BRITISH COLUMBIA

VANCOUVER 2010

IT'S OUR TIME TO SHINE

August 11, 2004

Files: 35050-20/P COQ-C-C
35050-20/COQ-CITY-C

Jeff Yip, P.Eng.
Deputy City Engineer
City of Port Coquitlam
2580 Shaughnessy Street
Port Coquitlam BC V3C 2A8

Ken Wright, P.Eng.
General Manager, Operations
City of Coquitlam
3000 Guildford Way
Coquitlam BC V3B 7N2

Dear Jeff Yip and Ken Wright:

Re: Coquitlam River Flood Hazard and Dike Safety Studies – New Dike Design Criteria

Over the past four years, the Cities of Port Coquitlam and Coquitlam and the ministry have jointly funded a series of flood hazard and dike safety studies of the Coquitlam River through the Flood Protection Assistance Fund. The latest report, "Coquitlam River Flood Hazard Mitigation Options", by Water Management Consultants, August 2004, completes the comprehensive technical review of hydrologic design criteria, flood level modeling and evaluation of mitigation options. As Inspector of Dikes, I hereby confirm that the new dike design criteria as recommended by this report would meet provincial standards and that the ministry concurs with and accepts the findings of the report.

The studies have determined that the Coquitlam River dikes, which were re-constructed (starting in 1989) under the Fraser River Flood Control Program, are generally deficient in crest elevation (typically 1 metre low). Immediately upstream of the CP Rail bridge, the dikes are up to 2.5 metres low as a result of the constriction of the bridge and probable debris jams. Overtopping and failure of these dikes could cause extensive flood damage to development in the Coquitlam

Ministry of Water, Land and Air Protection
Regional Operations
Lower Mainland Region
Mailing/Location Address:
10470 152 Street
SURREY BC V3R 0Y3
Telephone: (604) 582-5200
Facsimile: (604) 930-7119
http://www.gov.bc.ca/
http://www.gov.bc.ca/wlap/

River floodplain and is a serious public safety concern. The extent of possible flooding is shown on new floodplain mapping provided with the report.

The two primary mitigation options include:

1. increasing the low level outlet spill capacity at the Coquitlam Dam (constructing a high flow bypass facility) to reduce the magnitude of the design flood; and

2. raising existing dikes to contain the design flood.

While order of magnitude costs for both options are similar (roughly $4 to $6 million) and both options have a number of unresolved issues, Option 1 is recommended from a public safety perspective. Reducing the magnitude of peak flows lessens the potential for debris jamming, dike failure and flood damage.

The risk of flooding on the Coquitlam River is currently mitigated by BC Hydro's temporary operating procedures while the dam undergoes seismic upgrades. Until the upgrades are completed (expected completion date in 2006) BC Hydro will maintain the reservoir level at least 5 metres below the spillway elevation. This increased temporary storage will provide effective flood control for at least the next two years until the work is completed, but thereafter the flood risk will increase.

BC Hydro's recently completed Coquitlam-Buntzen Project Water Use Plan and a draft *Water Act* Order have been referred by Land and Water BC to the Cities and to the ministry. This office will be requesting that the Comptroller of Water Rights redraft the Order to require BC Hydro to examine the feasibility of increasing the capacity of the low level outlet at the Coquitlam Dam.

I wish to thank both Cities for co-operation on the Flood Protection Assistance Fund projects and for the continuing efforts to improve dike safety on the Coquitlam River.

Sincerely,

Neil Peters, M.A.Sc., P.Eng.
Inspector of Dikes

pc: Christine Houghton, Assistant Director, Public Safety and Prevention Initiatives
Lynn Bailey, Director, Regional Operations

WATER
MANAGEMENT
CONSULTANTS

130-10691 SHELLBRIDGE WAY
RICHMOND, B.C. V6X 2W8,
CANADA

TELEPHONE: (604) 273 6299
TELEFAX: (604) 270 3644

August 16, 2004

City of Port Coquitlam
2580 Shaughnessy Street
Port Coquitlam, BC
V3C 2A8

Attention: Allen Jensen, AScT

Re: Flood Hazard Mitigation Options –Coquitlam River

Dear Sir:

Water Management Consultants in association with Associated Engineering (BC) Ltd. are pleased to enclose our Report on Coquitlam River Flood Hazard Mitigation Options. Floodplain maps have been submitted as a separate document.

Thanks you for this opportunity to provide services to the City of Port Coquitlam.

Sincerely,

CDN WATER MANAGEMENT CONSULTANTS INC.

C. David Sellars, P. Eng.
Project Manager

CDS:ws
Attachment

COQUITLAM RIVER FLOOD HAZARD MITIGATION OPTIONS

August 2004

7095

Prepared for:

City of Port Coquitlam
2580 Shaughnessy Street
Port Coquitlam
V3C 2A8

Prepared by:

Water Management Consultants
130-10691 Shellbridge Way
Richmond, BC
V6X 2W8

with Associated Engineering of Burnaby, BC.

CONTENTS

FIGURES

DRAWINGS

1. INTRODUCTION

1.1. Background

Flooding in the Coquitlam River Valley generally occurs following major rainfall events in the winter months, typically between November and April. Flooding in the lower part of the valley also occurs in the late spring and summer from backwater effects from high Fraser River levels.

The interim floodplain maps for the Coquitlam River were based on the hydrology studies carried out for the Coquitlam River Water Management Study[1] published in 1978. The 200-year flood was derived using a unit hydrograph approach and flood routing was carried out in the Coquitlam Reservoir to determine the attenuating effect of the reservoir. As expected, it was found that the degree of attenuation depended mainly on the initial reservoir level, or the amount of vacant storage prior to the inflow. In these studies it was assumed, conservatively that the Coquitlam Reservoir was full to the spillway level at the start of flood routing.

Since 1978, there is considerably more data available for evaluating the magnitude of the 200-year flood. Furthermore, the Coquitlam Reservoir will be operated to meet the needs of both power generation and increasing water supply to the Greater Vancouver Regional District (GVRD). To provide flood reduction benefits, the reservoir will be operated to provide one m of freeboard below the spillway crest. When the reservoir rises above this elevation, the power tunnel and low level outlets will be operated at maximum capacity. The 200-year flood outflow from the reservoir was recalculated in a report by Water Management Consultants (March, 2003) using expected future Coquitlam Reservoir water levels prior to the onset of the flood. Flood levels along the Coquitlam River were calculated using water surface profile analysis to determine the magnitude of the flood reduction effects of the reservoir.

The scope of work for the study included the following key elements:

[1] Coquitlam River Water Management Study, Water Investigations Branch, British Columbia Ministry of Environment, 1978

- Review content of memo dated August 20, 2003 by Neil Peters, Inspector of Dikes.
- Carryout analyses of the sensitivity of the peak reservoir outflow magnitude and hydrograph shape to inflow hydrograph magnitude and shape.
- Perform additional hydrological analyses on tributary inflows below the dam. These analyses should consider more complex, longer duration storms for the determination of the inflow hydrographs and different approaches for combining the outflows from the dam and the inflows from the tributaries below the dam. The sensitivity of the design flow magnitude to various assumptions and uncertainties should be addressed.
- Estimate the confidence limits for the recommended final design flow and the sensitivity of the flood profile to various assumptions.
- Carryout additional hydraulic modeling, using the existing HEC RAS model, to simulate a partial debris blockage at the Kingsway Bridge. Also, modify the existing model to simulate the potential effects of sediment deposition in the constricted reach adjacent to downtown Port Coquitlam.
- Make recommendations on the final dike design flow, flood profile and freeboard allowance to be adopted for the Coquitlam River dikes.
- Update the floodplain maps based on the final flood profile for the Coquitlam River.
- Carry out site survey and preliminary design of flood mitigation works.
- Prepare a detailed design for debris control at the CPR Bridge.
- Design erosion protection works

1.2. Coquitlam River Watershed

The Coquitlam River watershed is located in the Coast Ranges of British Columbia. The northern part of the watershed is mountainous while the southern part is located in the lowlands of the Fraser River valley. The total catchment area is 292 km^2 of which 212 km^2 (73%) is controlled by the Coquitlam Dam, which forms the 12.5 km^2 Coquitlam Lake. There are two tributaries of note that enter the Coquitlam River downstream of Coquitlam Dam. The Larger of the two tributaries, Or Creek, has a catchment area of 21.5 km^2 and has a mountainous watershed. Scott Creek joins the Coquitlam River downstream of the Lougheed Highway and has a mostly urban catchment area of 17 km^2. The upper part of the Coquitlam River below the dam is steep and flood flows are generally confined to a narrow valley. There is a rapid decrease in bed slope downstream of the Lougheed Highway where the Coquitlam River flows onto its fan and the Fraser River floodplain (Northwest Hydraulics, 2002). This reach of the Coquitlam River is the most susceptible to flooding.

