysis. The results of the HEC-HMS flood routing are summarized in Table 8.

TABLE 8: RESULTS OF FLOOD ROUTING

Outflow	Peak Lake Level		Peak Inflow		Peak Outflow		Available Freeboard	
	(m)	(ft)	(m^3/s)	(cfs)	(m^3/s)	(cfs)	(m)	(ft)
Summer-Autumn IDF								
Case 1	780.12	2559.4	1146	40,474	519	18,341	1.39	4.55
Case 2	779.95	2558.9	1146	40,474	482	17,015	1.56	5.11
Spring IDF								
Case 1	780.26	2559.9	621	21,939	553	19,544	1.25	4.09
Case 2	780.16	2559.6	621	21,939	531	18,740	1.35	4.42

Freeboard was computed as the difference between the peak lake level and the dam crest elevation of 781.51 m (2564 ft). In all scenarios, a freeboard of at least 1.25 m would be available. During the summer-autumn IDF, a freeboard of 1.39 m would be available when the regulating tunnel and the overflow spillway are operated (Case 1) and a freeboard of 1.56 m would be available when the dam outlet works are operated together with the regulating tunnel and the overflow spillway (Case 2). During the spring IDF, a freeboard of 1.25 m would be available when the Case 1 rule curve is applied and a freeboard of 1.35 m would be available when the Case 2 rule curve is applied.

It should be noted that the above freeboard calculations have not included wind and wave effects. However, it was determined that the effective fetch length upstream of the dam would be limited and the waves generated are expected to be minor. Therefore, a freeboard of 1.35 m during the spring IDF when all outlets are operated is considered to be adequate.

One additional location to be considered is the intake structure approximately 500 m north of the dam. At this location, the reservoir has a longer fetch distance and wind/wave interactions would result in higher waves. However, available topographic information indicates that this area is bounded by high ground elevations and a freeboard of greater than 3 m would be available. Therefore, flooding concerns would be limited when wind and wave effects are considered.

Figures 13 to 16 present the results of the flood routing graphically.

6.3 ENERGY DISSIPATOR

A flip bucket is used as the terminal structure for the overflow spillway at Fulton Dam. Usually flip buckets are used where foundations are relatively non-erodible such as in this case. The objective of a flip bucket is to throw the jet into a pool as far downstream as possible so that scour does not endanger the dam, spillway, or other ancillary works. Based on drawings provided by DFO, the flip bucket structure in question has an invert elevation of 766.57 m (2515 ft) and a lip elevation of 767.39 m (2517.68 ft). The angle of the bucket lip is approximately 35 degrees.

As recommended by the CDA guidelines, flip buckets must always be situated above the maximum tailwater level to prevent damage to the lip due to turbulent effects. The normal practice is to select a bucket outlet angle of about 15 to 35 degrees with the lip set above the tailwater level for the maximum design flood. Based on observations made during the site visit, a fall with a vertical drop of more than 10 m (30 ft) is located approximately 17 m or 56 ft downstream of the flip bucket structure. Therefore, the horizontal throw distance from the bucket lip was first computed to check if the jet would terminate at a point that is downstream of the fall.

The throw distance calculations were performed in accordance with the method suggested by Hydraulic Design Criteria Handbook published by the US Army Corps of Engineers. The horizontal throw distance, X, and the vertical drop from the bucket lip to tailwater, Y, are expressed in terms of the jet velocity head, H_v. Throw distance curves for lip angles of 0 to 45 degrees were developed based on the following theoretical equation:

$$X/H_v = \sin 2\theta + 2\cos\theta(\sin^2\theta + Y/H_v)^{0.5}$$

Where: X = throw distance, m or ft

Y = vertical drop from lip to tailwater surface, m or ft

H_v = velocity head of jet at bucket lip, m or ft

θ = bucket lip angle, degree

The maximum water level during the Inflow Design Flood (IDF) was determined to be 780.17 m (2559.6 ft). The velocity head (11.7 m or 38.3 ft) was considered to be the difference between the reservoir elevation, halfway between the maximum water level and the spillway crest, and the invert elevation of the flip bucket structure. Finally, a tailwater level assumed to be 765.66 m (2512 ft) was used to determine the minimum vertical drop from the bucket lip to the tailwater level (1.73 m or 5.68 ft). By using Hydraulic Design Chart 112-8, the horizontal throw distance from the bucket lip was determined to be approximately 23.4 m or 76.6 ft.

Based on the estimated throw distance, the bucket jet would land beyond the falls during the IDF. Therefore, the bucket lip would be much higher than the tailwater level, and this would meet the requirement suggested by the CDA guidelines.

6.4 RESERVOIR DRAWDOWN

An analysis was conducted to determine the amount of time required to drain the reservoir through the outlet works to a low operating level. The hydrological model developed for freeboard determination was used as a basis for this analysis. It was assumed that a constant inflow equivalent to the mean monthly flow during the summer months (20.5 m^3/s) would occur throughout the reservoir drawdown period. The initial water level was set at the spillway crest elevation, 776.33 m. The two outflow curves used in the freeboard determination were both applied in this analysis for comparison purposes. Results of the hydrological model are summarized in Table 9 and are shown on Figure 17.

TABLE 9: RESULTS OF RESERVOIR DRAWDOWN ANALYSIS

Scenario	Condition	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Approx. Time Required to Drain Reservoir (days)
1	Constant Inflow 20.5 m^3/s Outflow Curve 1	20.5	96.8	29
2	Constant Inflow 20.5 m^3/s Outflow Curve 2	20.5	151.8	24

7.0 DAM BREAK ANALYSIS

7.1 PREVIOUS STUDY

A dam breach inundation study for Fulton Dam was conducted by Patrick Fawkes & Associates in 1986. The development of an assumed breach in the Fulton Lake Dam combined with the peak outflow from the simultaneous occurrence of the Probable Maximum Flood (PMF) was simulated using the Simplified Dam Break (SMPDBK) model developed by the US National Weather Service. An additional check of the results obtained from the SMPDBK model was performed using the US Army Corps of Engineers program HEC-1. Dam breach parameters were derived by assuming that two of the four blocks comprising the spillway section of the dam slide downstream, leaving a gap between the low level outlet block and the two blocks on the right of the dam. This gap was assumed to extend down to the foundation rock at about elevation 764.74 m (2509 ft). The effective breach width was determined to be 12.8 m (42 ft). It was also assumed that the dam breach would be completed in 4 minutes. To reflect the backwater effect produced by the reduction in channel cross-sectional area at the bridge and the control of Babine Lake, a backwater study was carried out. The peak water elevations at each section derived from the combination of the SMPDBK program and the backwater study were reported and used to estimate the extent of inundation.

7.2 DAM BREACH MODEL

In accordance with the 2007 CDA Guidelines, an updated dam break analysis, including characterization of a hypothetical dam breach, flood wave routing, and inundation mapping, was carried out.

A flood-induced dam failure scenario was considered in the analysis. Flood wave routing was carried out using SMPDBK. Input data requirements for the dam break scenario include parameters defining the geometry of the dam, reservoir and downstream valley, and timing of the breach. Certain dam breach parameters reported in the previous inundation study such as the final breach elevation, final breach width and breach development time were considered to be still valid and were applied in the model. The remaining dam breach parameters were obtained based on the conditions of the reservoir during the spring IDF. A summary of the overall dam breach parameters are provided in Table 10.

TABLE 10: SUMMARY OF DAM BREACH INPUT		
Type of Dam	Concrete Gravity Dam	
Dam Breach Elevation (DBE)	780.17 m	(2559.6 ft)
Final Breach Elevation	764.74 m	(2509.0 ft)
Volume of Reservoir at DBE	63,610,000 m^3	(51,570 acre-ft)
Surface Area of Reservoir at DBE	13,227,500 m^2	(3269 acres)
Final Breach Width	12.8 m	(42.0 ft)
Non-Breach Flow	530.65 m^3/s	(18,740 cfs)
Breach Development Time	4 min	

A total reach length of approximately 6 km was modelled extending from just downstream of the dam to Babine Lake. Since no digital mapping information was available at the beginning of the study, available maps (DFO Dwg. No. 21-19-402 and 983-15-10) were digitized for cross section extraction and floodplain mapping. Using the digitized mapping information, eight cross-sections were extracted along the reach downstream of the dam. Where necessary, minor modifications of the cross sections were made based on channel invert information reported in the previous inundation study. The downstream channel roughness coefficient was determined based on a reasonable combination of in-bank and out-of-bank Manning's n values.

The parameters were input to the SMPDBK model to determine the preliminary flood levels and timing in the receiving channel. The peak outflow from the dam breach and the time to reach the peak elevation at each channel cross section were calculated.

As mentioned for the previous study by Fawkes, the maximum water elevations calculated in SMPDBK did not consider backwater effects due to the highway bridge crossing near Spawning Channel No. 2, or similar effects due to the control at Babine Lake. A backwater calculation was therefore carried out using HEC-RAS 3.1.3, a river modelling system developed by the US Army Corps of Engineers, to refine the maximum water elevations. Highway bridge information was obtained from DFO Dwg. No. 21-19-P75A and estimated from field photos. A typical high lake level of 711.16 m (2333.2 ft) at Babine Lake, as determined in the previous inundation study, was considered to be still valid and applied as the starting water surface in the model. As a simple and conservative approach, the peak outflow estimated in the SMPDBK model at each cross section was used in determining the peak backwater elevation at that particular cross section, assuming that the river flow is constant throughout the downstream reach.

Results of the dam break analysis, including that of the backwater calculations, are presented in Table 11.

TABLE 11: RESULTS OF DAM BREAK ANALYSIS

Cross Section	Distance from Dam (km)	Max Flow (m^3/s)	Approx. Channel Invert Elev. (m)	Max Water Elev. (m)	Max Water Depth (m)	Time to Reach Max Water Elev. (min)
XS 1	0.00	1857	747.1	755.0	7.9	0.0
XS 2	1.37	1820	720.2	725.4	5.2	2.4
XS 3	2.86	1753	712.6	719.2	6.6	8.4
XS 4	3.88	1713	709.0	717.1	8.1	12.6
XS 5	4.14	1695	708.4	716.8	8.4	15.6
XS 6	4.81	1678	708.1	715.4	7.3	19.8
XS 7	5.49	1662	707.4	712.4	5.0	21.6
XS 8	5.84	1645	701.0	711.2	10.2	21.6

It should be noted that the peak flow as a result of the dam breach during the spring IDF varies from 1857 m^3/s at the dam to 1645 m^3/s at the lake. There is an attenuation of about 11% within this 6 km long study reach. In addition, the time to reach the peak water elevation near the outlet of the river is approximately 22 minutes. Since a different flood condition was considered compared to that in the previous inundation study, the percentages of flow attenuation as well as the time required to reach peak flow are not comparable.

Finally, peak flows obtained in the previous study were applied in the current HEC-RAS model for comparison purposes. Results of this scenario provided maximum water elevations similar to those in the previous study with the maximum difference in water levels corresponding to cross sections 2 and 6. This is likely due to the differences in model assumptions for local channel geometry and channel roughness.

7.3 INUNDATION MAPPING

Based on available topographic information, an inundation map was prepared for the modelled dam breach scenario (Figure 18). Flood levels were interpolated from the resulting flood profile and used to draw flood level isograms at 2 m intervals in the upper reach (XS 1 to XS 3) and at 1 m intervals in the lower reach (XS 3 to XS 8).

The downstream points of interest were determined to be Spawning Channels No. 1 and No. 2 (approximately 2 km to 4 km downstream of the dam), the highway bridge (Provincial Highway 118, approximately 5 km downstream of the dam), and the DFO facility buildings (approximately 5 km downstream of the dam). Potential flood depths at these locations above the average ground elevations in the case of a dam breach scenario are summarized in Table 12. The approximate times required to reach the peak flood depths at these locations after the completion of the breach are also presented in the same table.

TABLE 12: DAM BREAK ANALYSIS RESULTS AT POINTS OF INTEREST				
Location	Approx. Max Water Elevation (m)	Average Ground Elevation (m)	Potential Flood Depth (m)	Approx. Time to Reach Peak (min)
Spawning Channel No. 1	721	718	3	5
Spawning Channel No. 2	718 to 717	716 to 714	2 to 3	10
Highway Bridge Crossing	716.8 (at XS 5)	715.1 (Top of Deck Elevation)	1.7 (Overtopping)	15
DFO Facility Buildings	716.5	718	-	-
Indian Settlement	711.3	N/A	N/A	20

It should be noted that both spawning channels are partly located within the inundated area in the case of a dam breach combined with the spring IDF. The times required to reach the peak flood depths at these two locations would be about 5 minutes for Spawning Channel No. 1 and about 10 minutes for Spawning Channel No. 2 after the completion of the dam breach. The highway bridge crossing located at XS 5 has a deck elevation of 715.1 m (2346 ft), and overtopping by up to 1.7 m would occur in a dam breach scenario. The time required to reach the maximum overtopping height at the bridge would be approximately 15 minutes after the completion of the breach. Different from the results in the previous inundation study, the latest results of the dam break analysis indicate that the DFO facility buildings located southeast of the highway bridge crossing are just outside of the inundation limits. This indicates that there would be minor flooding concerns at this location. On the other hand, similar to the results in the previous study, the Indian settlement near the outlet of the river into Babine Lake would be inundated. However, no further information is available to confirm the average ground elevation in the vicinity of the settlement, and therefore it is difficult to assess the magnitude of flood depth at this location. The approximate time required to reach the peak water elevation near the Indian settlement would be 20 minutes.

8.0 CONCLUSIONS AND RECOMMENDATIONS

- A hydrotechnical assessment has been conducted as part of the Fulton Dam Safety Review. The study involved a hydrologic assessment to determine the approximate Inflow Design Flood (IDF), a flood routing and hydraulic analysis to assess the capacity of the dam to pass the IDF, and a dam break analysis to evaluate the impacts of a potential dam breach.

- Based on previous dam safety reviews, it is understood that Fulton Dam is currently classified as a High Consequence Dam. In accordance with the 2007 CDA Guidelines, the IDF for Fulton Dam was chosen to be 1/3 between an annual exceedance probability of 1 in 1000 and the Probable Maximum Flood (PMF). Two IDFs were developed: the summer-autumn IDF and spring IDF. The rainfall total used to determine the summer-autumn IDF was calculated to be 104 mm. The governing spring IDF was determined to have a total volume of 13,996 m^3/s-days and was distributed over a 65-day period (April 15 to June 30). The peak inflow during the summer-autumn IDF was determined to be 1146 m^3/s while the peak inflow during the spring IDF was determined to be 621 m^3/s.

- Flood routing was performed to assess the capacity of the dam to pass the summer-autumn IDF and the spring IDF. Two rule curves were applied: Case 1 corresponding to the capacity of the regulating tunnel and the overflow spillway; and, Case 2 corresponding to the combined capacity of the regulating tunnel, the overflow spillway and dam outlet works. In all scenarios, a freeboard of at least 1.25 m would be available. Due to the limited effective fetch length upstream of the dam, waves generated are expected to be minor. Therefore, a freeboard of 1.35 m during the spring IDF when all outlets are operated is considered to be adequate.
- The intake structure located approximately 500 m north of the dam has a relatively long fetch distance, likely resulting in higher waves. However, available topographic information indicates that this area is bounded by high ground elevations and a freeboard of greater than 3 m would be available. Consequently, there are no flooding concerns at this location.
- A flip bucket is used as the terminal structure for the overflow spillway at Fulton Dam. As recommended by the CDA guidelines, flip buckets must always be situated above the maximum tailwater level to prevent damage to the lip due to turbulent effects. To check the adequacy of the flip bucket, the throw distance was calculated. Results of the calculations indicate that the bucket jet would land beyond the falls during the spring IDF. Therefore, the bucket lip would be much higher than the tailwater level, which satisfies the requirement of the CDA guidelines.
- A reservoir drawdown analysis was performed to determine the amount of time required to drain the reservoir to a low operating level through the outlet works during an emergency. It was assumed that this would coincide with an average summer daily inflow of 20.5 m^3/s. Results of the analysis show that with the capacity of the regulating tunnel and the overflow spillway, it would take approximately 29 days to drain the entire reservoir. If all outlet works are operated, only 24 days would be required to drain the entire reservoir.
- An updated dam break analysis was carried out using SMPDBK. Dam break parameters were determined based on information provided in the 1986 inundation study report and based on the conditions of the reservoir during the spring IDF. Since no digital mapping information was available at the beginning of the study, available maps were digitized for cross section extraction and floodplain mapping. Subsequently, a backwater study was performed using HEC-RAS to take account of the backwater effects from the highway bridge crossing near Spawning Channel No. 2 and the control elevation at Babine Lake. The refined flood profile was used to prepare the flood inundation map.
- Spawning Channel No. 1 and Spawning Channel No. 2 would both be partly inundated in the case of a dam breach combined with the spring IDF. The highway bridge would be overtopped by a maximum of 1.7 m during a dam breach scenario. However,

the DFO facility buildings, located southeast of the highway bridge, are located just outside of the inundation limits, indicating that there would only be minor flooding concerns at this location. Similar to the results in the 1986 study, the Indian settlement would be inundated near the outlet of the river into Babine Lake. However, further topographic information is required to assess the magnitude of flood depths at this location.