Urban development has occurred in the southern part of the watershed and therefore floodplain mapping is required to guide appropriate land uses and to provide Flood Construction Levels (FCLs).

Figure 1.1 is a map of the watershed showing the location of the Coquitlam Reservoir and the two WSC gauges on the Coquitlam River:

- Coquitlam River above Coquitlam Lake: Gauge # 08MH141
- Coquitlam River near Port Coquitlam: Gauge # 08MH002

2. PREVIOUS STUDIES

The hydrology of the Coquitlam River was analyzed as part of the Coquitlam River Water Management Study in 1978 (Water Investigations Branch, 1978). A 200-year flood hydrograph was routed through Coquitlam Lake and added to an estimated 200-year downstream hydrograph. Alternative initial Coquitlam Reservoir starting elevations were investigated as part of this analysis. However, the design condition used was based on the initial reservoir level at the spillway crest level.

The Coquitlam River hydrology was reviewed in 1988 as part of the design work for the Coquitlam River Flood Control Works (Associated Engineering, 1988). That report relied extensively on the 1978 report but peak daily flows were reported rather than 12-hour flows. Instantaneous flood peaks were not addressed in either the 1978 or the 1988 report.

Water Management Consultants (March, 2003) carried out a hydrology review of the Coquitlam River and incorporated the BC Hydro Coquitlam Reservoir operating rules in the assessment. This resulted in significantly more reservoir routing effects and the design downstream flood flow reduced to a peak instantaneous flow of 402 m^3/s at the CP Rail Bridge and 425 m^3/s at the confluence with the Fraser River

The flood estimates from the 1978, 1988 and 2003 reports are presented in Table 2.1.

Table 2.1: 200-year peak flows in m^3/s from previous studies

	12- hour peak flows from 1978 report	Peak daily flows from 1988 report	Peak instantaneous flows from March 2003 report
Inflow to Coquitlam Lake	561		855
Outflow from Coquitlam Lake	386		314
Inflow below dam	311		88
Flow at gauge 8MH002	586	573	402
Flow downstream of Scott Creek		640	425

The Coquitlam dyking system was designed based on the peak daily flows from the 1988 report listed in Table 2.1. The water levels were derived by water surface profile analysis and 0.6 m of freeboard was added. The British Columbia provincial standard is to include 0.6 m of freeboard for analysis based on daily flows and 0.3 m of freeboard when the analysis is based on instantaneous peak flows.

Cross sections of the Coquitlam River were re-surveyed by Bland (2001a). Hydraulic modelling was carried out by Bland (2001b) and flood levels estimated in the vicinity of the First Nation Lands downstream of Pitt River Road. Peak daily flows from the 1988 report were used for the hydraulic modelling.

Northwest Hydraulics (2002) re-assessed flooding and erosion hazards along the Coquitlam River and carried out a water surface profile analysis of the Coquitlam River from Hockaday Street to the Fraser River. The peak daily flows from the 1988 report were again used for the modelling.

3. FLOOD FREQUENCY ANALYSES

3.1. Peak instantaneous flows

The Coquitlam River is regulated by Coquitlam Reservoir which impacts the magnitude of downstream floods. To estimate the 200-year natural inflows to the reservoir and the contributions of the downstream tributaries, a regional flood frequency analysis was conducted.

Using the FFAME program developed by the former BC Ministry of Environment Lands and Parks, a flood frequency analysis was carried out for several watersheds with similar characteristics. All of the watersheds are in mountainous terrain with catchment areas ranging from 2.6 km^2 for Noons Creek to 172 km^2 for Capilano River. Geographically they extend from Norrish Creek near Mission to Capilano River near Vancouver, consistent with the coastal hydrologic zone.

Four frequency distributions were considered in the analysis; three parameter log-normal, Gumbel, Pearson Type III, and Log Pearson Type III. Pearson Type III was the distribution that best fits the data as a whole for all the stations.

Table 3.1: Estimates of peak instantaneous 200-year floods

Station ID	Station Name	Area (km^2)	Years of instantaneous data	200 yr Flood (m^3/s)
08GA065	Noons Creek at Meridian Substation Road	2.59	15	17
08GA061	Mackay Creek at Montroyal Blvd	3.63	23	19
08MH006	North Alouette River at 232nd Street	37.3	27	186
08MH141	Coquitlam R above Coquitlam Lake	54.7	15	200
08MH076	Kanaka Creek near Webster Corners	47.7	33	279
08MH058	Norrish Creek Near Dewdney	117	28	479
08GA010	Capilano Creek above Intake	172	40	716

Table 3.1 lists the stations, sorted by catchment area, with the estimate of the 200-year return period flood. The 200-year flood discharge generally increases with catchment area with the smallest estimate of 17 m^3/s for Noons Creek and the largest 200-year flood estimate of 716 m^3/s for Capilano River. Figure 3.1 illustrates the relationship between the estimated 200-year flood discharges and the catchment area.

The line of best fit in Figure 3.1 has the equation:

$$Q_{200} = 6.62\ A^{0.9076}$$

where Q is the instantaneous flood peak in m^3/s and
A is the catchment area in km^2.

The correlation coefficient R^2 is 0.99. The line passes through the 200-year estimate for the Capilano River which has a similar watershed characteristics to the Coquitlam Watershed in terms of aspect, elevations and slope.

The line of best fit in Figure 3.1 was used to estimate the instantaneous 200-year flood peak inflow to Coquitlam Reservoir. The value indicated in Figure 3.1 is 855 m^3/s for the 212 km^2 watershed area.

Also shown in Figure 3.1 is the line representing the upper 95% confidence limit for the regional flood frequency analysis. On average, the upper 95% confidence limit provides a 200-year discharge estimate about 25% larger than the average projected discharge estimate.

3.2. Reservoir Inflow Volume

Inflow volume frequency analyses were carried out for periods ranging from 1 to 110 days. Daily reservoir inflow data were provided by BC Hydro for a period of 44 years. These data were compiled by back-calculating inflows from changes in reservoir levels and outflows.

The results of the frequency analyses are shown in Table 3.2 and Figure 3.2. The 200-year peak daily inflow was found to be 615 m^3/s. This is in reasonable agreement with the estimated instantaneous peak 200-year inflow flood to the reservoir of 855 m^3/s. This results in a ratio of 1.39 between the peak instantaneous flood and the daily flow. The ratio for the Capilano River above the intake was found to be 1.51, consistent with the smaller catchment area of 172 km^2 compared with 212 km^2 for the Coquitlam Reservoir.