- It should be noted that both the dam break analysis and the inundation mapping relied upon limited topographic information. No new survey information was available to refine local topography or channel geometry. It is recommended that additional surveys be conducted in the future to refine the extent of the inundation during a dam breach scenario and to confirm the magnitude of the flood depths at locations of interest.

9.0 LIMITATIONS OF REPORT

This report and its contents are intended for the sole use of the Department of Fisheries and Oceans and their agents. Hayco does not accept any responsibility for the accuracy of any of the data, the analysis or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than the Department of Fisheries and Oceans, or for any Project other than at the subject site. Any such unauthorized use of this report is at the sole risk of the user. Use of this report is subject to the terms and conditions stated in Hayco's Services Agreement and in the General Conditions provided in Appendix C of this report.

10.0 CLOSURE

We trust this report meets your present requirements. Should you have any questions or comments, please contact the undersigned at your convenience.

Hay & Company Consultants
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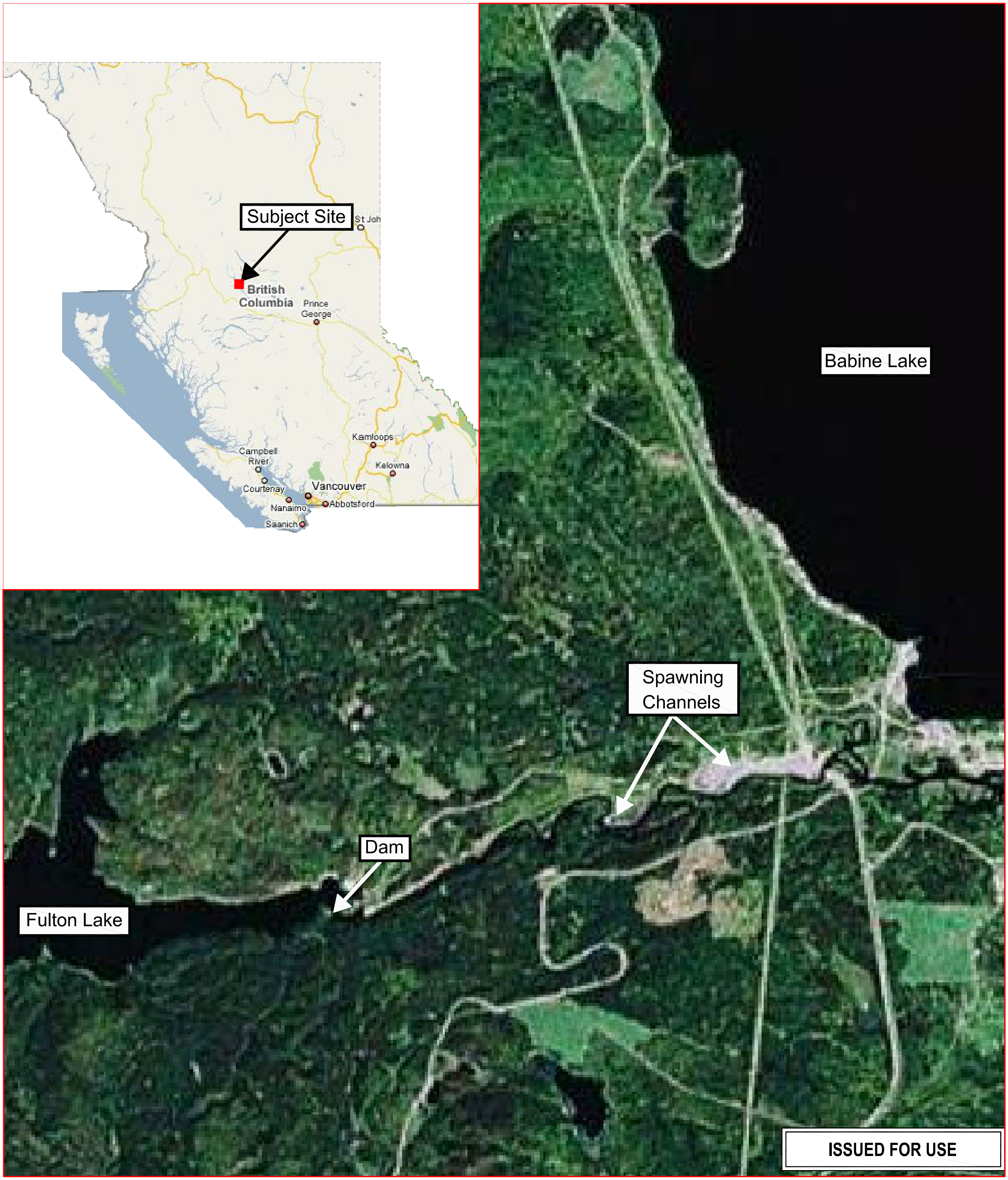
FIGURES



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

LOCATION MAP

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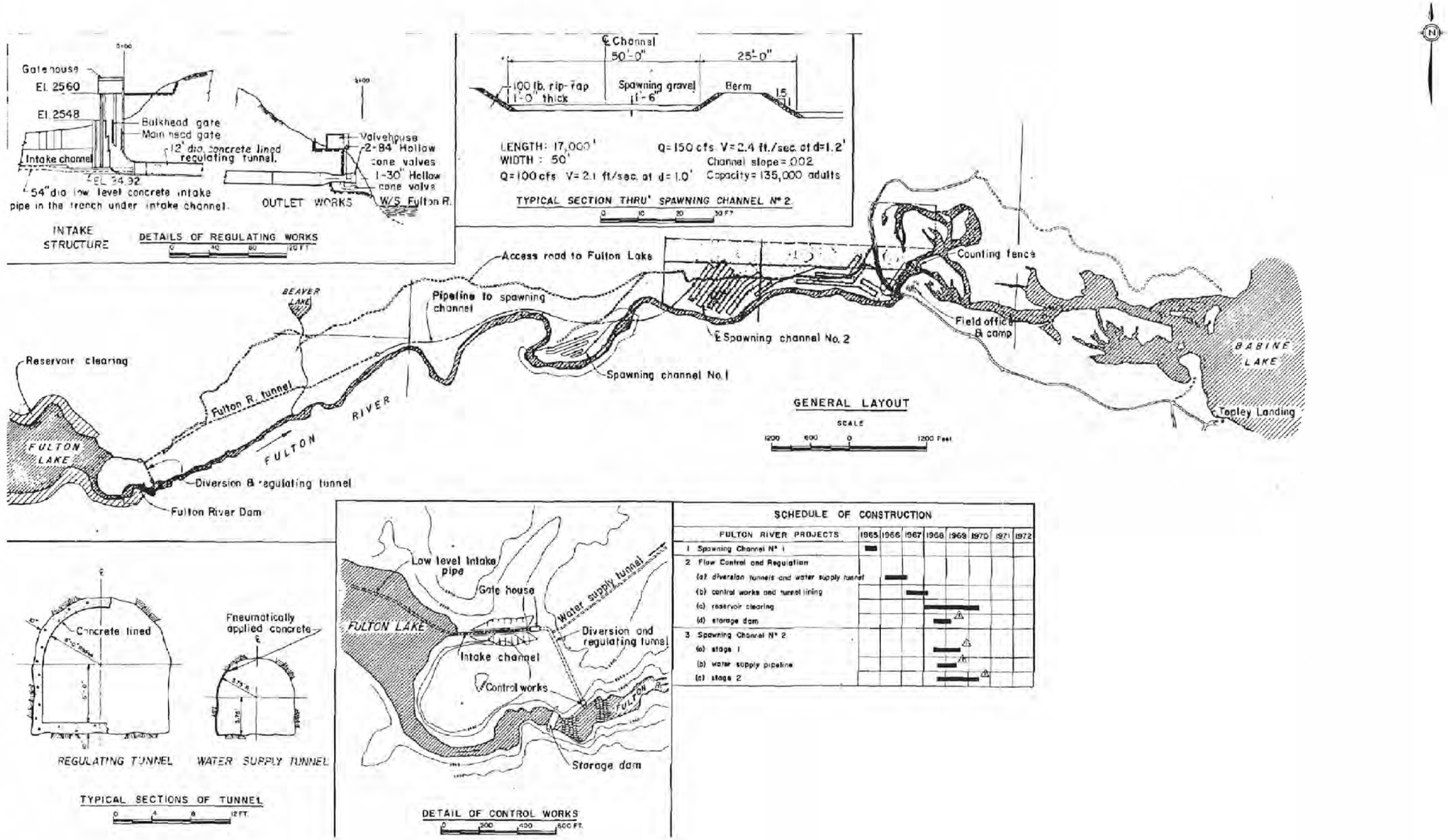
VANC	March 11, 2009

Figure 1



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NOTES

Base drawing obtained from Fulton Dam Safety Review Report by UMA Engineering Ltd. dated August 1997 (2507-0175-001-00-01)2007 based on DFO Drawing No. 21-19-G75R1 Babine Lake Development Fulton River General Layout

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	FULTON DAM SAFETY REVIEW 2008 HYDROTECHNICAL ASSESSMENT
	OVERALL PLAN OF THE FULTON RIVER PROJECT
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PROJECT NO.	DWN	CKD	REV	
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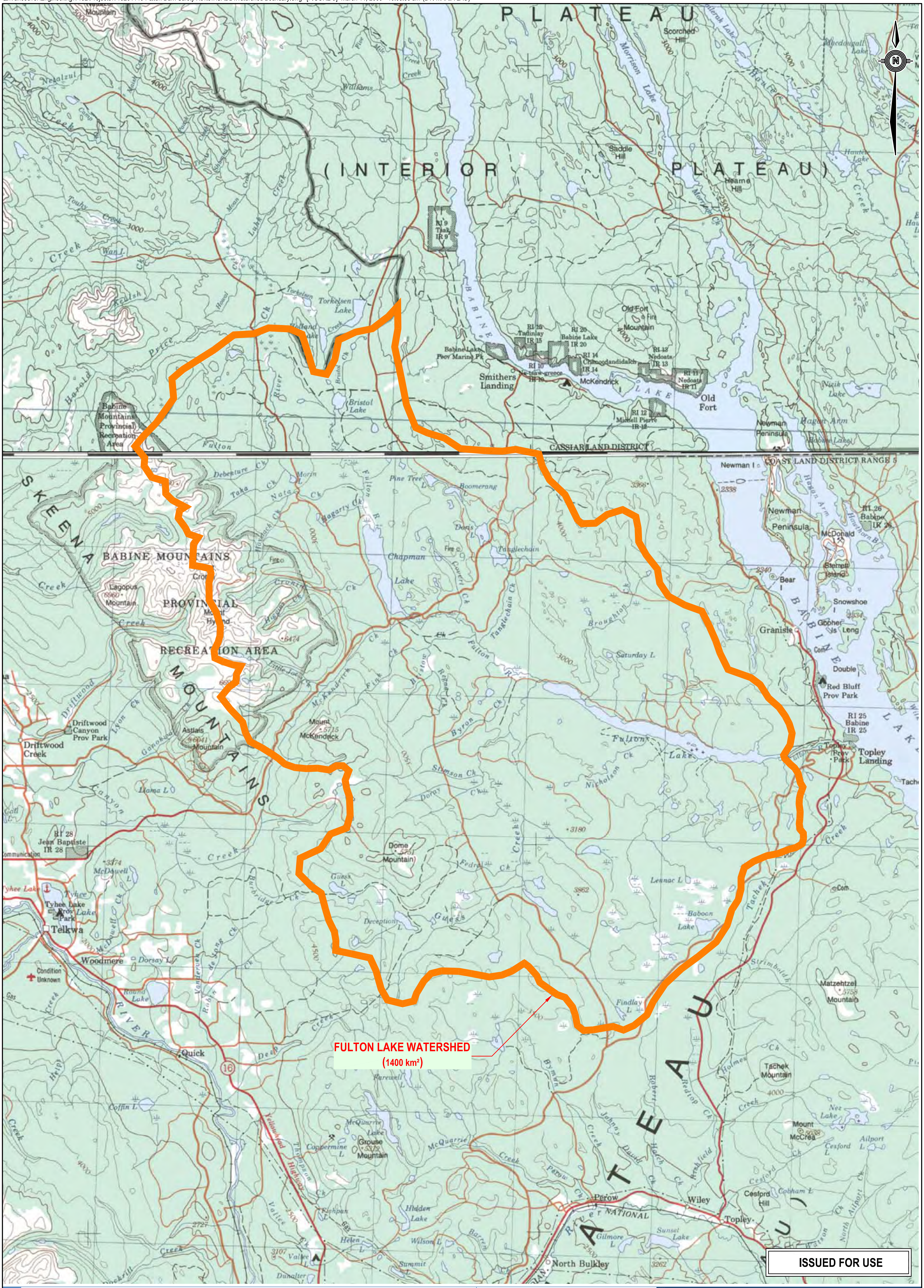

LEGEND

Watershed Boundary

SCALE 1:250,000

2000 0 2000 6000 10000m

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HYDROTECHNICAL ASSESSMENT

FULTON RIVER DAM
WATERSHED BOUNDARY

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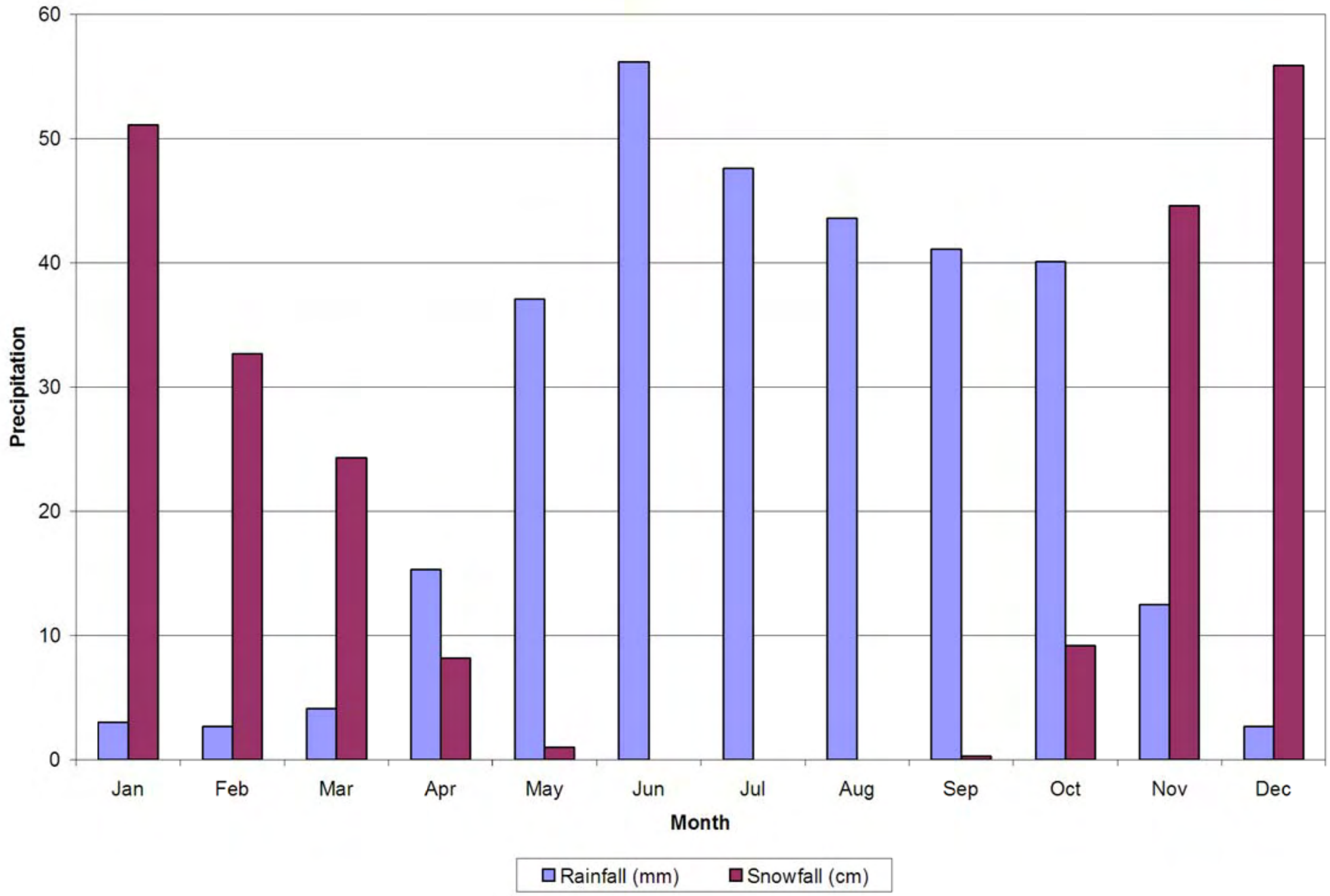