Table 3.2: Inflow volume frequency analysis

Days	200-Year Inflow Volume in $m^3 \times 10^6$
1	53
2	71
3	86
5	104
7	124
10	149
15	179
30	261
60	376
80	442
90	469
100	494
110	518

3.3. Data from Water Investigations Branch 1961 report

'Preliminary Report on Coquitlam River Flooding' by R. A Pollard of the provincial Water Rights Branch was issued in December 1961. The report contains data on the two largest floods on record (1921 and 1961) and a complete series of maximum annual daily inflows to the Coquitlam Reservoir from 1914 to 1961. BC Hydro used this data from the Pollard Report to extend their 44 years of maximum annual daily inflows for analysis in their dam safety studies for Coquitlam Dam.

The maximum annual peak daily inflow data set to Coquitlam Lake was extended back to 1914 using the data from the Pollard Report. This gave a total period of record of 86 years. A frequency analysis was done on the extended period of record with the results shown in Table 3.3 together with recorded data from the 1921 and 1961 floods.

Table 3.3: Comparison of peak reservoir inflows in m^3/s

	Peak daily inflow	Peak instantaneous inflow
200-year with 44 years of record	613	855[1]
200 year with 86 years of record	585	
1921 recorded	508	822
1961 recorded	424	728

1. From regional flood frequency analysis

Table 3.3 shows that extending the flow record to 86 years results in a slight decrease of the 200-year peak daily inflow estimate. The recorded daily and peak instantaneous inflows for the maximum recorded floods of 1921 and 1961 were both less than the 200-year flood estimates. For this study the frequency analysis from the 44-years of data was used as the data are quality controlled and the estimates provide slightly higher values..

Both the 1921 and the 1961 floods caused significant damages downstream because the reservoir was full prior to the major inflows and thus had limited flood routing capacity. The Pollard report contains reservoir water level data prior to these floods. In 1921, the reservoir was above the spillway crest prior to the flood and was spilling about 50 m^3/s at 8.00 am on October 27. Over the next 24 hours, 192 mm of rain was recorded. On October 28, 206 mm of rain fell and the peak runoff from the watershed occurred on October 28.

Coquitlam Reservoir was also full prior to the 1961 flood. The reservoir was spilling intermittently throughout December and early January 1961. The peak daily inflow occurred on January 15.

As mentioned in Section 1.1, the current Coquitlam Reservoir operation policy is to maintain a maximum reservoir level one metre below the spillway crest for flood relief storage. Furthermore the reservoir is currently operated at lower levels to minimize spilling, a strategy that is feasible with the larger current power demands and additional power sources providing flexibility. Had such practices been applied in 1921 and 1961 the flood damages from these storms would likely have been greatly reduced.

4. DEVELOPMENT OF DESIGN FLOOD HYDROGRAPHS

There are three key issues in the development of the design 200-year flow for the Coquitlam River.

1. The shape of the 200-year inflow hydrograph to the reservoir.
2. The magnitude of the concurrent downstream tributary flows
3. The starting reservoir levels.

This section of the report addresses the shape of the 200-year inflow hydrograph and the magnitude of the concurrent downstream flows. Starting reservoir levels are addressed in Section 5.

4.1. Analysis of the timing of reservoir inflows and downstream tributary flows

To investigate the timing of reservoir inflow hydrographs and concurrent downstream flows, hourly flow data for the ten largest floods of record were obtained from Water Survey of Canada. The gauge 08MH141 is upstream of the reservoir and is indicative of reservoir inflows. The gauge 08MH002 is located at the CP Rail Bridge and records the combined flows from the downstream tributaries, including Or Creek, and any releases from the Coquitlam Dam.

Figures 4.1 to 4.5 show the concurrent flows for the upstream and downstream gauges during the floods of 1989, 1990, 1995, 2002, and 2003. Coquitlam Dam releases were removed from the recorded downstream hydrographs in 1995 and 2002 to reflect natural flows in the tributaries below the dam. It can be seen in Figures 4.1 to 4.5 that the peak flows to the reservoir and from the downstream tributaries generally occur within one or two hours of each other. The coincident timing of peak discharges occur because the downstream discharges are dominated by the flows from Or Creek which has a similar hydrologic regime as the tributaries upstream of the reservoir.

When a storm hydrograph is routed through the reservoir the outflow peak is reduced and, depending on the initial reservoir levels and hydrograph shape, can be delayed up to 15 hours after the peak inflow occurs. The delay in peak flow from the reservoir alleviates downstream flooding since the high downstream flows have essentially passed prior to the peak reservoir outflow. However, a second storm peak, such as occurred in 2003, could increase flows in downstream tributaries at the same time peak reservoir outflow is occurring.

On the other hand, storms with large discharges after the peak normally have lower flows prior to the peak resulting in more available reservoir storage that is able to absorb the storm volume and thus lower spillway discharges during the second wave of the storm. This trade-off in volume before and after the peak is discussed further in the following sections.

4.2. Development of multi-day inflow hydrographs

In general, significant storms occur consecutively about 2 to 4 days apart. There are very few occurrences of second peaks occurring within 24 hours of a major peak flow. The hourly flow records were examined and the hydrograph of November 22 to 26, 1995 was selected as the base storm for consecutive peak events. The storm of 2003, with major peak discharges 29 hours apart, was also investigated.

The key issue for flood routing in reservoirs is the antecedent conditions. The appropriate inflow conditions prior to the 200-year flood peak have to be defined together with the appropriate starting reservoir water level. This is particularly important for the Coquitlam Reservoir which was not originally designed as a flood control reservoir and has limited outlet capacity for maintaining reservoir elevations for flood control during high inflow periods.

To determine the most conservative reservoir outflow condition, 200-year storms of various durations were developed and routed through the reservoir. The multi-day hydrographs were developed such that larger durations 200-year storms contained each of the shorter duration events nested within the storm.

The above procedure is in accordance with standard practices that are followed by dam engineers that were originally developed by the US Bureau of Reclamation for flood control dams in the United States and have been adopted around the world. Operation of flood control reservoirs in the United States is now primarily the responsibility of the US Army Corps of Engineers. In the US Army Corps Engineering Manual Hydrologic Engineering Design Requirements for Reservoirs Chapter 10, Flood Control Storage, (1997) It is noted in Section 10-4 b that:

> "A hypothetical flood corresponding to a specified frequency should contain runoff volumes for all pertinent durations corresponding to that specified frequency."

The 200-year 1990 and 1995 synthetic multi-day hydrographs (2 to 15 days) were developed by modifying recorded flows around the peak ensuring that incremental inflow volumes from Table 3.2 are incorporated. The storm peak for the 2 to 15 day events was arbitrarily located at the tail end of the time period. For longer duration events, 15 to 110 days, the shorter periods containing the 200-year instantaneous flood peak were inserted in the middle of the longer duration events. The middle of the period on longer events was selected so that the synthetic storms would not be overly biased toward having the majority of volume either before or after the peak period.

4.3. Approach using the 1990 flood and unit peak runoff for tributaries

In the WMC (March 2003) report, a seven-day hydrograph was developed based on hourly flow data for the flood of November 9-13 1990 from the flow gauge on the Coquitlam River above Coquitlam Lake (08MH141). This was the largest flood for the period of record for the gauge. Using the hourly data from the highest 24-hour inflow period (November 10 to November 11), flows were scaled such that the peak matched the 200-year instantaneous peak of 855 m^3/s and the 24-hour volume was equal to the 200-year daily volume of 53 Mm^3. The 2, 3, 5, and 7-day 200-year synthetic storm hydrographs were then developed such that the flow volumes were consistent with the values in Table 3.2 with shorter events nested within the longer durations. The 200-year daily flow and instantaneous peak were located on the second day of the two-day event, the middle of the three-day storm, and near the end of the five and seven-day periods.

The catchment boundaries for the areas downstream of Coquitlam Dam are shown in Figure 4.6. To generate hydrographs for these catchment areas, the synthetic 1990 200-year inflow hydrograph to Coquitlam Lake was scaled to match the expected 200-year instantaneous peak flow for the respective catchment area. The relationship between peak flows and catchment area is shown in Figure 3.1. For the 50 km^2 Or Creek catchment and downstream area (Area A and C on Figure 4.6), the 200-year instantaneous peak discharge is 231 m^3/s. Section 5 discusses routing the upstream hydrograph through the Coquitlam reservoir and combining the resulting spill with downstream flows.

4.4. Approach using the 1995 flood event and concurrent downstream flows

The storm of November 22 to 26 was selected as it provided a scenario for consecutive peaks and high-sustained flows. The inflow hydrographs from the hourly records for this storm at gauge 08MH141 were adjusted to account for the entire catchment above the Coquitlam Dam and then scaled further to match the 200-year instantaneous peak and volumes from Table 3.2. As was done with the 1990 event, the 2 to 15 day hydrographs were developed such that the volumes are consistent with Table 3.2. The majority of the volume for the various durations was inserted before the maximum one-day volume requiring considerable distortion of the 1995 hydrograph as the actual 1995 event had very little flow volume immediately prior to the peak inflow.

The synthetic 200-year storms developed for 1990 and 1995 are similar regarding the timing of the peak daily flow, but the arbitrary allocation of volume surrounding the peak was different. As shown in Table 4.1, the synthetic 1995 storm consistently has the majority of volume prior to the peak as compared to the 1990 event that has a more even distribution of volume on either side of the peak for periods from one to three days. Figure 4.7 is a plot of cumulative volumes for the 1990 and 1995 storms that illustrates the large difference in flow volume at the time of the peak; 97 Mm^3 for 1995 versus 78 Mm^3 for 1990. The distribution of volume is critical to reservoir response and is discussed further in Section 5.

The actual 1990 and 1995 hydrographs both recorded about 50% of the 7-day volume prior to the peak flow. The synthetic 1990 hydrograph was more conservative than the recorded event with 64% of the 7-day inflow prior to the peak. The synthetic 1995 hydrograph was even more conservative with 78% of the 7-day inflow prior to the peak.

Table 4.1 – Volume Distributions

		Event Durations				
		1-Day	2-Day	3-Day	5-Day	7-Day
1990 Synthetic Storm	% of Volume Before Peak	48	61	50	58	64
	% of Volume After Peak	52	39	50	42	36
1995 Synthetic Storm	% of Volume Before Peak	72	79	79	76	78
	% of Volume After Peak	28	21	21	24	22

The downstream flows were calculated by scaling up the recorded 1995 hydrograph from gauge 08MH0002 to match the 200-year flood peak. As mentioned, Figure 3.1 provides a 200-year instantaneous peak flow of 231 m^3/s for catchments A and C (Figure 4.6). Section 5 discusses reservoir routing and calculating the resulting downstream 200-year water surface profile for the Coquitlam River.

5. COQUITLAM RESERVOIR MODELLING

5.1. Reservoir Operation Modelling

BC Hydro has carried out reservoir operation modelling as part of the work on the Coquitlam River Water Use Plan. BC Hydro's optimization model (AMPL) was used to investigate future operation of the Coquitlam Reservoir under GVRD maximized withdrawals and target fish flows. The AMPL model is integrated with the other operations of BC Hydro and therefore provides an overall picture of how the Coquitlam Reservoir fits into the BC Hydro operating system. Target elevations for power generation were established in the model runs that optimize operations, minimize spill and meet the GVRD water supply requirements. However, the model uses a "perfect" five-day forecast and is therefore somewhat idealistic.

Following discussions with BC Hydro and inspection of the AMPL model outputs, it was concluded that the model outputs of water levels provide the most realistic simulation available of future reservoir operations under the assumed working scenarios. BC Hydro agreed to permit use of these model runs for establishing initial design water levels in the reservoir for flood routing.

Following a review of the model outputs and design criteria, it was concluded that the STP5 simulation would be used for this study. The STP5 simulation assumes maximum projected GVRD withdrawals in the summer months that are not expected to occur for at least another 20 years. In order that these demands can be met, the reservoir operating plan cuts back on power generation in the late winter and spring to ensure that the reservoir is full at the beginning of the summer period. This simulation resulted in the highest reservoir water levels in the winter and thus could be considered a "worst-case" operating scenario. We considered this appropriate to counter the potential concern that the operation scenario is idealistic. Output water levels from the STP5 runs are shown in Figure 5.1.

The reservoir elevations are influenced significantly by the monthly target elevations for power, which are listed in Table 5.1. When the reservoir reaches these elevations, BC Hydro increases the power flows to maintain the reservoir at the target elevation. If the reservoir continues to increase, power flows are set to maximum until the reservoir level drops to the target elevation. It can be seen in Table 5.1 that the target reservoir elevation is relatively low from October to January and then allowed to rise to a maximum in June.

Table 5.1: Monthly target elevations for power generation

Month	Target elevation (m)
Oct	145
Nov	145.5
Dec	147.5
Jan	148
Feb	148.5
Mar	149
Apr	150
May	150.5
Jun	153
Jul	152
Aug	151
Sep	149

The output water levels in Figure 5.1 were analyzed for high inflow periods to determine reservoir level duration curves prior to high inflow periods. A number of high inflow periods were examined including 1-day, 2-day, 3-day, 5-day, 7-day, 15-day, 30-day, 60-day, 80, day, 90-day, 100-day and 110-day periods. Using the 7-day period as an example, the methodology was as follows:

1. The highest 7-day inflow periods for each year of the 39 years of record were identified.
2. The reservoir water level on the day before each 7-day high inflow period was tabulated producing 39 water levels.
3. The water levels were analyzed using a statistical histogram program to produce the probability of exceedance for any given water level.
4. Duration curves showing the percent of time the reservoir water level is equalled or exceeded were plotted. The duration curves are shown in Figure 5.2.

5.2. Reservoir Flood Modelling

A reservoir simulation model was developed using VBA (Visual Basic Applications) for Excel. The model incorporates the operating rules for flood operation provided by BC Hydro, which are as follows:

1. Above 153 m the power tunnel should be operating at capacity.
2. Above 153.86 m (the buffer) the low level outlet should be opened to return the reservoir elevation below the buffer, if possible. If the reservoir is rapidly approaching the buffer elevation the low level outlet would be opened one day prior.

3. Above 154.86 m - the weir crest - the reservoir will start free spilling and the discharge of the low level outlet is transferred to the spillway.

The Coquitlam-Buntzen Project Water Use Plan (BC Hydro, 2004) indicates that the reservoir may be operated above the flood buffer elevation during the period April 1 to September 30. Nevertheless it is understood that the target elevations for power generation (Table 5.1) will be maintained and therefore the reservoir level duration curves (Figure 5.2) will remain valid. However, in the event that the reservoir rises above the buffer elevation in the spring/summer, a contingency plan should be put in place by BC Hydro to restore the flood buffer if a summer storm is forecast.

BC Hydro provided rating curves for the key reservoir operating components including the power tunnel, low level outlet and fisheries release valves. The GVRD water supply outflow was conservatively assumed to be zero for modelling during a very wet period.

The flood inflow hydrographs for durations of 1 to 110 days were routed through the reservoir using the reservoir flood routing model. The operating rules listed above were followed for each duration flood modelled. In addition, for the long duration floods, the power tunnel was turned on at elevation 148 m in accordance with the target power elevations established by BC Hydro in mid-winter for model run STP5 as shown in Table 5.1.