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MONTHLY PRECIPITAITON
AT TOPLEY LANDING

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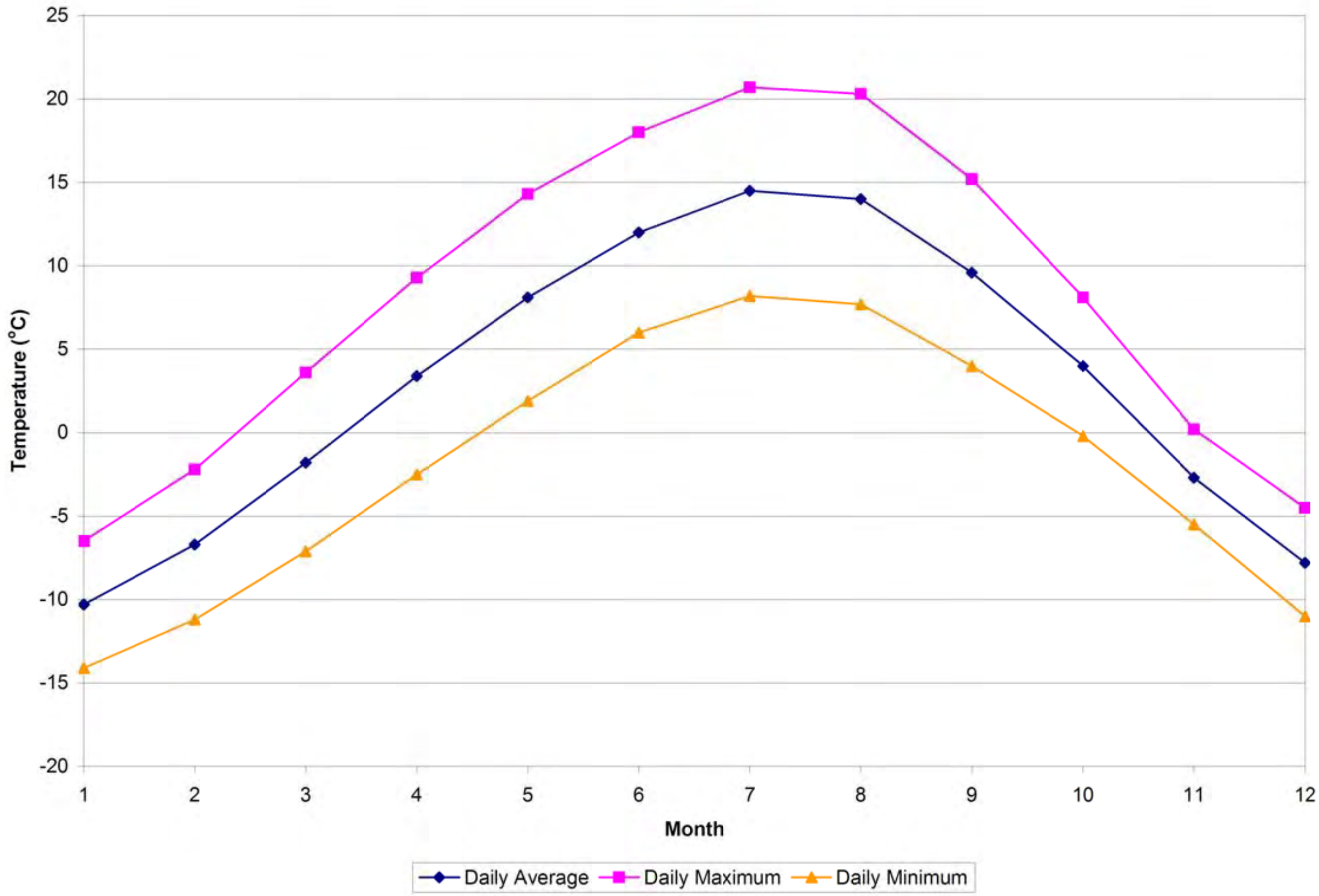

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MONTHLY TEMPERATURE
AT TOPLEY LANDING

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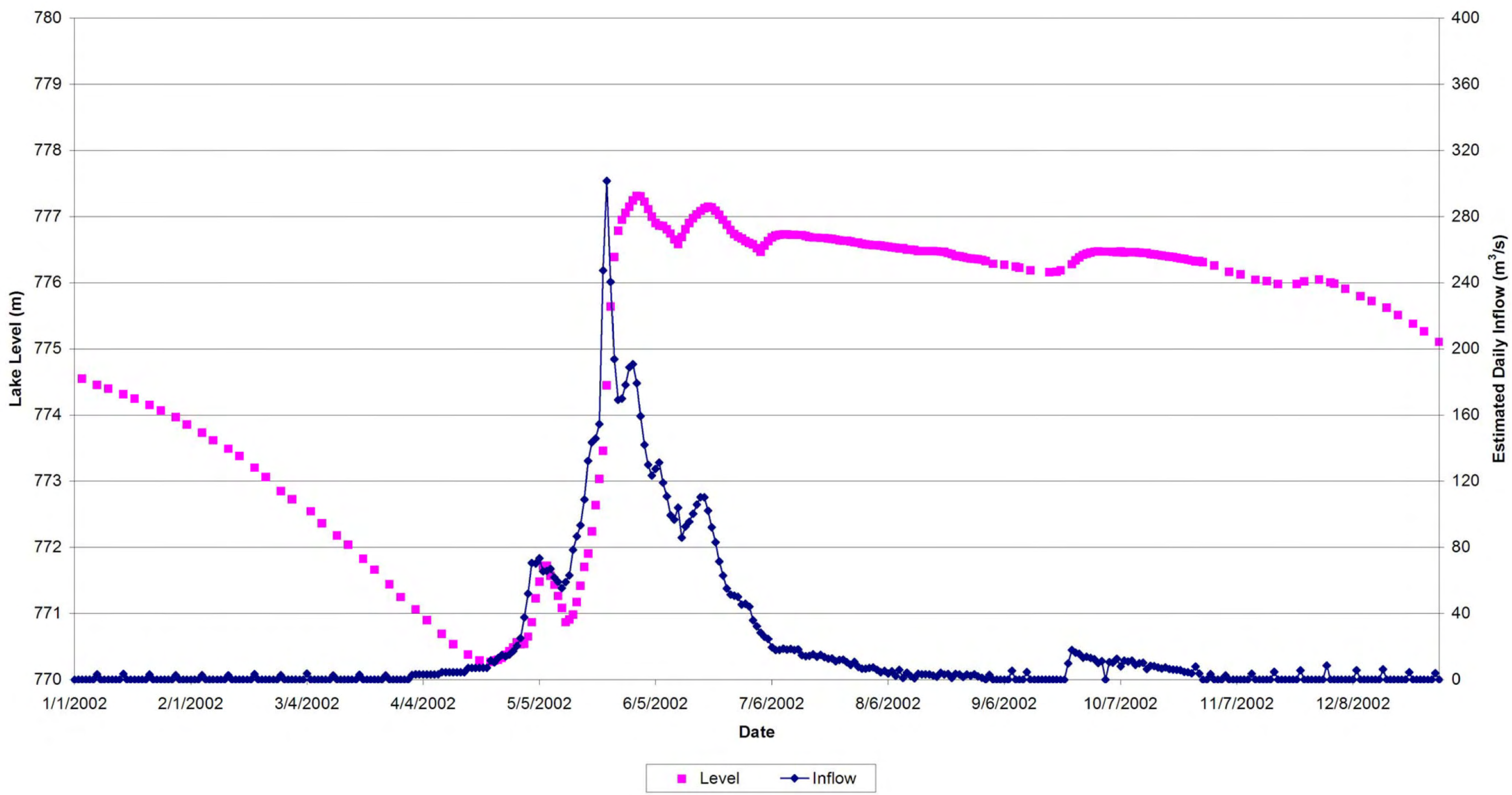
Figure 5

2002 Fulton Lake Levels and Inflows (Jan-Dec)



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Department of Fisheries and Oceans	FULTON DAM SAFETY REVIEW 2008 HYDROTECHNICAL ASSESSMENT			
	2002 FULTON LAKE INFLOW HYDROGRAPH AND LAKE ELEVATIONS			
HAYCO HAY & COMPANY CONSULTANTS A DIVISION OF EBA	PROJECT NO. V13201140	DWN ML	CKD RJW	REV 0
	OFFICE VANC	DATE February 16, 2009		Figure 6

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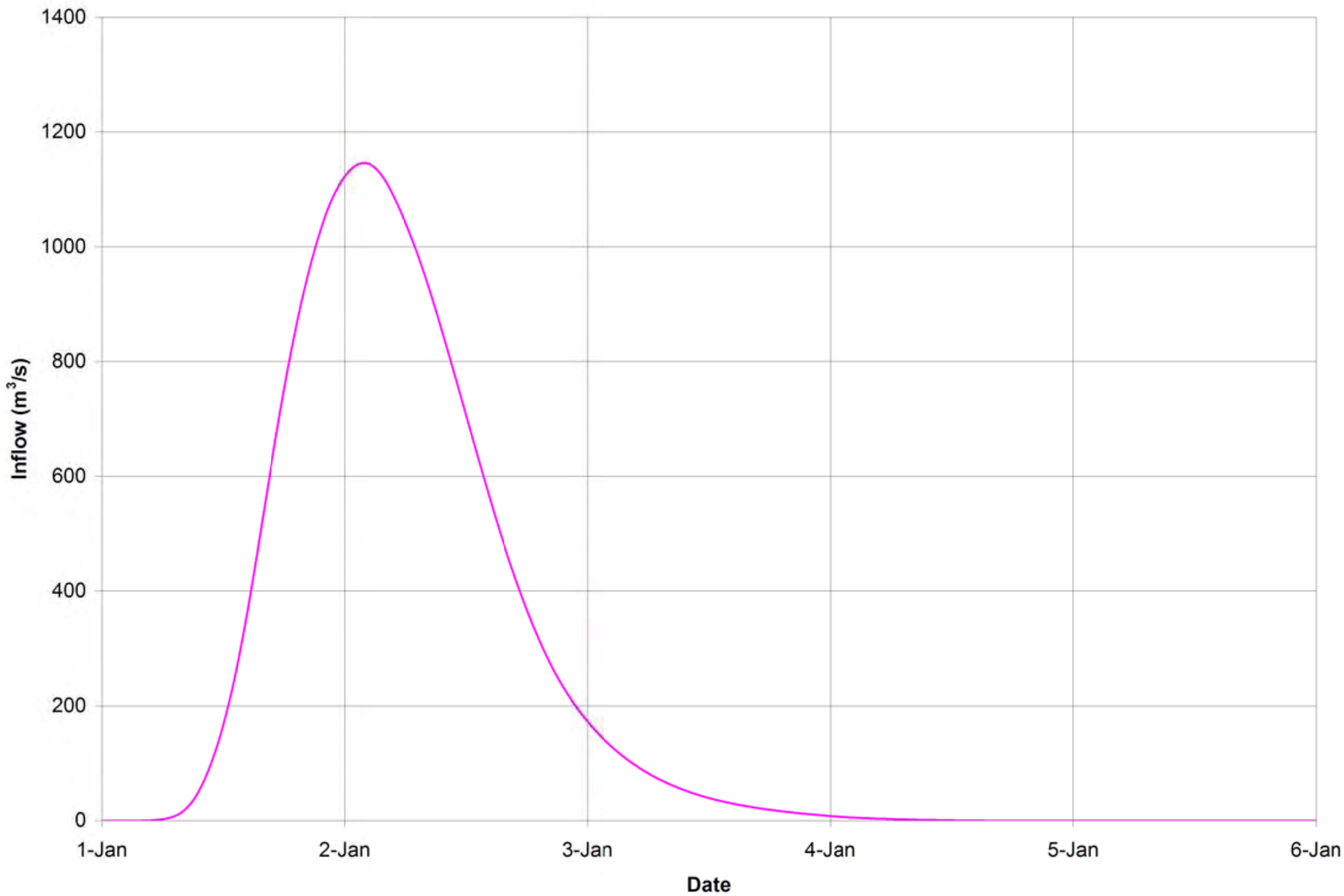

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SUMMER-AUTUMN IDF HYDROGRAPH

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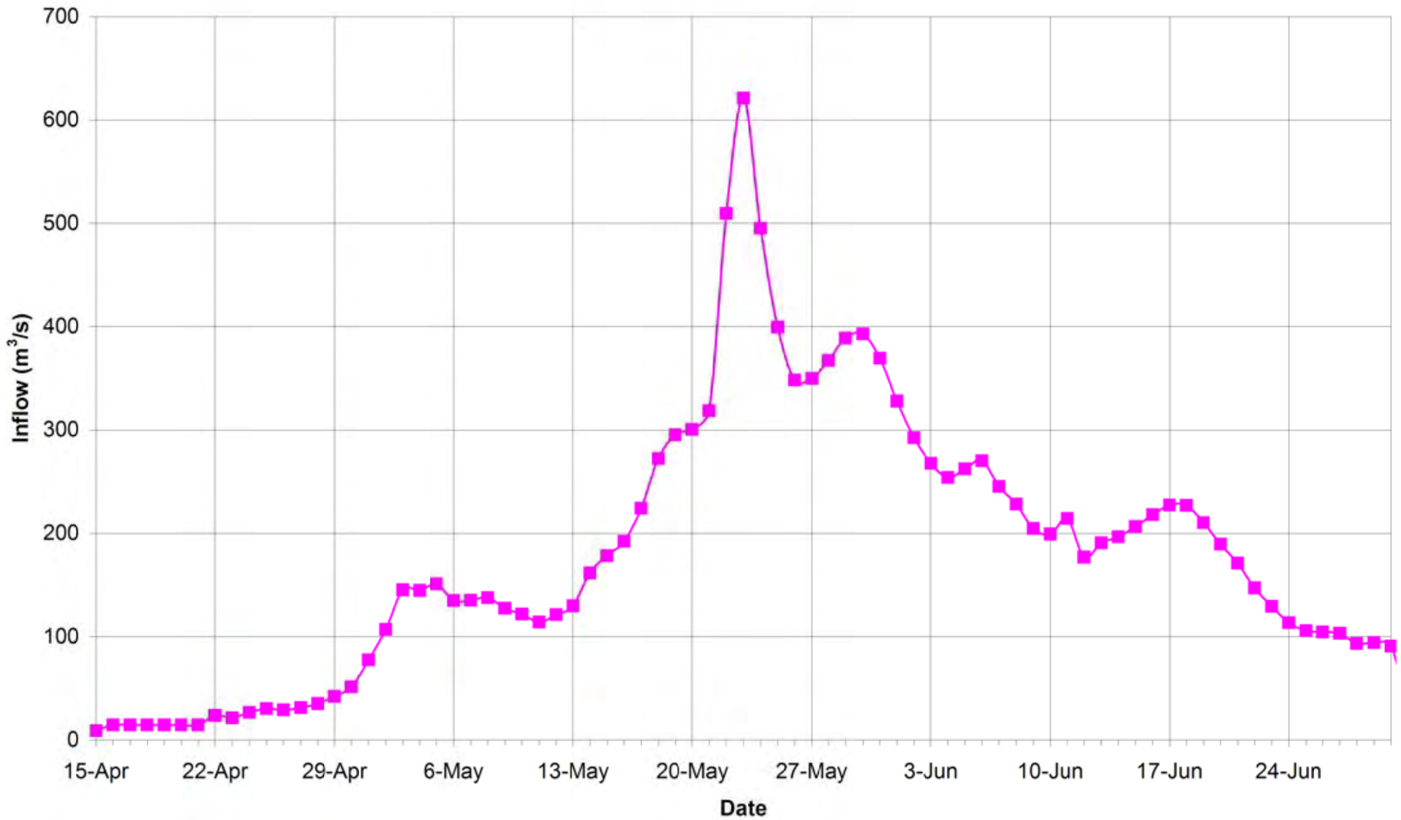
Figure 7