The initial reservoir elevation prior to the onset of each flood period was derived from the reservoir level duration curves (Figure 5.2), which were developed to represent the conditions prior to high inflow periods. For a given flood period, the 200-year multi-day flood hydrograph was combined with the initial reservoir elevation corresponding to a 50% exceedance level. By combining the 200-year inflow flood with the average initial reservoir level that has occurred prior to major inflows, the outflow probability of the flood for the 200-year return period is maintained. Figure 5.3 shows the 50% probability values for flood durations from one to 110 days for reservoir levels prior to high inflow periods.

It was determined from the reservoir modelling that the 110-day event provides the most conservative estimate on reservoir outflows and was therefore used for further downstream flood analysis.

6. SENSITIVITY ANALYSIS

6.1. Analysis using synthetic 1990 flood

The synthetic 1990 reservoir inflow hydrograph, as described in Section 4.3, was routed through the Coquitlam Reservoir using the operating rules as described in Section 5.2. Figure 6.1 displays the reservoir inflow, outflow, and water level series for the peak period of the 110-day storm event using the 50% probability value of 143.3 m for the starting reservoir water level. The reservoir spillway hydrograph was combined with the downstream hydrograph for areas A and C as shown in Figure 6.2. It was assumed that the peak of the downstream hydrograph occurred simultaneously with the peak of the reservoir inflow hydrograph.

The combined flows result in a 200-year instantaneous peak flow of 453 m^3/s at the CP rail bridge and a peak daily flow of 377 m^3/s. These values are slightly higher than the discharges derived in the WMC March 2003 report because opening of the low level outlet one day prior to reaching the buffer level (based on forecasting) was not included.

6.2. Analysis using synthetic 1995 event

The inflow hydrograph derived for the 1995 event was routed through the reservoir using the 110-day period and 50% probability reservoir starting level of 143.3 m. The peak period of the reservoir routing is shown in Figure 6.3. The spillway outflow hydrograph was combined with the concurrent 200-year downstream flows derived in Section 4.4, as shown in Figure 6.4.

The combined flows produce a 200-year instantaneous peak flow of 632 m^3/s at the CP rail bridge and a peak daily flow of 448 m^3/s. These values are higher than the analysis using the 1990 flood event because of the much higher spillway outflows and reduced delay in the peak of the spillway outflow. It was found that adopting a more conservative shape for the inflow hydrograph, specifically distributing more flow prior to the one-day peak, raised reservoir levels above the spillway crest prior to onset of the peak daily inflow. As discussed in Section 4.4 and illustrated in Figure 4.7, the synthetic 1995 200-year storm has 19 Mm^3 (24%) more inflow volume prior to the peak than the 1990 synthetic 200-year storm.

In the synthetic 1990 event the reservoir level is only just above the buffer level prior to the onset of the peak daily inflow. In the synthetic 1995 event the reservoir is above the spillway crest level prior to the onset of the peak daily inflow. The contribution of the downstream tributaries for the 1995 event was about 230m^3/s at the peak of the downstream flood, which occurs at a spillway outflow of about 400 m^3/s, prior to the peak spillway outflow of 425 m^3/s.

6.3. Analysis of the October 2003 flood event.

Preliminary data for the October 2003 event were available from the real-time data output on the Water Survey Canada website. The gauged discharge upstream of the reservoir was scaled to represent the inflow from the entire Coquitlam watershed above the dam. The flood was routed though the reservoir using a starting reservoir level at the buffer elevation of 153.86 metres. The routed outflow hydrograph was combined with the downstream flows and the results are shown in Figure 6.5. The combined flows indicate a peak instantaneous flow of 317 m^3/s at the CP rail bridge and a peak daily flow of 231 m^3/s. The calculated instantaneous flow estimate is higher than the recorded discharge (180 m^3/s) because there was no spill from the Coquitlam Dam during the actual event.

To investigate this flood as a potential 200-year event, the upstream and downstream hydrographs were scaled-up to represent 200-year instantaneous peaks. The same starting elevation of 153.86 metres was used for reservoir routing. The resulting discharges were 407 m^3/s for the reservoir spillway and 541 m^3/s at the CP Rail Bridge, as shown in Figure 6.6. The contribution from the downstream tributaries at the peak was about 60 m^3/s larger than the contribution for the synthetic 1990 and 1995 events. The synthetic 2003 200-year event included a one-day volume that is 81% of the 200-year one-day volume and a two-day volume that is 111% of the 200-year two-day volume. The calculated discharge at the CP Rail Bridge of 541 m^3/s results in an event that is larger than the synthetic 1990 200-year event but smaller than the synthetic 1995 200-year storm.

A starting reservoir level at 153.86 metres is very conservative. For a two-day storm event, Figure 5.2 suggests the average reservoir level prior to a two-day event is just less than 146 metres and a maximum starting level of 150 metres 90% of the time. Lowering the reservoir starting elevation would both delay and reduce the spillway outflow and subsequently reduce downstream discharges.

6.4. Effect of using upper bound estimates

The analyses described in sections 6.1 and 6.2 were repeated using the upper value from the 95% confidence limits for the peak daily inflow to the Coquitlam Reservoir.

The analyses were also repeated using a Coquitlam Reservoir starting level of 146.3m for the 110-day event corresponding to the level that would only be exceeded 10% of the time (Figure 5.2).

It was found that using the 95% confidence limits increased the peak downstream flows about 25%. However, the analyses were found to be not very sensitive to the starting water level elevations as shown in Figures 6.7 and 6.8. This is because the power tunnel flows start earlier with the higher starting reservoir elevation and are sufficiently large to prevent the reservoir rising at the early stages of the 110-day hydrograph.

6.5. Water surface profile sensitivity

Models unsteady flows

The HEC-RAS model developed by Northwest Hydraulics, 2002 was used for modelling the Coquitlam River. The 12 km long study reach extended from the mouth of the Coquitlam River to approximately 5 km below the Coquitlam Dam. The model was based on 53 cross-sections provided by BC Hydro surveyed between October and December 2000 (Bland, 2000). The model was originally run in steady-state mode using the peak daily flows from the 1988 report listed in Table 2.1.

HEC-RAS version 3.1 was released by the US Army Corps of Engineers in January 2003 and was used for this study.

A survey of high water marks was carried out for this study for the peak flow of October 16 2003. The high water marks were flagged on October 17 and surveyed the following week. Appendix A summarizes the data. There is some uncertainty regarding the magnitude of the peak flow on October 16, 2003. The highest flow that has actually been measured at the WSC gauge was 132 m^3/s on August 30 1991 at a gauge height of 6.76 m geodetic. The current rating curve used by WSC is dependent on this value for flows above 60 m^3/s. Recent bed aggradation documented in Northwest Hydraulics 2002 has resulted in higher water levels for the same flow but the WSC rating curve has not been updated to reflect this. No measurements were made by WSC during the October 2003 flood event.

We used the HEC-RAS model to extend the rating curve and concluded that the peak flow on October 16 2003 was about 130 m^3/s. The HEC-RAS model was calibrated using this flow value and an excellent fit was found for most of the Coquitlam River recorded high water marks. Higher Manning's n values would be required to match the recorded water surface profile upstream of the Lougheed Highway Bridge and it was concluded that aggradation during the 2003 flood event had resulted in higher bed levels. The Manning's n values were set so that they were consistent for the entire model reach.

Northwest Hydraulics had increased roughness coefficients in the floodplain from 0.08 to 0.12 to reflect the presence of dense bank and floodplain vegetation observed at the beginning of November. However, the analysis presented in Section 5 shows that the reservoir is drawn down in the summer and refills in the winter. The reservoir is therefore unlikely to spill in November as the reservoir is still filling and the reservoir is being used for maximum power generation as there is still plenty of time to refill the reservoir before next summer. Furthermore, the critical inflow flood duration is 110 days of high inflows which means that the reservoir peak outflow would occur, at the earliest in December. Therefore the roughness factors were not increased to reflect dense summer-type growth.