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2008 HYDROTECHNICAL ASSESSMENT

SPRING IDF HYDROGRAPH

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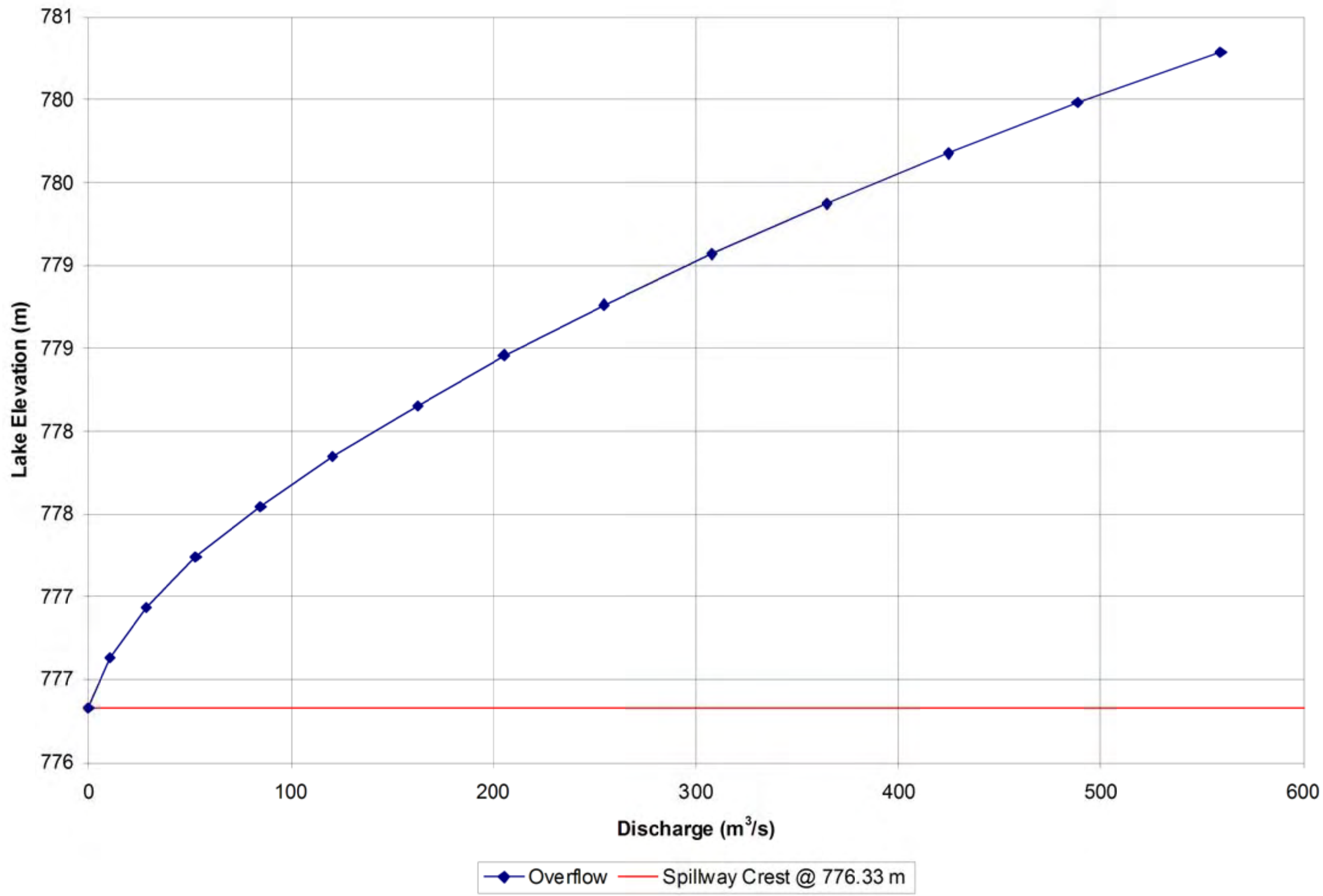
Figure 8



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

RATING CURVE FOR FREE OVERFLOW SPILLWAY

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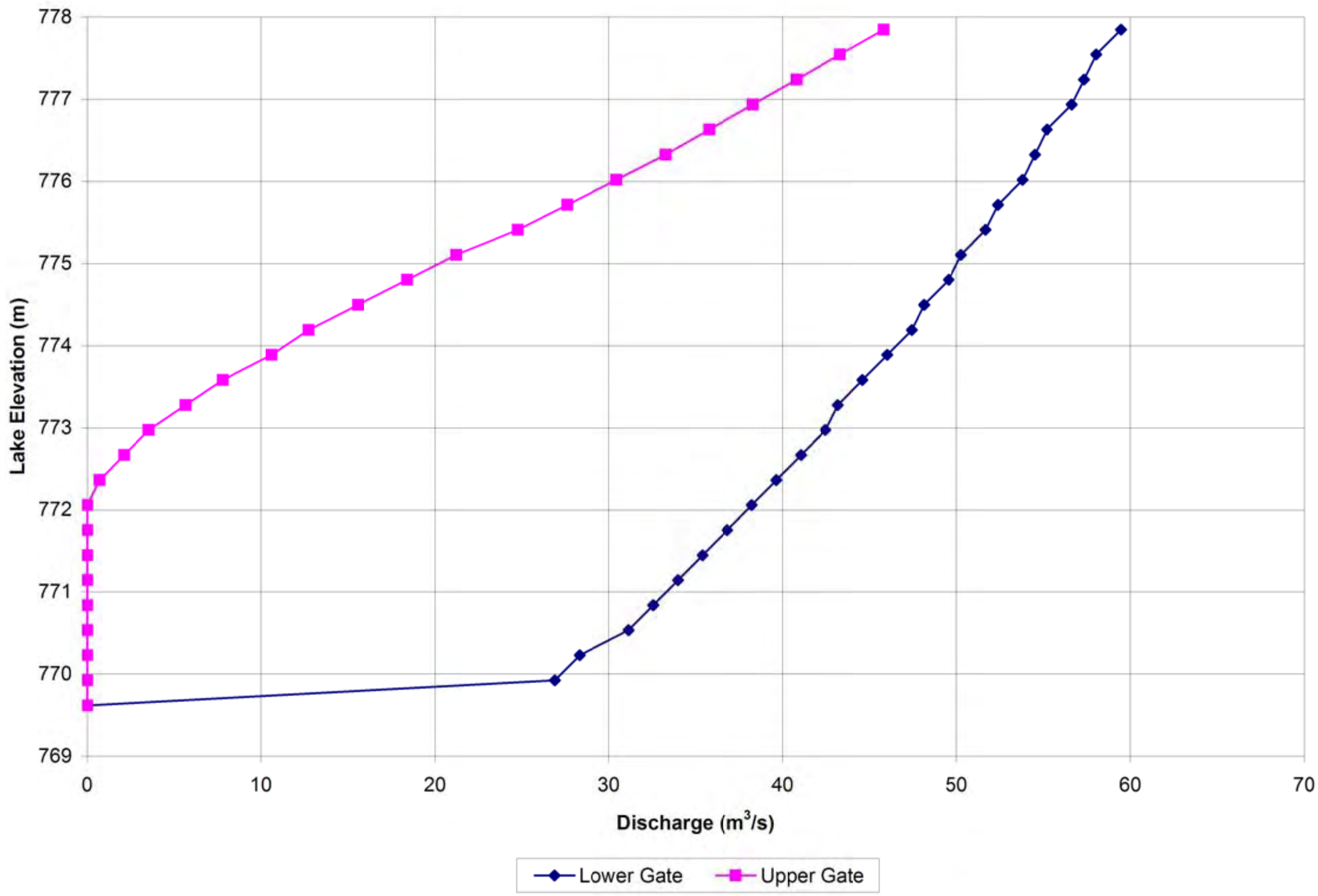
Figure 9



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

RATING CURVES FOR UPPER AND LOWER GATES

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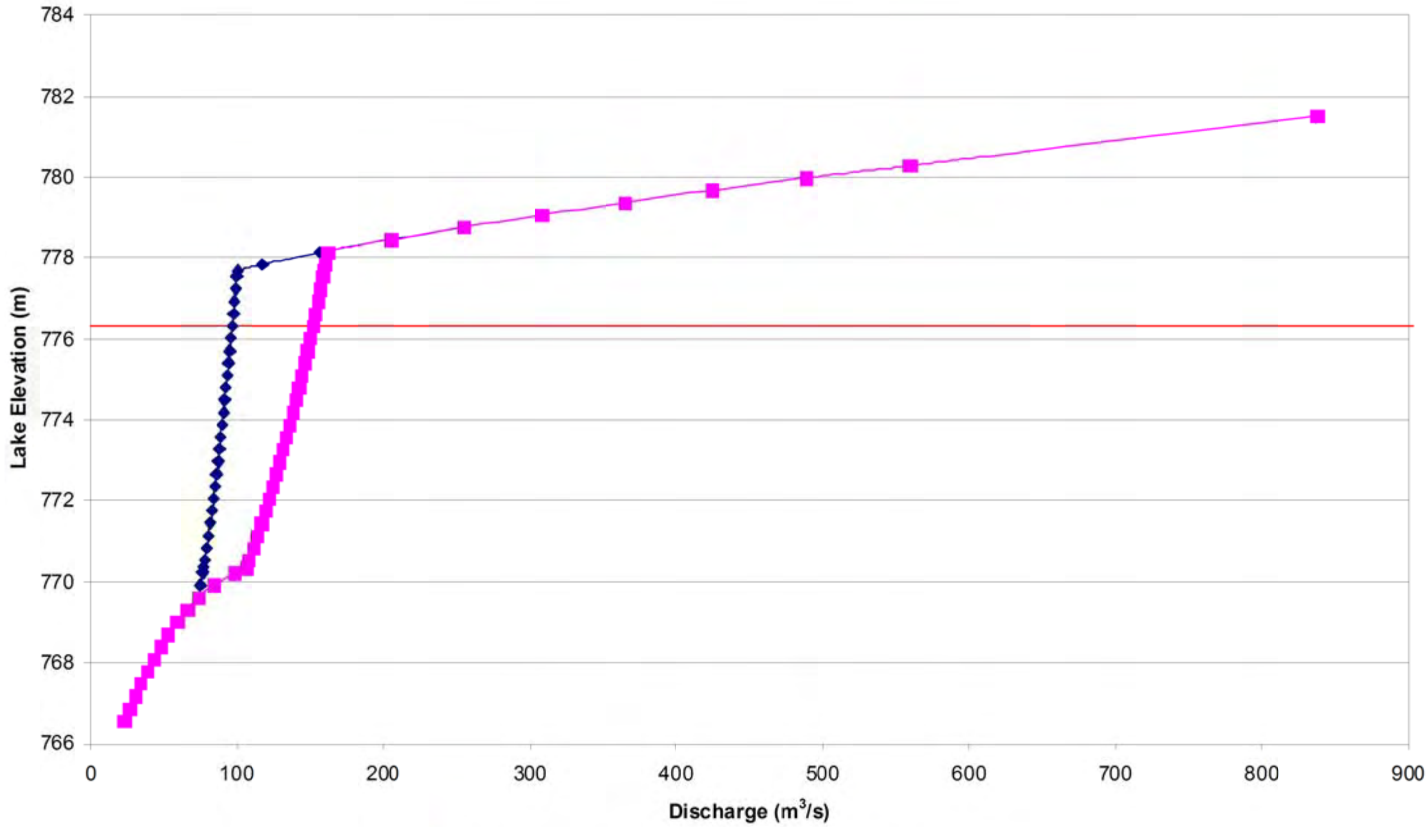
VANC	March 11, 2009		

Figure 10



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

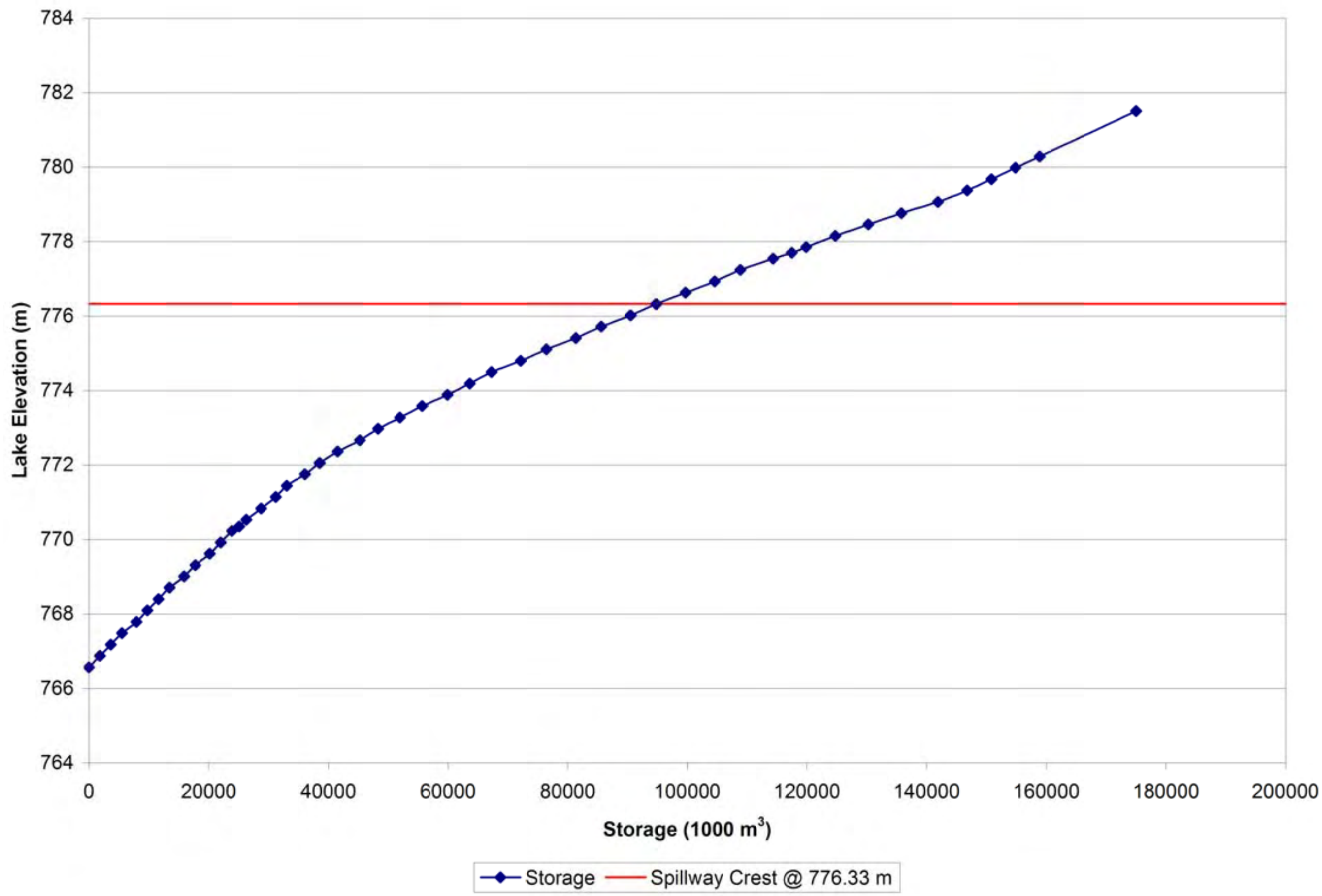
CASE 1 AND CASE 2
RULE CURVES

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0

OFFICE	DATE
VANC	March 11, 2009

Figure 11

Q:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Report Figures.dwg [FIGURE 12] March 11, 2009 - 1:11:47 pm (BY: MARIA LAU)



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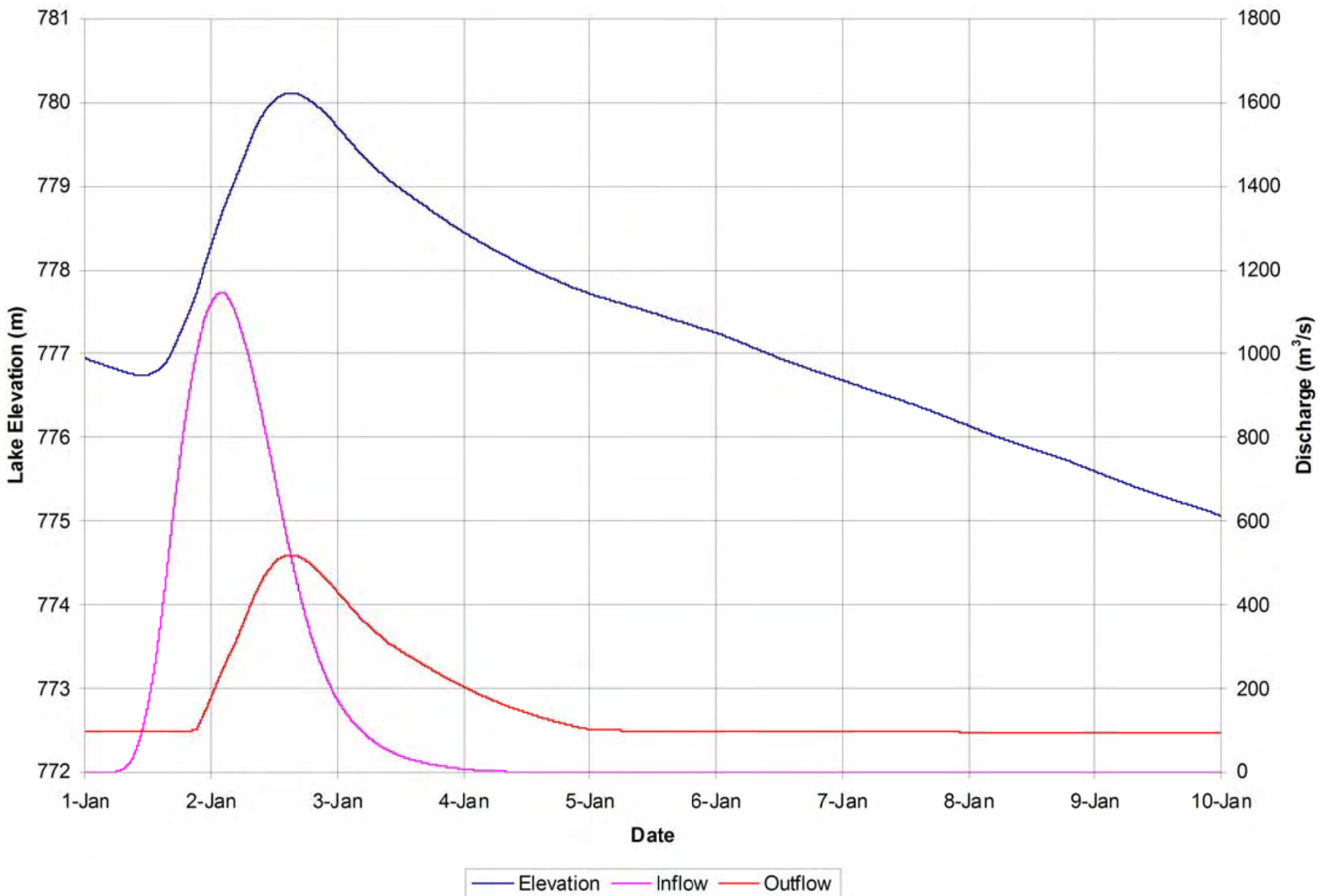
FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

LAKE STORAGE-ELEVATION RELATIONSHIP

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure 12



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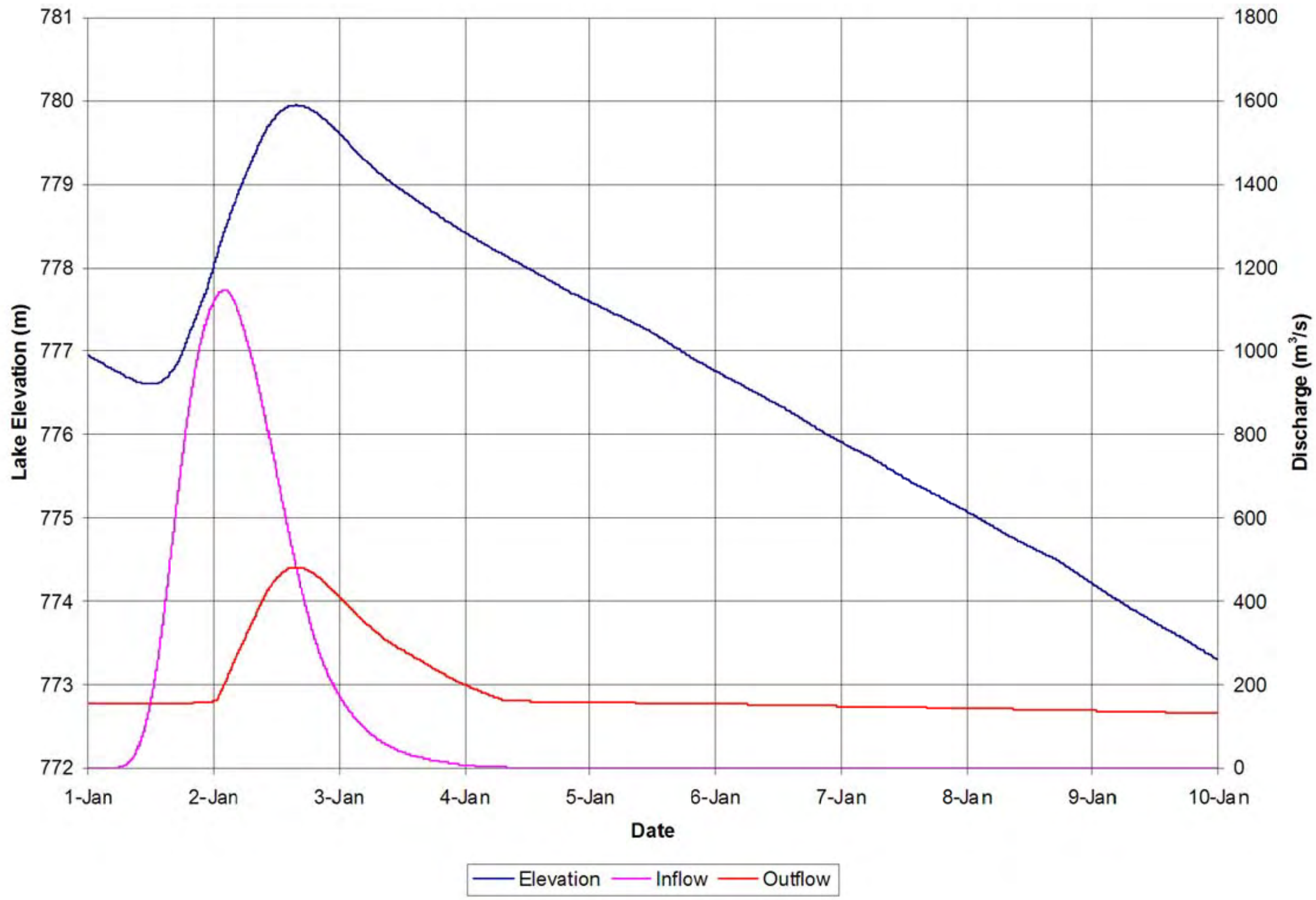
FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

RESULTS OF FLOOD ROUTING
SUMMER-AUTUMN IDF (CASE 1 RULE CURVE)

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure 13



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

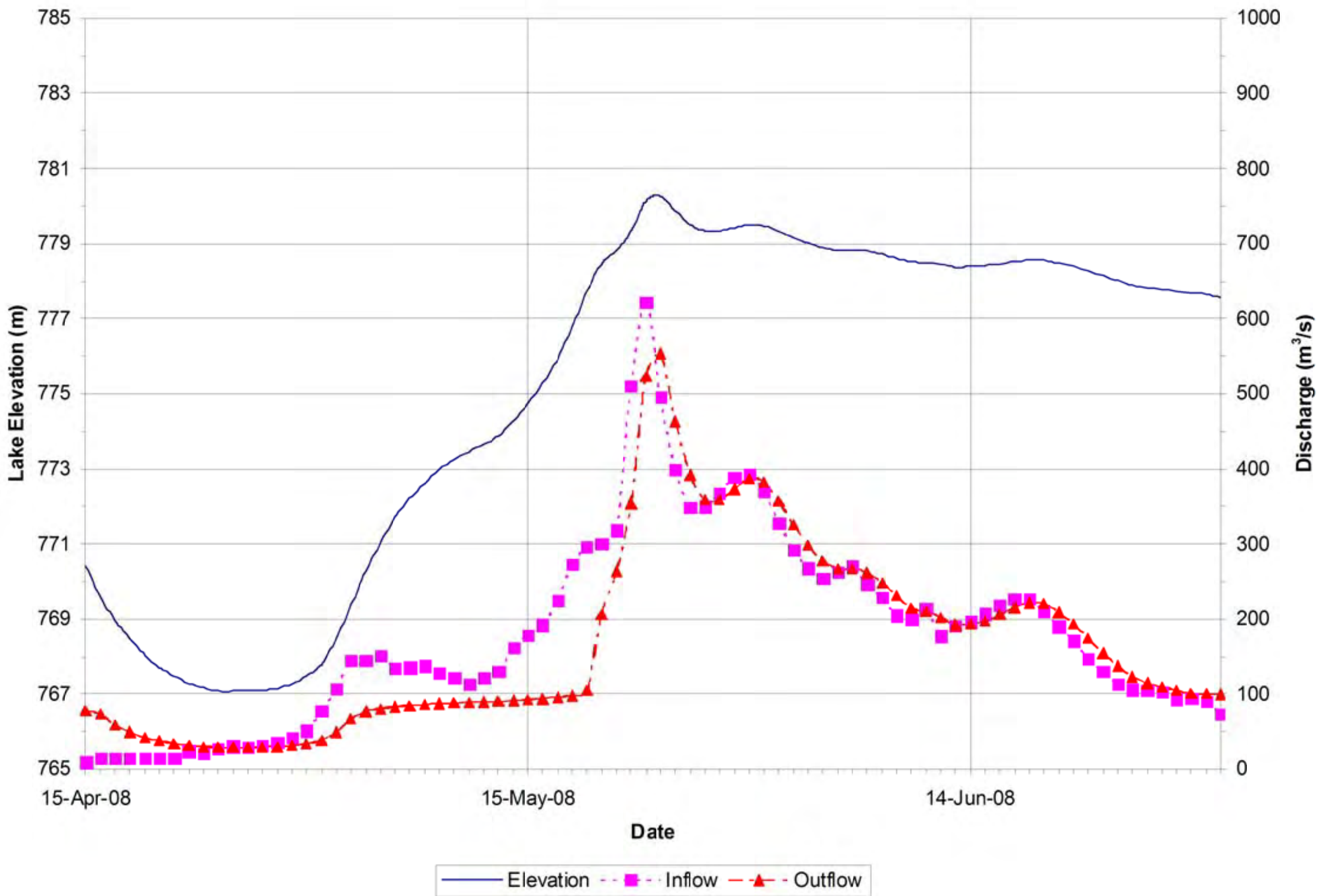
RESULTS OF FLOOD ROUTING
SUMMER-AUTUMN IDF (CASE 2 RULE CURVE)

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0

OFFICE	DATE
VANC	March 11, 2009

Figure 14

Q:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Report Figures.dwg [FIGURE 15] March 11, 2009 - 1:36:46 pm (BY: MARIA LAU)



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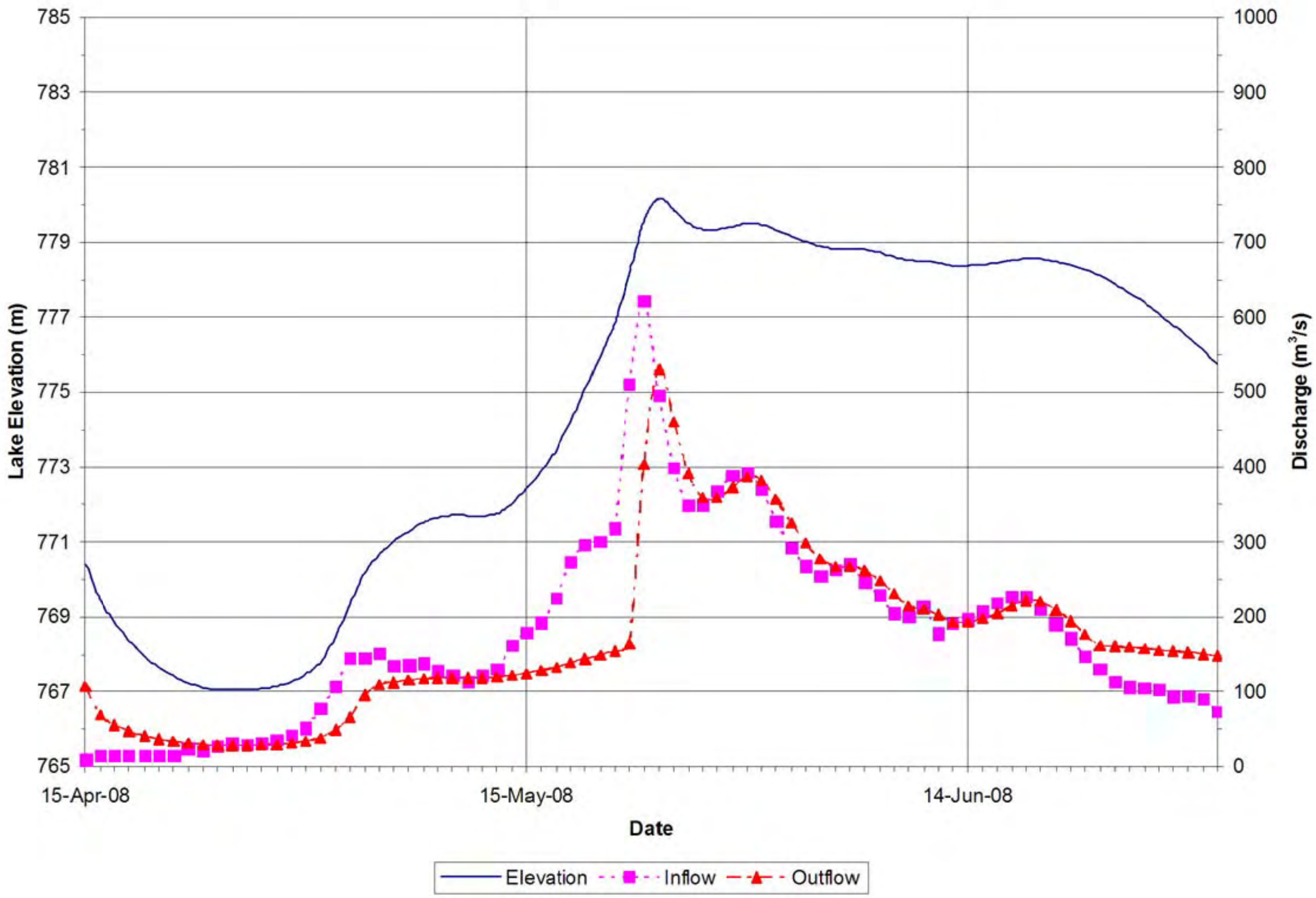
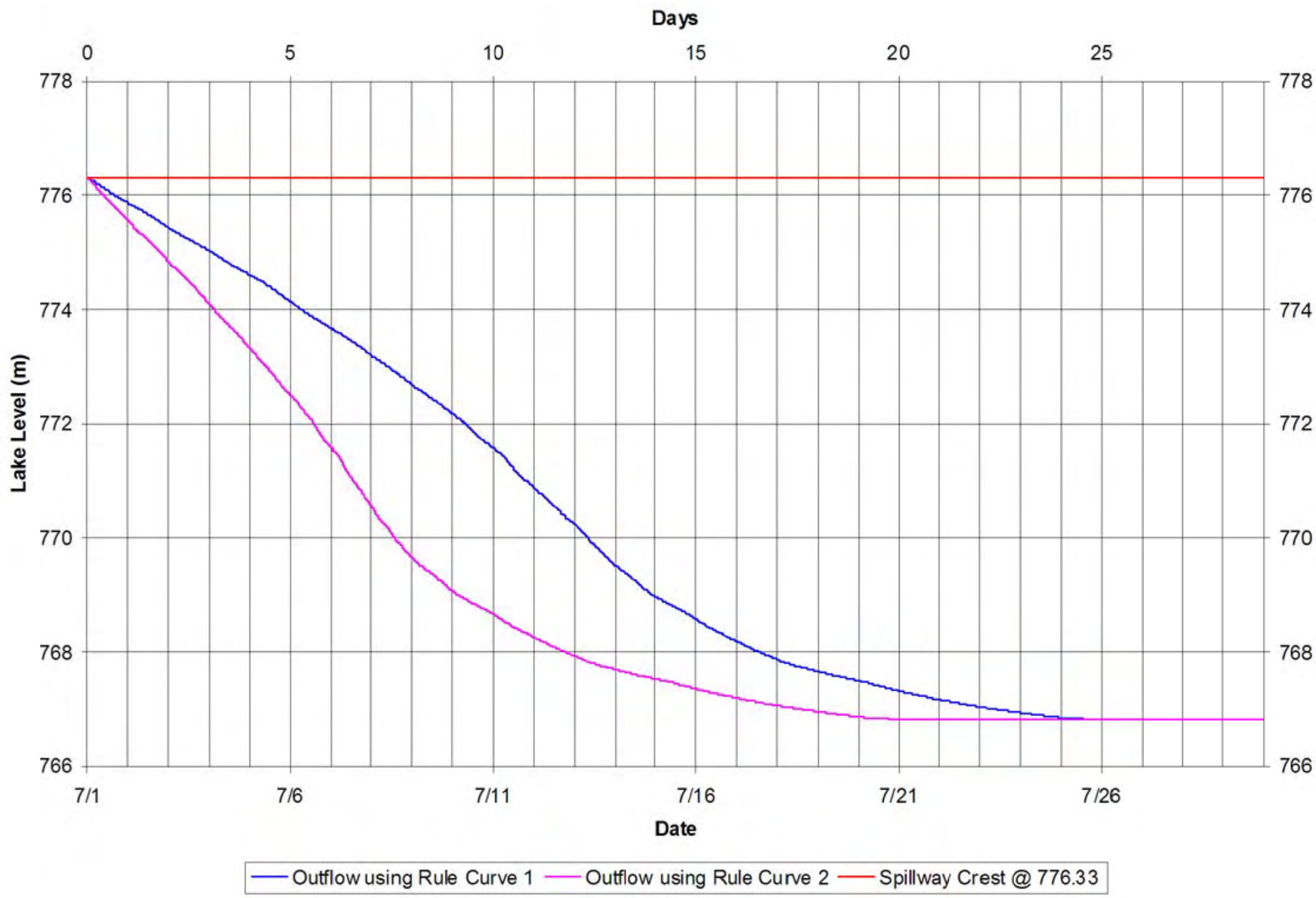
CLIENT					
Department of Fisheries and Oceans	FULTON DAM SAFETY REVIEW 2008 HYDROTECHNICAL ASSESSMENT				
	RESULTS OF FLOOD ROUTING SPRING IDF (CASE 1 RULE CURVE)				
HAY & COMPANY CONSULTANTS A DIVISION OF EBA	PROJECT NO. V13201140	DWN ML	CKD RJW	REV 0	Figure 15
	OFFICE VANC	DATE March 11, 2009			