The roughness factors used for the HEC-RAS model were based on values used in the literature, the model calibration and experience on other rivers. The reference, US Army Corps of Engineers (1994) provides a composite of information drawn from a number of references on selection of Manning's n.

The roughness factors used for the modelling were:

- Channel 0.032 –0.048
- Overbank 0.08

Sections upstream and downstream of the CP Rail Bridge were selected for comparison of water surface profile elevations. Section 13.25 is about 600 m downstream of the CP Rail Bridge and Section 15.5 is about 200 m upstream of the bridge. Water surface profile elevations for peak instantaneous and peak daily flows were computed for all developed scenarios. For this comparison, the model included a blockage at the CP Rail Bridge that extended across the channel below the bottom of the bridge deck and no blockages at the Kingsway Bridge. The results are presented in Table 6.1. For the dyke elevations freeboard of 0.3 m was added to the peak instantaneous flow elevations and 0.6 m to the peak daily flow elevations.

It is apparent from Table 6.1 that the peak instantaneous flows govern the dyke elevations as these discharges contribute more than 0.3 m of increased elevation compared with the peak daily flows.

The increase in discharge using the 95% confidence limits raised water levels significantly upstream of the bridge from 0.7 m to 1.1 m and only slightly downstream from 0.3 m to 0.5 m.

The sensitivity to increases in bed levels following sediment aggradation was also investigated. It was found that an increase in bed levels increased the water surface profile by about 0.25 m. Figure 6.9 summarizes the results of the water surface profile sensitivity analysis in the critical reach near downtown Port Coquitlam.

Table 6.1 - Water Surface Profile Sensitivity

			Peak Instantaneous					Peak Daily				
Scenario	Year	Reservoir Starting El.	Flow (m3/s)	WSP El XS-15.5	Dyke El.	WSP El XS-13.25	Dyke El.	Flow (m3/s)	WSP El XS-15.5	Dyke El.	WSP El XS-13.25	Dyke El.
200-Year	1990	143.3	453	10.4	10.7	8.6	8.9	377	9.9	10.5	8.2	8.8
200-Year	1995	143.3	632	12.0	12.3	9.3	9.6	448	10.4	11.0	8.5	9.1
200-Year	2003	153.86	541	11.2	11.5	8.9	9.2	409	10.0	10.6	8.4	9
200-yr 95% Confidence	1990	143.3	572	11.5	11.8	9.1	9.4	480	10.9	11.5	8.7	9.3
200-yr 95% Confidence	1995	143.3	786	12.7	13.0	9.6	9.9	550	11.3	11.9	9.0	9.6
10% Starting Level	1990	146.0	455	10.4	10.7	8.6	8.9	379	9.9	10.5	8.2	8.8
10% Starting Level	1995	146.0	640	12.1	12.4	9.3	9.6	451	10.4	11	8.6	9.2

7. HYDROLOGIC DESIGN CRITERIA

7.1. Summary of hydrologic analysis

The sensitivity of the peak reservoir outflow magnitude and shape to the inflow magnitude and shape has been investigated by creation of synthetic 200-year hydrographs based on the recorded inflow events in 1990 and 1995. Each hydrograph included the 200-year inflow volumes for all durations from one to 110 days. It was found that the key variable is the proportion of flow volume allocated prior to the occurrence of the peak inflow.

The actual 1990 and 1995 hydrographs both recorded about 50% of the 7-day volume prior to the peak flow. The synthetic 1990 hydrograph was more conservative than the recorded event with 64% of the 7-day inflow prior to the peak. The synthetic 1995 hydrograph was even more conservative with 78% of the 7-day inflow prior to the peak.

The timing of the occurrence of downstream tributary flows with reservoir inflows was investigated by comparing recorded hydrographs upstream and downstream of the reservoir. It was found that the reservoir inflow peaks essentially occurred at the same time as the downstream tributary peaks. This is largely because the major downstream tributary of Or Creek has similar watershed characteristics as the reservoir tributary watersheds.

The 200-year downstream flows were derived by scaling up recorded hydrographs. For the synthetic 1990 event the downstream hydrographs peaks were set at the same time as the reservoir inflow peaks. For the synthetic 1995 and 2003 events the timing was based on recorded flows.

The synthetic 1990 event produced a peak flow at the CP rail bridge of 453 m^3/s and the synthetic 1995 event produced a peak flow of 632 m^3/s. In the synthetic 1990 event the reservoir level was just above the buffer level at the onset of the peak daily inflow. In the synthetic 1995 event the reservoir was already spilling prior to the onset of the peak daily inflow.

The 2003 flood event was also investigated using the recorded inflows and recorded downstream flows. It was found that if the reservoir had been at the buffer elevation prior to the event the peak flow in the Coquitlam River at the CP rail bridge would have been 317 m^3/s rather than the 180 m^3/s recorded. The 2003 hydrographs were also scaled up to represent a 200-year flood event for both upstream and downstream. With a starting reservoir elevation at the buffer elevation the peak flow at the CP rail bridge would have been 541 m^3/s.

Using the 95% confidence upper limit for the peak daily flow volume the peak instantaneous flow at the CP rail bridge would be 572 m^3/s for the synthetic 1990 event and 786 m^3/s for the 1995 synthetic event. The additional data used in this study from the Pollard report indicated that the extended 86 years of record did not result in an increase in the 200-year daily flow estimates. Furthermore the recorded values for the instantaneous and peak daily flows for the 1921 and 1961 floods were both less than the 200-year flood estimate from the original 44 years of record and the regional analysis. Therefore using the upper 95% confidence limit for the design is not recommended, as the frequency analysis using the 44-year period of quality controlled data appears reasonable.

It was found that the peak reservoir outflows were not sensitive to reservoir starting levels for the 110-day hydrographs. This was because the power outflows can maintain reservoir levels at the relatively low inflows at the initial stages of the 110-day hydrographs.

7.2. Recommended hydrologic design criteria

It is recommended that the synthetic 1995 hydrograph be used to generate the design flows in the Coquitlam River. This synthetic hydrograph is more conservative than the synthetic 1990 event used in the Water Management Consultants March 2003 report. The volume distribution is significantly more conservative than the recorded event.

It is recommended that the 50% probability starting water level be used with the 1995 synthetic hydrograph to maintain a 200-year probability event criterion.

It is recommended that the peak instantaneous flows be used for dike design and floodplain mapping with 0.3 m of freeboard. This gives a higher dike design level than using the peak daily flow and a freeboard of 0.6 m. Where levels are higher using daily flows plus 0.6 m of freeboard it is recommended to use the higher value.

The recommended peak instantaneous design flow at the CP Rail Bridge is therefore 632 m^3/s.

For tributary inflows from Scott Creek and downstream of Scott Creek, it is recommended to use the 200-year peak flows from the regional analysis and the shape of the recorded hydrographs from the gauge at the CP Rail Bridge. The timing of these tributary inflow peaks would coincide with the other inflow hydrographs as discussed in this report.

8. DESIGN FLOOD LEVELS

8.1. Boundary Conditions

Table 8.1 shows the peak daily flows from the 1988 report and the peak flows derived for this study.

Table 8.1: 200-year peak flows in the Coquitlam River in m^3/s

	Peak daily flow from 1988 report	Peak daily flow used in this report	Peak instantaneous flow used in this report
At gauge 08MH002	573	448	632
At confluence with Fraser River	640	577	777

The key assumptions used in deriving the flows for this study are described in previous sections of this report and are summarized as follows:

- Peak instantaneous 200-year reservoir inflow were embedded in multi-day hydrograph up to 110 days with inflow volumes for all durations corresponding to 200-year events
- The starting reservoir level was based on future Coquitlam Reservoir operations using the BC Hydro STP5 run and reservoir level corresponding to 50% probability of occurrence
- A synthetic 1995 hydrograph was used for the inflow volume sequence and to synchronize inflows from downstream tributaries
- Downstream hydrographs were based on scaling up the 1995 hydrograph recorded at the CP Rail bridge using estimated 200-year flow peaks from the regional analysis.