ISSUED FOR USE

CLIENT	FULTON DAM SAFETY REVIEW 2008 HYDROTECHNICAL ASSESSMENT				
Department of Fisheries and Oceans	RESULTS OF FLOOD ROUTING SPRING IDF (CASE 2 RULE CURVE)				
HAY & COMPANY CONSULTANTS A DIVISION OF EBA	PROJECT NO. V13201140	DWN ML	CKD RJW	REV 0	Figure 16
	OFFICE VANC	DATE March 11, 2009			

Q:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Report Figures.dwg [FIGURE 17] March 11, 2009 - 1:40:14 pm (BY: MARIA LAU)



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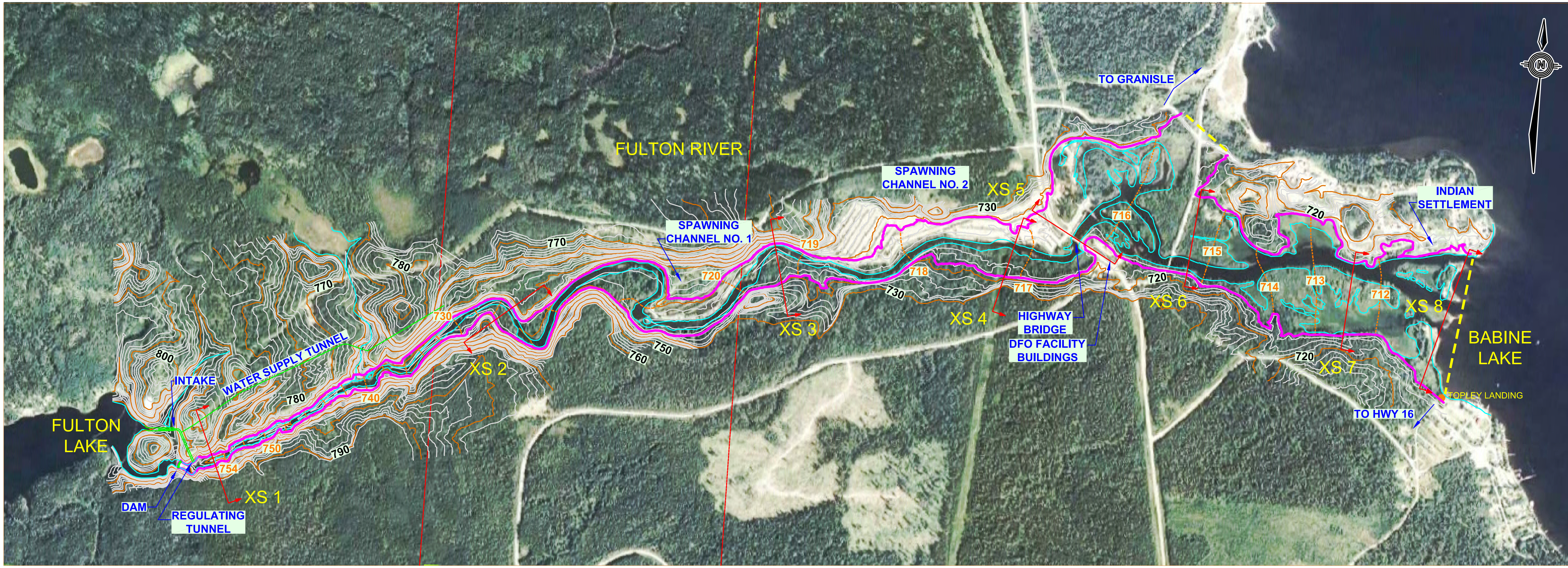
FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

RESERVOIR DRAWDOWN ANALYSIS

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure 17



DAM BREAK ANALYSIS - SUMMARY OF RESULTS

CROSS SECTION	XS 1	XS 2	XS 3	XS 4	XS 5	XS 6	XS 7	XS 8
DISTANCE FROM DAM (km)	0.00	1.37	2.86	3.88	4.14	4.81	5.49	5.84
TIME TO PEAK ELEVATION (min)	0.0	2.4	8.4	12.6	15.6	19.8	21.6	21.6
PEAK WATER ELEVATION (m)	755.0	725.4	719.2	717.1	716.8	715.4	712.4	711.2

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LEGEND/NOTES
- Base information taken from DFO Dwg No. 21-19-401
- Airphoto obtained from Google Earth
- Contours are 2m intervals

CONTOURS
WATERCOURSES
PROJECT STRUCTURES
SECTION LINES
FLOOD EXTENTS
FLOOD LEVEL ISOGRAMS
LIMIT OF STUDY

SCALE 1:8000
80 0 80 200 320m

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Department of Fisheries and Oceans

FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

INUNDATION MAP

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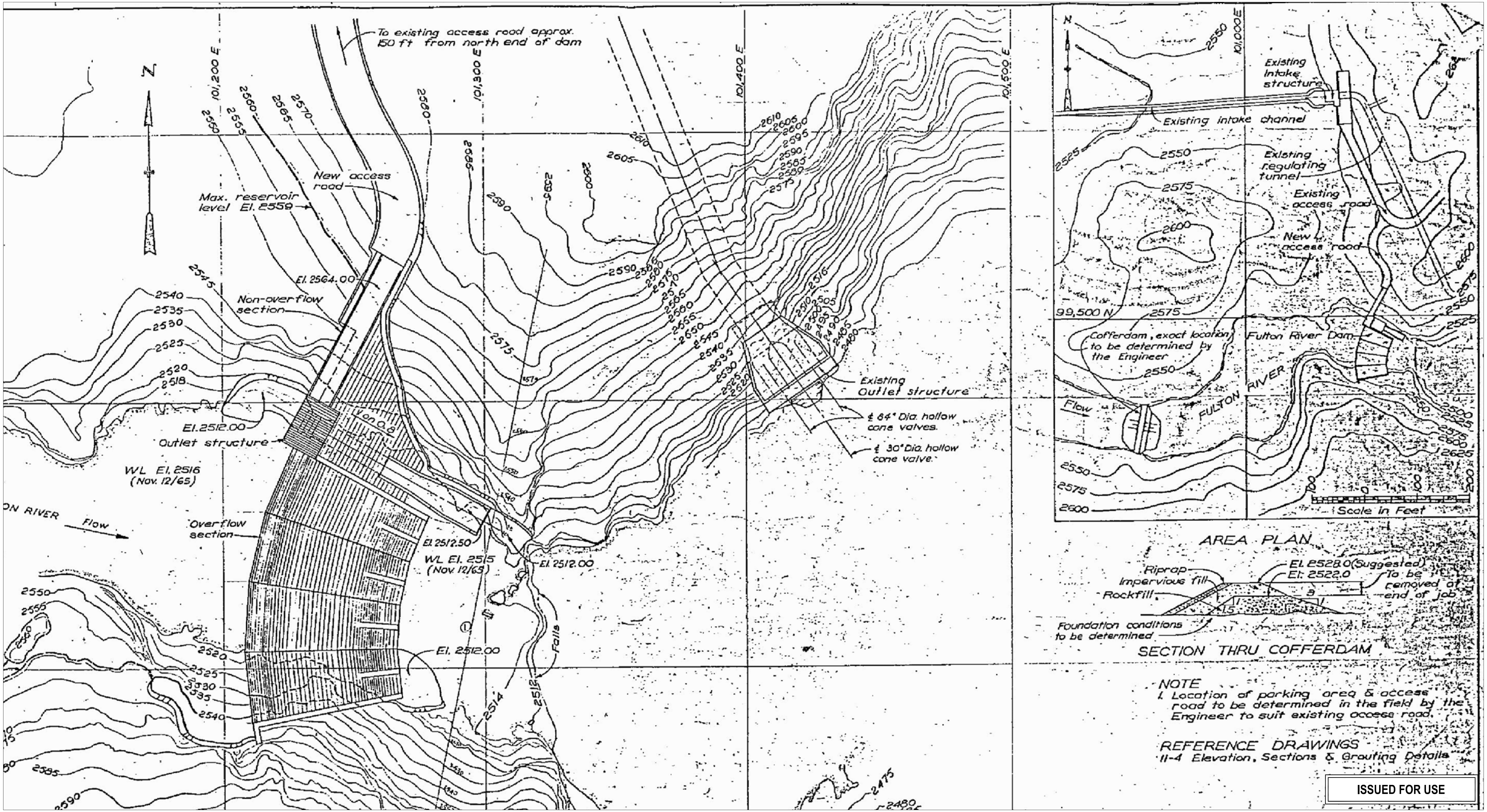
PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0

OFFICE	DATE
VANC	March 11, 2009

Figure 18

APPENDIX

APPENDIX A FULTON RIVER PROJECT DESIGN DRAWINGS

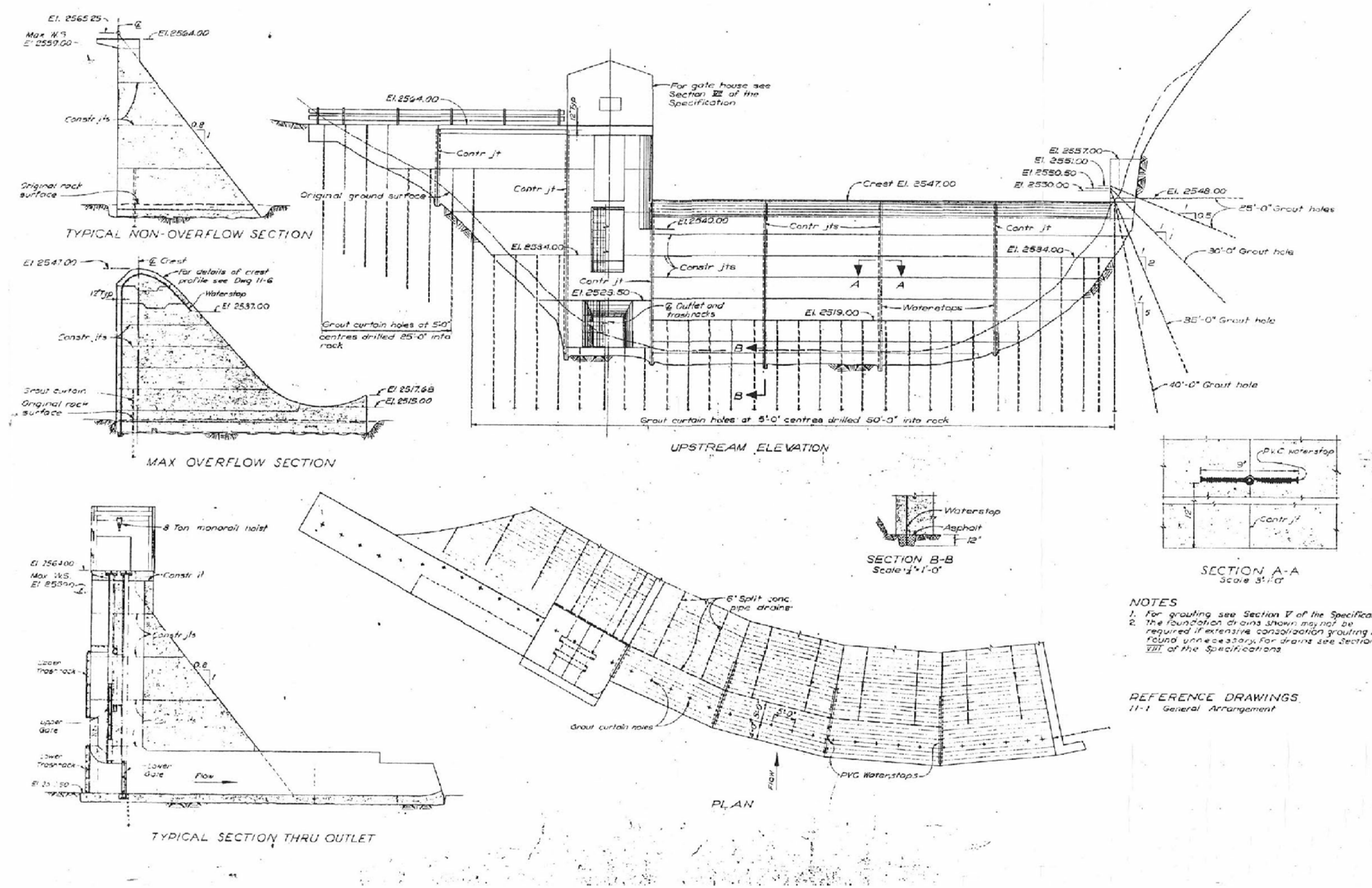


NOTES

Base drawing obtained from 1986 Fulton River Project Dam Breach Inundation Study by Patrick Fawkes & Associates - Figure 2 General Arrangement

CLIENT	Department of Fisheries and Oceans
	FULTON DAM SAFETY REVIEW 2008 HYDROTECHNICAL ASSESSMENT
	FULTON DAM PLAN VIEW
HAY & COMPANY CONSULTANTS A DIVISION OF EBA	

PROJECT NO.	DWN	CKD	REV	
V13201140	ML	RJW	0	Figure A-1
OFFICE	DATE			
VANC	March 11, 2009			



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NOTES

Base drawing obtained from Fulton Dam Safety Review report by UMA Engineering Ltd. dated August 1997 (2507-0175-001-00-01) based on DFO Drawing No. 983-II-4 Babine Lake Development Project No.II Fulton River Dam Elevation, Sections & Grouting Details

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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

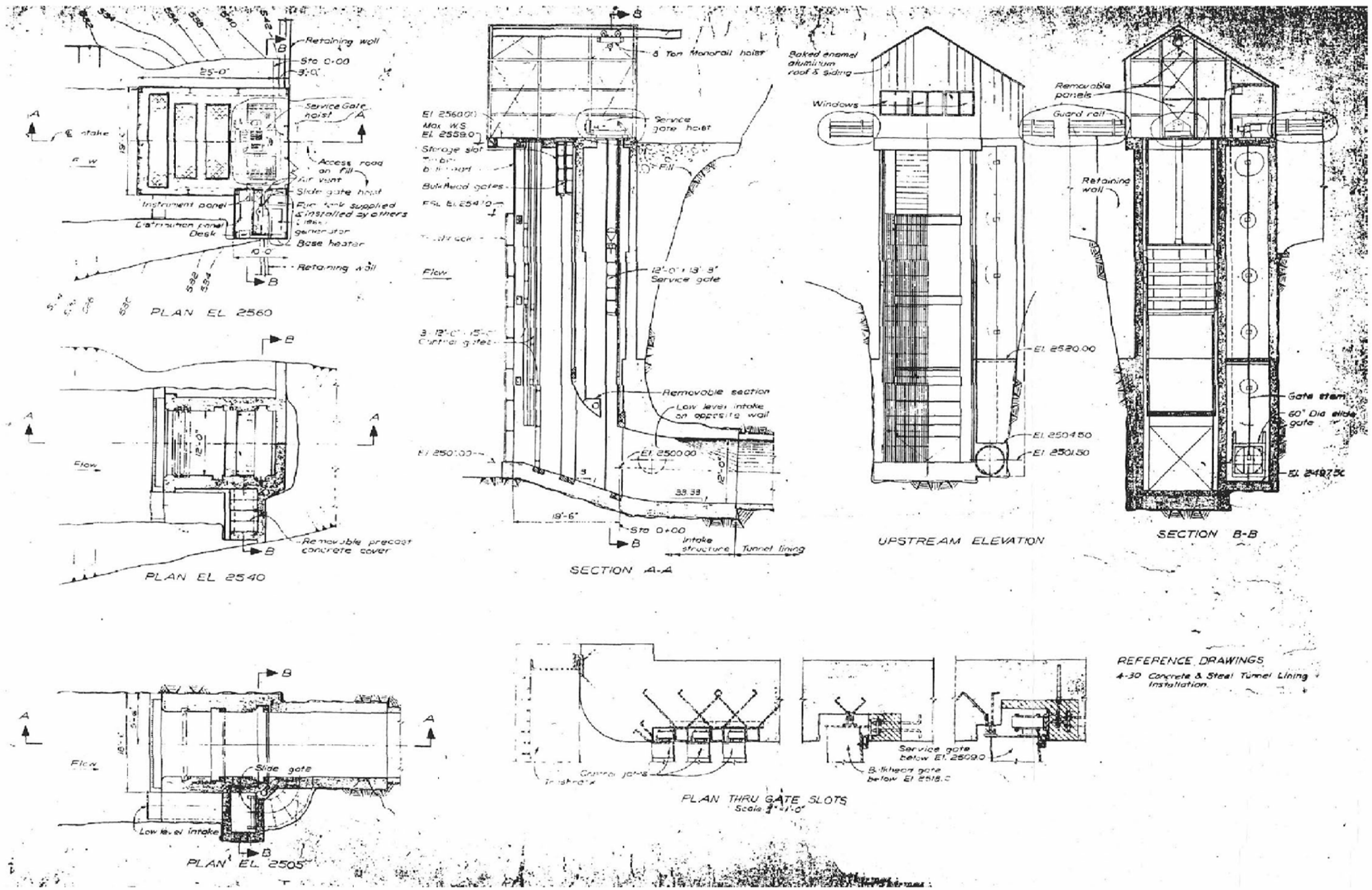
FULTON DAM
PLAN AND SECTIONS OF DAM

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0

OFFICE	DATE
VANC	March 11, 2009

Figure A-2

Q:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Appendix A.dwg [FIGURE A-2] March 12, 2009 - 1:53:23 pm (BY: MARIA LAU)



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NOTES

Base drawing obtained from Fulton Dam Safety Review report by UMA Engineering Ltd. dated August 1997 (2507-0175-001-00-01)
based on DFO Drawing No. 983-4-2R2 Babine Lake Development Project No. 4 Fulton River Regulating Works Intake General Arrangement

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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

FULTON DAM
REGULATING WORKS INTAKE

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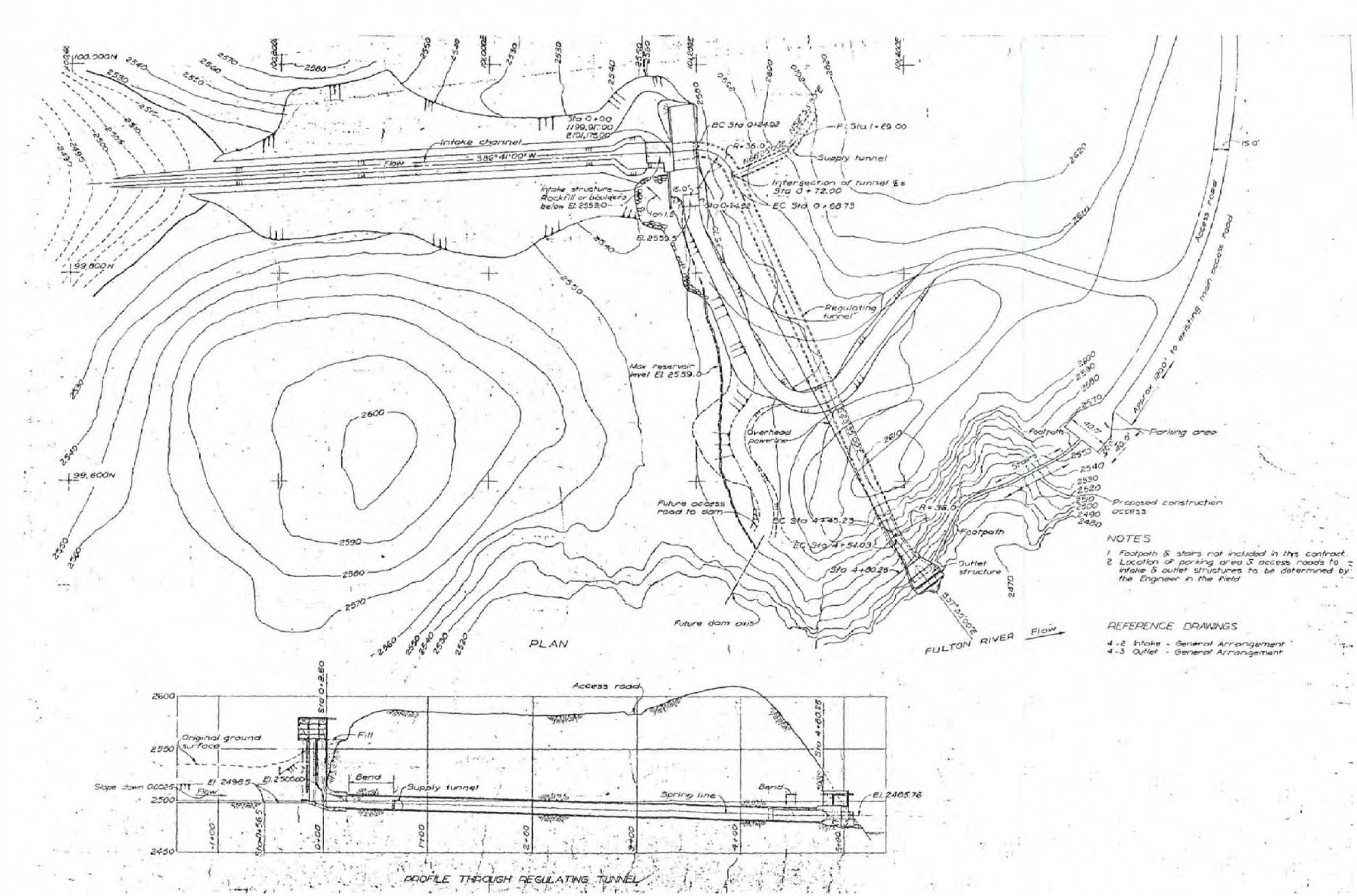
PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure A-3



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NOTES

Base drawing obtained from Fulton Dam Safety Review report by UMA Engineering Ltd. dated August 1997 (2507-0175-001-00-01) based on DFO Drawing No. 983-4-1R1 Babine Lake Development Project No. 4 Fulton River Regulating Works General Arranagement

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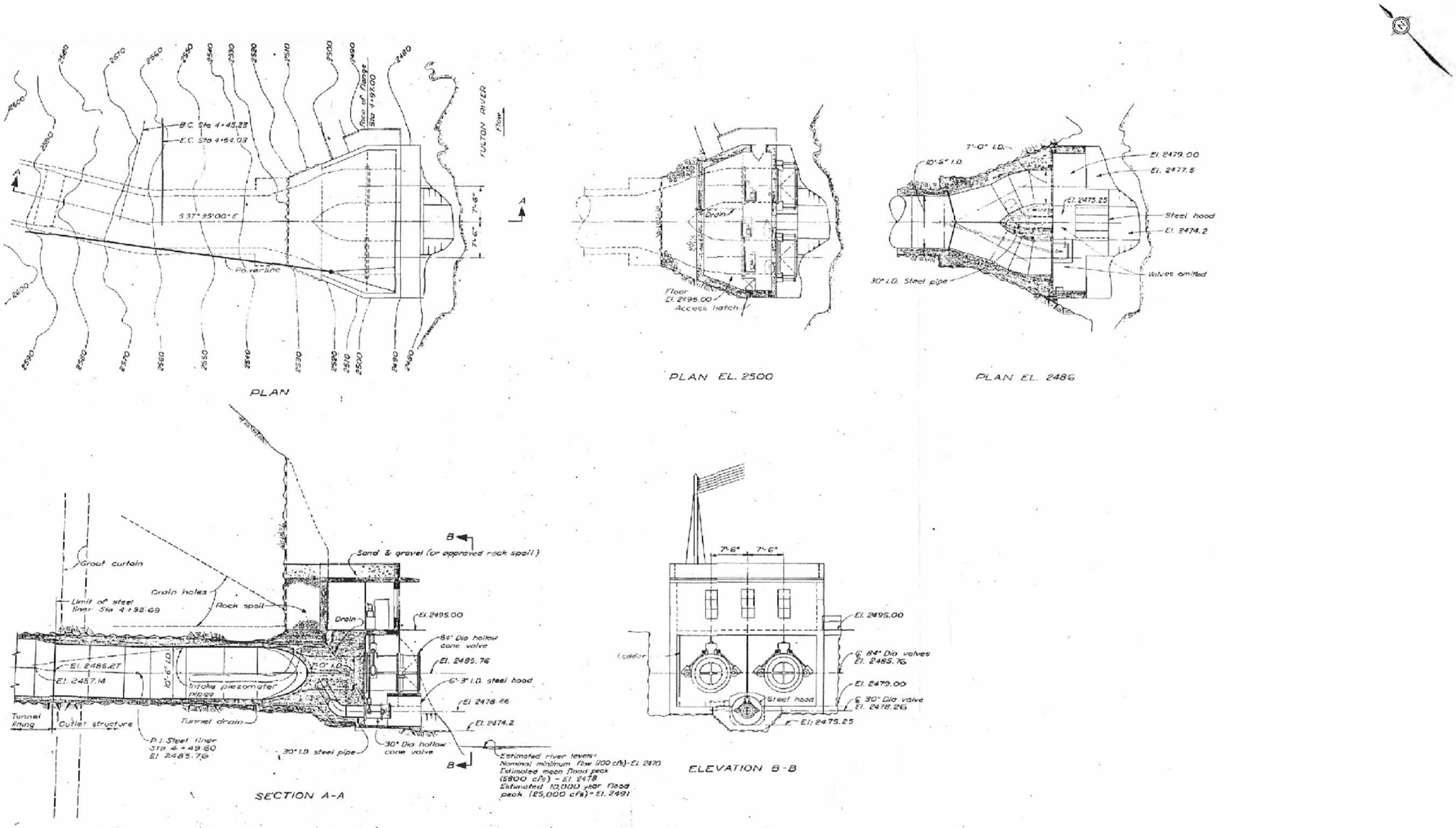
FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

FULTON DAM
PLAN AND SECTION OF REGULATING WORKS

PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure A-4



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NOTES

Base drawing obtained from Fulton Dam Safety Review report by UMA Engineering Ltd. dated August 1997 (2507-0175-001-00-01)
based on DFO Drawing No. 983-4-3R3 Babine Lake Development Project No. 4 Fulton River Regulating Works Outlet General Arranagement

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Department of Fisheries and Oceans	**FULTON DAM SAFETY REVIEW** **2008 HYDROTECHNICAL ASSESSMENT**
	FULTON DAM **REGULATING OUTLET STRUCTURE**

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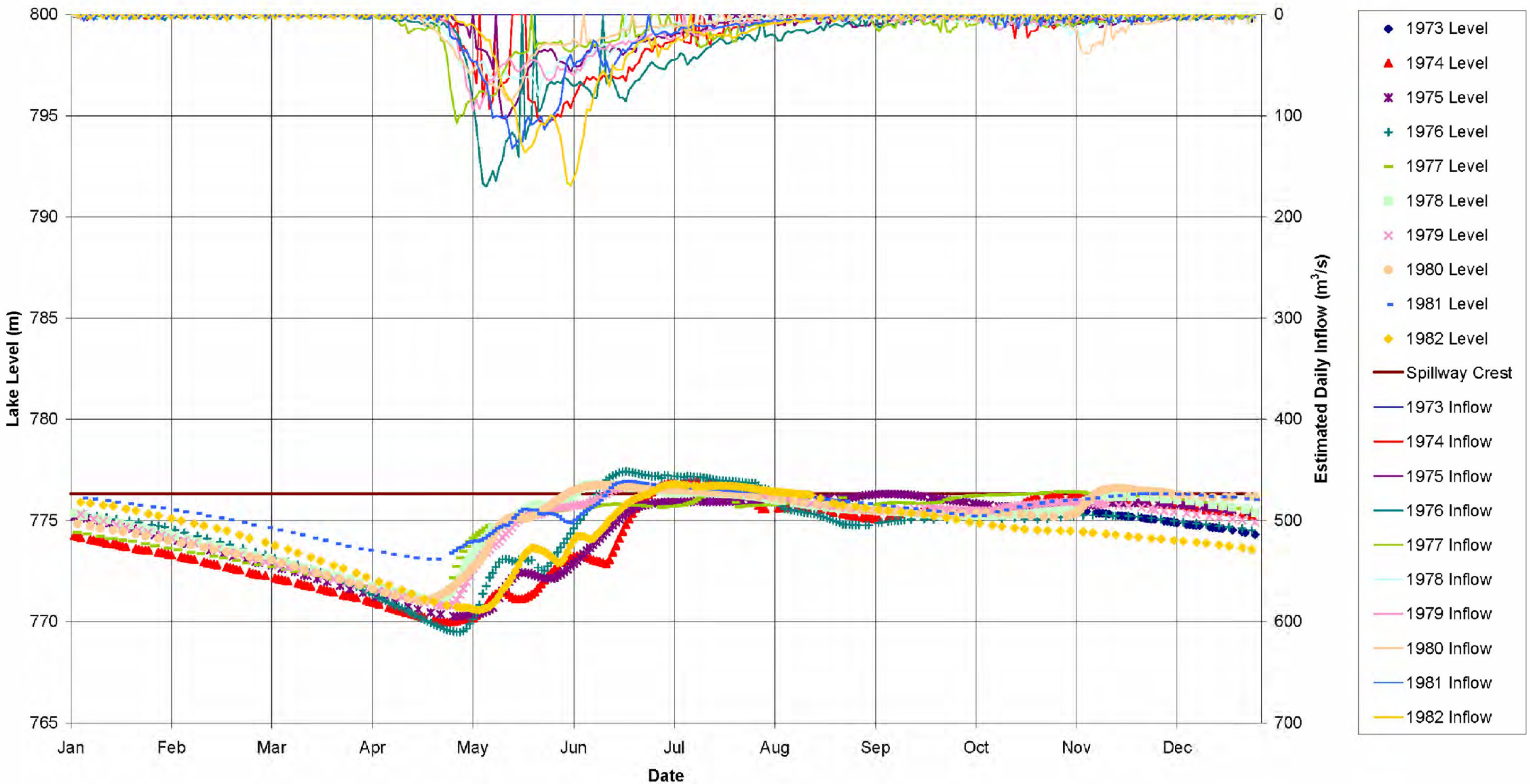
PROJECT NO.	DWN	CKD	REV	
V13201140	ML	RJW	0	**Figure A-5**
OFFICE	DATE			
VANC	March 11, 2009			