The peak daily flows from the 1988 report were derived on the basis of the analysis carried out for the Water Investigations Branch (1976) as part of the technical studies for the Coquitlam River Water Management Plan. The 1976 report used a unit hydrograph technique based on the records from the July 1972 storm event. In that storm the recorded unit peak runoff from the downstream areas, including Or Creek, was twice as high as the unit peak runoff over the Coquitlam Watershed above the dam. The highest rainfall intensities were centred on the downstream areas. The hydrological analysis in the 1976 report was based on factoring up the 1972 hydrographs using a ratio of the 200-year rainfall to the rainfall recorded at the Coquitlam Lake station. This resulted in a very high 200-year unit peak runoff estimate for the downstream area of 5.7 $m^3/s/km^2$ compared with 2.7 $m^3/s/km^2$ upstream of the dam.

The downstream boundary condition for winter flows was that used in Associated Engineering Services (1988) and Northwest Hydraulics (2002). The level of 2.13 m roughly represents a five to 10-year return period maximum October to January water level on the Fraser River.

The downstream boundary used for the summer flows was a 200-year Fraser River level of 4.37 m (0.1 m higher than the level used in Northwest Hydraulics (2002). The summer flows used were 228 m^3/s above Scott Creek and 255 m^3/s below Scott Creek, which were the flows, used in Northwest Hydraulics (2002) and Associated Engineering (1988).

Northwest Hydraulics included a log jam at the CP Rail bridge which increased water levels by about 0.36 m. The potential log jam at the CP Rail bridge is well-supported in Northwest Hydraulics (2002). The 200-year peak flow intersects the soffit of the bridge causing pressure flow. The log jam was assumed to be formed across the whole section about a metre below the bridge soffit.

We included the log jam at the CP Rail bridge in the model for this study for defining FCL. A second log jam at the Kingsway Bridge is less likely to form because of the higher localized velocities through this confined section and the material being collected at the upstream jam. Nevertheless, a partial log jam at the Kingsway Bridge could occur and a log jam was included in the model by blocking one span of the bridge between piers.

8.2. Design Water Surface Profile

The modelled winter and summer profiles are illustrated in Figure 8.1. The resulting peak water surface profile was slightly lower than the profile calculated by Northwest Hydraulics in 2002. Near the Fraser River, up to about 1500 m upstream of the Mary Hill Bypass Bridge, the summer FCL governs.

Flood Construction Levels (FCL) were derived by adding freeboard to modelled peak water levels depending on the method used to generate the inflow boundary condition to the model. As discussed in Section 3, the winter flood flows were developed from a regional flood frequency analysis using instantaneous peaks requiring the provincial standard of adding 0.3 m of freeboard to the estimated peak water surface profile. The summer flood flows were derived from daily peaks requiring, by provincial standards, 0.6 m of freeboard. The winter peak daily flood was also modelled and if the flood level plus 0.6 m was greater than the instantaneous level plus 0.3 m, the higher level governed. In general the instantaneous flow peak plus 0.3 m provided the higher FCL value.

The calculated FCL values are listed in Table 8.2 together with the results from previous studies. Table 8.2 includes the elevations of the right and left bank dikes based on information from the Northwest Hydraulics Report (2002). The table shows that the dikes are up to 1.61 m low in the Colony Farm area and up to 1.5 m low between Pitt River Road and the CP Rail bridge. Immediately upstream of the CP Rail bridge, the dikes are up to 2.52 m too low as a result of the constriction at the bridge and the debris jams.

The base mapping used for this study was supplied in AutoCAD format by the City of Coquitlam and the City of Port Coquitlam with a contour interval of 2 m. Flood Construction Levels (FCL) were determined as described above. These water levels assume that the existing FRFCP Dikes, the left and right dikes upstream of Lougheed Highway and the Colony Farm Dikes contain the design flood. Floodplain limits were then determined by extending the FCL to high ground.

The set of floodplain maps are submitted as a separate document.

Table 8.2 - Comparison of Flood Construction Levels

River Cross-Section	Bridge Location	Previous 1976[1] FCL (m)	Previous 2002[1] FCL (m)	Revised 2003 FCL (m)	Revised 2004 FCL[a] (m)	Left (east) dike crest/bank elevation [1] (m)	Right (west) dike crest/bank elevation [1] (m)	Left (east) dike excess freeboard	Right (west) dike excess freeboard
0.5			5.0	5.0	5.0	1.5	2.6		
1	Maryhill By-pass	4.87	5.0	5.0	5.0	4.8	4.8		
1.25			5.0	5.0	5.0	4.8	4.8		
1.5			5.0	5.0	5.0	1.3	**4.9**		-0.1
1.75			5.0	5.0	5.0	**4.5**	**5.9**	-0.6	0.9
3			5.0	5.0	5.0	**4.5**	**3.9**	-0.5	-1.1
4			5.0	5.0	5.0	**4.4**	**3.9**	-0.7	-1.2
5		5.02	5.1	5.1	5.1	**4.0**	**4.1**	-1.1	-1.0
5.5			5.6	5.2	5.5	**4.1**	**4.3**	-1.4	-1.2
6			5.9	5.3	5.8	**4.2**	4.2	-1.6	
6.3		5.1	6.1	5.3	6.1	**5.9**	5.4	-0.2	
6.5			6.2	5.4	6.2	**5.9**	7.0	-0.2	
7		5.93	6.6	5.6	6.4	**6.1**	7.0	-0.3	
7.2			6.7	5.7	6.5	**6.5**	8.0	0.0	
8		6.09	6.7	5.7	6.5	**6.1**	8.0	-0.4	
9	Pitt River Road	6.24	6.7	5.7	6.4	6.0	6.0		
9.1			6.8	5.7	6.7	6.0	6.0		
9.5			7.1	5.9	7.1	**6.2**	10.0	-0.9	
10		6.54	7.3	6.3	7.2	**6.3**	10.0	-0.9	
11		7	7.7	6.9	7.5	**6.8**	8.0	-0.7	
11.1			7.6	6.9	7.5	**7.2**	8.0	-0.3	
11.25			8.1	7.1	7.9	**7.5**	8.0	-0.4	
12		7.15	8.4	7.1	8.2	**7.5**	**8.0**	-0.8	-0.2
12.5			8.5	7.2	8.3	**7.7**	**8.7**	-0.7	0.4
13		7.61	8.9	7.6	8.7	**7.9**	**8.6**	-0.8	0.0
13.25			9.8	8.5	9.6	**8.5**	**8.3**	-1.0	-1.3
13.5			10.6	9.3	10.4	**9.5**	**9.4**	-0.9	-1.0
13.75			10.7	9.4	10.5	**9.2**	**9.0**	-1.3	-1.5
14		8.83	10.3	9.5	10.5	11.0	11.0		
14.1			10.8	9.7	11.4	11.0	11.0		
14.9			12.2	9.8	11.7	11.5	11.5		
15	CP Rail	8.83	12.4	9.8	12.0	11.5	11.5		
15.1			12.9	9.9	12.6	11.5	11.5		
15.25			13.0	10.1	12.7	**10.2**	8.3	-2.5	
15.5			13.1	10.2	12.8	**10.3**	8.8	-2.5	
16		10.05	13.1	10.3	12.8	**11.7**	11.7	-1.1	
16.1			13.1	10.3	12.8	**11.8**	11.9	-0.9	
16.9			13.1	10.3	12.8	**11.9**	11.9	-0.9	
17	Lougheed Hwy	10.2	13.1	10.3	13.1	**11.9**	**12.0**	-1.2	-1.1
17.25			13.3	11.2	13.3	**13.0**	**13.3**	-0.4	0.0
17.5			13.8	12.1	13.8	**14.4**	**14.6**	0.6	0.9
17.75			14.2	12.8	14.2	**14.5**	**15.3**	0.3	1.1
18		15.23	15.2	14.6	15.2	16.8	17.4		
18.5			17.1	15.9	17.2	19.7	19.9		
19			21.4	20.6	21.2	27.0	22.0		
20			24.8	23.9	24.7	24.8	26.2		
21			28.3	27.4	28.3	27.9	28.7		
22			30.0	28.7	29.8	28.6	29.0		
23			30.1	29.0	29.9	29.7	30.4		
24			31.2	30.2	31.1	31.7	33.6		
25			35.1	34.8	34.9	38.0	37.4		
26			38.3	37.5	37.6	42.5	46.3		
27			39.8	38.5	39.6	42.1	39.1		
28			41.4	41.3	41.2	44.0	41.0		
29			43.4	42.7	43.3	44.0	43.1		
30			46.1	45.7	45.9	50.0	44.5		
31			48.1	47.3	47.9	50.0	48.0		
32			50.3	49.3	50.1	49.8	49.8		
33			52.3	51.3	51.7	55.7	55.0		
34			54.9	53.9	54.8	55.0	56.0		
35			57.0	56.2	56.8	57.9	62.3		
36			58.0	57.3	57.7	60.0	60.0		