APPENDIX

APPENDIX B HISTORICAL DAILY LAKE ELEVATIONS AND ESTIMATED INFLOWS



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

HISTORICAL LAKE LEVELS AND ESTIMATED INFLOWS (1973 - 1982)

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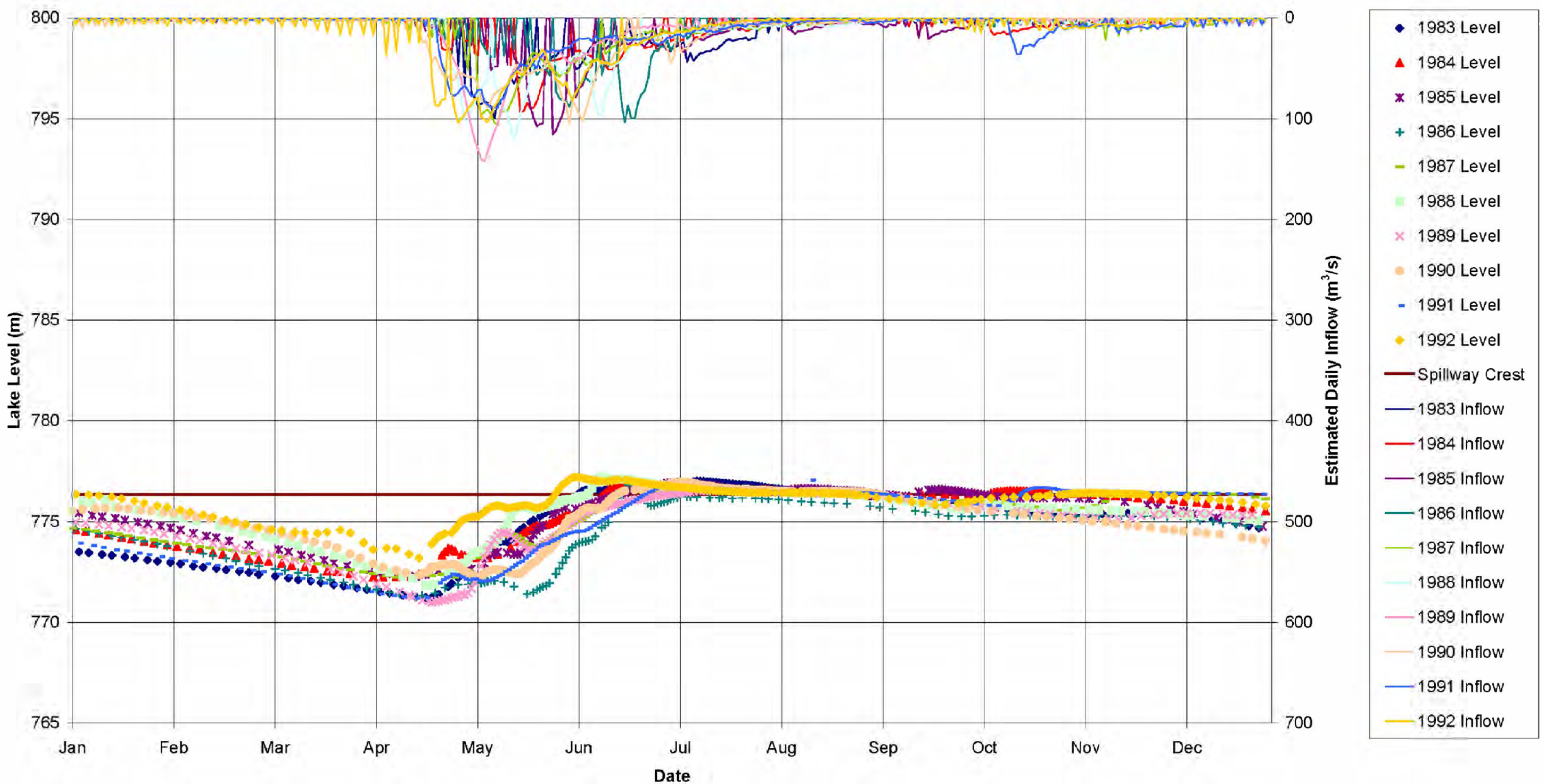
PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure B-1

Q:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Appendix B.dwg [FIGURE B-1] March 12, 2009 - 2:03:20 pm (BY: MARIA LAU)



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

HISTORICAL LAKE LEVELS AND ESTIMATED INFLOWS (1983 - 1992)

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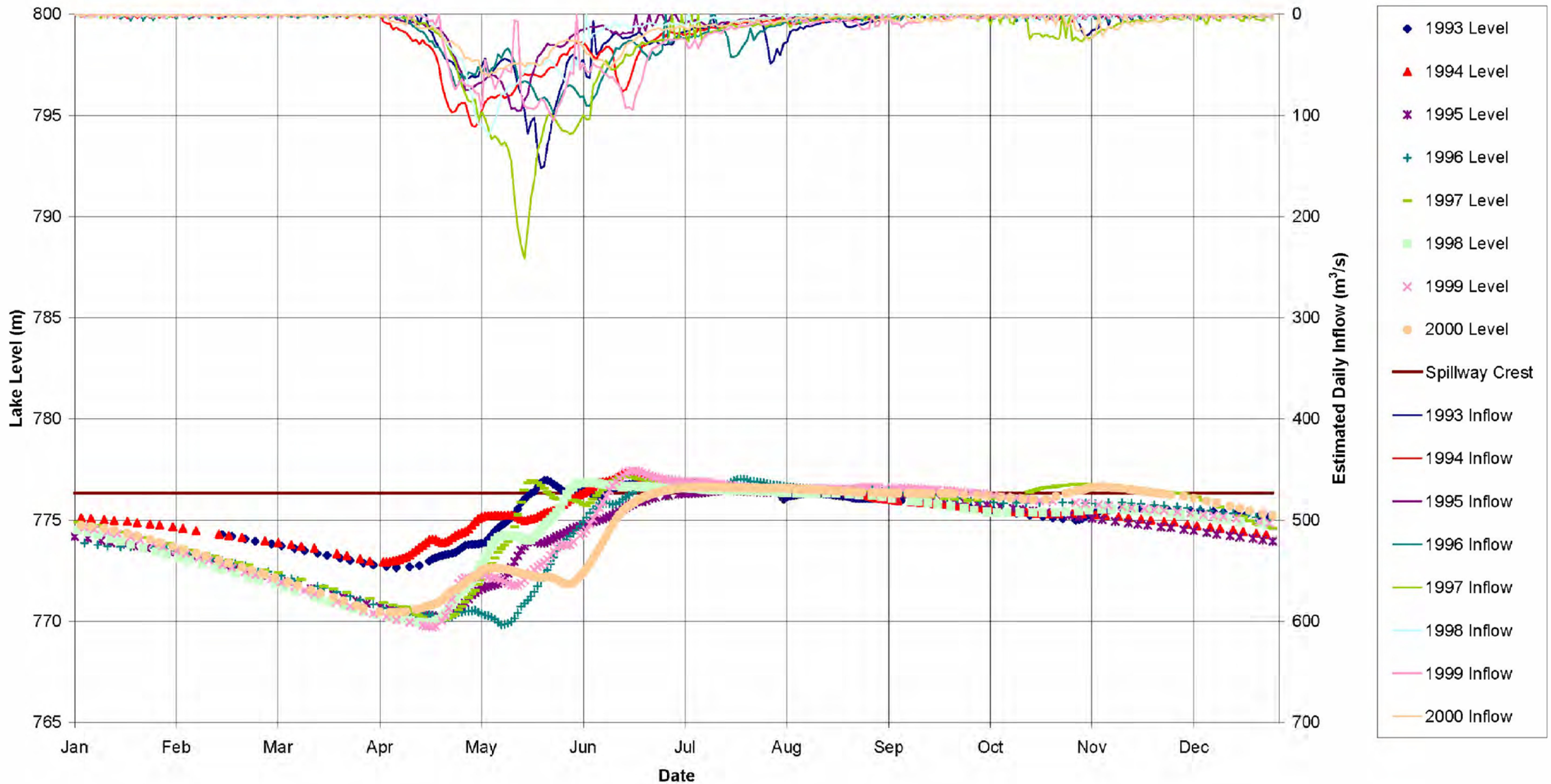
PROJECT NO.	DWN	CKD	REV	
V13201140	ML	RJW	0	Figure B-2
OFFICE: VANC	DATE: March 11, 2009			

O:\Vancouver\Engineering\V132\Projects\V13201140 Fulton Dam Safety Review\CADD\Appendix B.dwg [FIGURE B-2] March 12, 2009 - 2:04:52 pm (BY: MARIA LAU)



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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

HISTORICAL LAKE LEVELS AND ESTIMATED INFLOWS (1993 - 2000)

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PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure B-3

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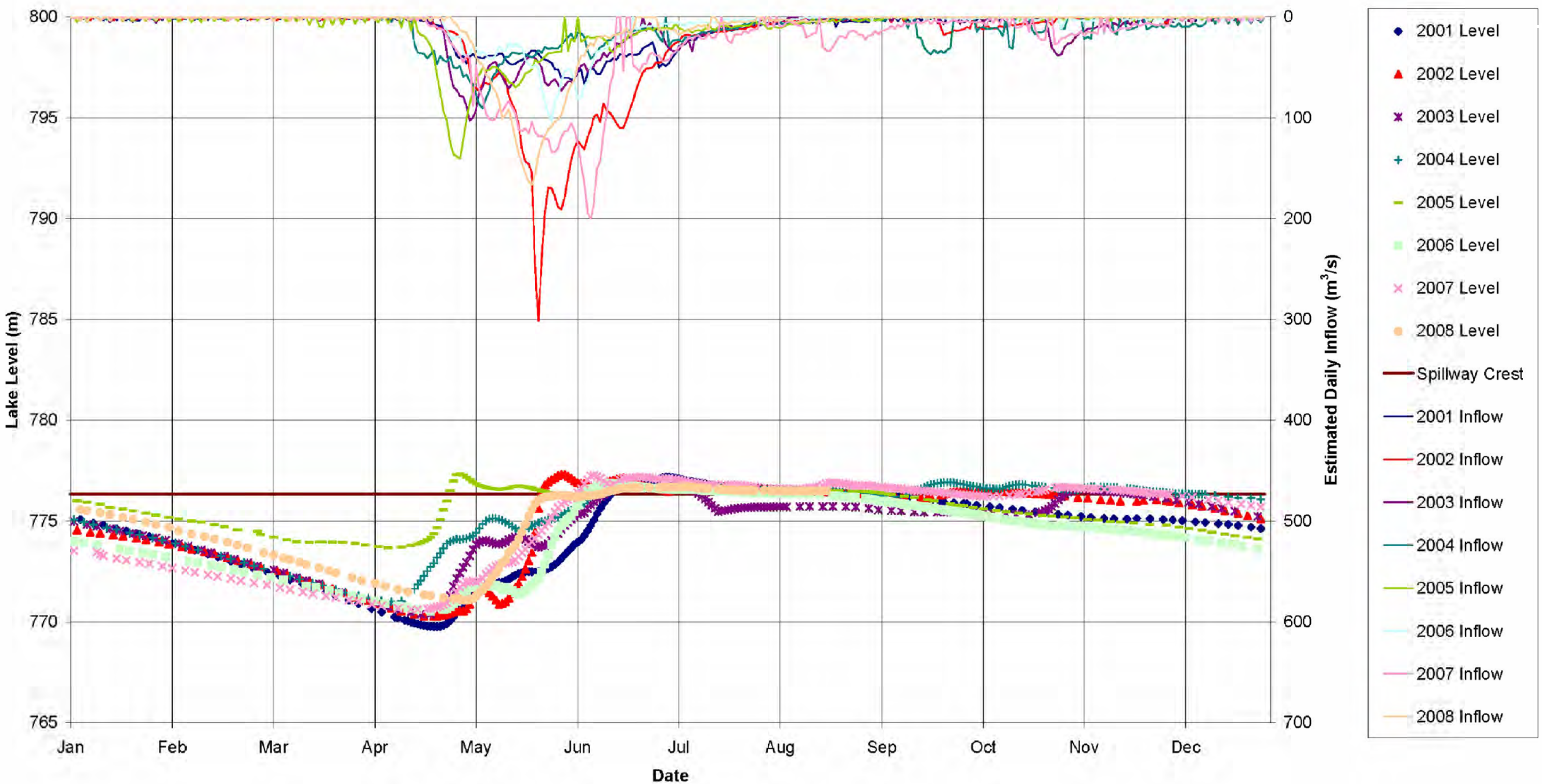

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FULTON DAM SAFETY REVIEW
2008 HYDROTECHNICAL ASSESSMENT

HISTORICAL LAKE LEVELS AND ESTIMATED INFLOWS (2001 - 2008)

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PROJECT NO.	DWN	CKD	REV
V13201140	ML	RJW	0
OFFICE	DATE		
VANC	March 11, 2009		

Figure B-4

APPENDIX

APPENDIX C HAYCO'S GENERAL CONDITIONS

PROJECT REPORT – GENERAL CONDITIONS

This Report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

These General Conditions apply to this Report, which Hay & Company Consultants, a Division of EBA Engineering Consultants Ltd. (Hayco) has prepared in fulfilment of certain project specific requirements that have been previously agreed to by Hayco and its Client. The Report may include plans, drawings, profiles and other support documents that collectively constitute the Report.

This Report pertains to a specific site, a specific development, and a specific scope of work. The Report and all supporting documents are intended for the sole use of Hayco's client. Hayco does not accept any responsibility for the accuracy of any of the data, analyses or other contents of the Report when it is used or relied upon by any party other than Hayco's Client, unless authorized in writing by Hayco. Any unauthorized use of the Report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without prior written permission of Hayco. Additional copies of the report, if required, may be obtained upon request.

2.0 CALCULATIONS AND DESIGNS

Hayco has undertaken design calculations and has prepared project specific recommendations or designs in accordance with terms of reference that were previously set out in consultation with, and agreement of, Hayco's client. These recommendations or designs have been prepared to a standard that is consistent with industry practice. Notwithstanding, if any error or omission is detected by Hayco's client or any party that is authorized to use the Report, the error or omission should be immediately drawn to the attention of Hayco.

3.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless so stipulated in the Report, Hayco was not retained to investigate, address or consider, and has not investigated, addressed or considered any environmental or regulatory issues associated with the project specific design.

4.0 ALTERNATIVE REPORT FORMAT

Where Hayco submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed Hayco's instruments of professional service); only the signed and/or sealed versions shall be considered final and legally binding. The original signed and/or sealed version archived by Hayco shall be deemed to be the original for the project.

Both electronic file and hard copy versions of Hayco's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except Hayco. Hayco's instruments of professional service will be used only and exactly as submitted by Hayco.

Electronic files submitted by Hayco have been prepared and submitted using specific software and hardware systems. Hayco makes no representation about the compatibility of these files with the client's current or future software and hardware systems.