[1]From Northwest Hydraulics (2002)

[a]Based on partial blockage at Kingway Bridge and without raising bed level

Highest of Instantaneous peak +0.3 and Daily Peak +0.6

14.40 Dyke crest

9. FLOOD MITIGATION ALTERNATIVES

The flood mitigation alternatives that were considered included:

- Channel excavation/sediment removal
- Modifying the CP Rail and Kingsway bridges
- Raising existing dikes
- Allocating additional flood storage in Coquitlam Reservoir
- Increasing the low level outlet spill capacity at Coquitlam Reservoir

9.1. Channel excavation / sediment removal

Sediment removal was considered as a flood control alternative. From the modelling analysis it was found that a major excavation would be required to make a significant difference to the dike design crest level.

For example, an excavation of about one metre below the lowest point of the existing channel was modelled including the removal of gravel bars. The excavation analysis extended from the bridge at Pitt River Road to the Kingsway Bridge with an average depth of excavation of 2.4 m. The HEC-RAS analysis indicated that the dike design levels would be reduced from the Pitt River Road Bridge to the Lougheed Highway Bridge by an average of 0.46 m with the greatest reduction of 0.76 m just downstream of the Kingsway Bridge. Therefore the model analysis indicates that for an average 2.4 m excavation, the dike system would still require significant raising to meet the required flood protection level. The total quantity of excavated material would be about 175,000 m^3. The unit costs for excavation include potential cost recovery by selling material. Details of the cost breakdown are provided in Appendix B. The capital costs for the channel excavation option were found to be $3.4 million higher than the option that only included dike raising for the I km of dikes considered in this study. When all the dikes are considered the difference in costs would be less.

In addition to the capital costs, there would be environmental (i.e. assessment, mitigation, compensation) and annual channel maintenance and monitoring costs.

Fisheries and Oceans Canada (DFO) has commented on the channel excavation option as follows:

> "Pursuant to the federal Fisheries Act, DFO has developed a policy that applies to all projects and activities, large and small, in or near the water, that could "alter, disrupt or destroy" fish habitats, by chemical, physical or biological means; thereby potentially undermining the economic, employment and other benefits that flow from Canada's fisheries resources. More details of the policy may be seen on the web, at http://www.dfo-mpo.gc.ca/canwaters-eauxcan/infocentre/legislation-lois/policies/fhm-policy/index_e.asp
>
> The federal Fisheries Act defines "fish habitats" as those parts of the environment "on which fish depend, directly or indirectly, in order to carry out their life processes". The Act also defines "fish" to include all the life stages of "fish, shellfish, crustaceans, marine animals and marine plants".
>
> DFO considers that sediment removal, and associated activities, can "alter, disrupt or destroy" fish habitat. Works that "alter, disrupt or destroy" fish habitat cannot be legally undertaken without an authorization pursuant to subsection 35(2) of the federal Fisheries Act. Assessments of such projects follow the "Policy for the management of fish habitat" (the policy), and DFO's fish habitat conservation and protection guidelines; and must also adhere to the Canadian Environmental Assessment Act (CEAA) and the Species at Risk Act (SARA). For works authorized pursuant to subsection 35(2) of the Fisheries Act, compensation or mitigation would likely be required to satisfy DFO's requirements for no net-loss of fish habitat.
>
> DFO supports works, including sediment management, that address public safety issues. Following the policy, and no net loss of habitat guidelines, it has been DFO practice to support the removal of sediment from river channels, if such works address flood, erosion, or navigation safety issues; and if other solutions are not available. Some other long-term solutions that should be investigated in the case of the Coquitlam River may include:
>
> - providing a floodway through Colony Farm or reactivating other side channels
> - providing an early flow release capability at a new BC Hydro dam
> - ensuring that bridges do not unreasonably restrict flood flows
> - vegetation management within the floodway"

As channel excavation does not appear to be cost-effective and environmental approvals would be difficult to obtain, it is recommended that no further analysis be undertaken with this alternative.

9.2. Modifying the CP Rail and Kingsway bridges

The CP Rail and Kingsway bridges form a significant constriction in the Coquitlam River which raise upstream water levels during the design flood event. The feasibility of modifying the bridges was investigated using the HEC-RAS model. The bridges were widened in the model to the full width of the Coquitlam River channel upstream of the CP Rail bridge. This represents an increased span of 40 m which is 160% of the existing span width. The channel immediately upstream and downstream of the bridges was also widened and excavated in the model. The total quantity of excavation would be about 45,000 m^3.

Low! Should at least 1.5 m

Even with these modifications the 200-year instantaneous peak water surface profile was 0.3 m above the soffit of the CP Rail bridge. For debris clearance under design conditions, at least one metre of freeboard should be allocated. Therefore, the bridges would have to be raised about 1.3 m in addition to being widened. As maximum rail grades are about 2%, it would require extensive raising of the CP rail bed east and west of the bridge possibly including part of a large railway marshalling yard east of the bridge.

The following comments were forwarded by Jan Kubik of CP Rail:

The CPR Coquitlam yard is saucer shaped with the high points of the saucer rim located at the bridges at the Pitt & Coquitlam rivers. When switching railway cars at the west end of the yard up to and over the Coquitlam River Bridge our switching locomotives are moving the maximum tonnage of cars that the grade leading up to the bridge permits. Raising the CPR Coquitlam River Bridge would adversely impact our operations by reducing the tonnage our switching locomotives are able to move. This would be unacceptable to CPR. Mitigation would require either more yard engines or raising the elevation of the yard so that the grade up to the Coquitlam River Bridge is no more than it is at present. The elevation of the track at the Coquitlam River Bridge is about 20 ft higher than the middle of the yard (the bottom of the saucer) and this elevation difference occurs over 6000 ft. To put a monetary value to the mitigation required if the CPR Coquitlam River Bridge were to be raised would require a lot of study. It is likely a detailed study would rule out raising the CPR bridge as an economic option.

Since the modifications to the railway bridge and Kingsway Bridge would entail significant costs, it is recommended that no further analysis be undertaken with this alternative.

9.3. Raising existing dikes

Compared with the above alternatives, raising the existing dikes provides a cost-effective flood mitigation alternative with less environmental impact. Preliminary designs for dike raising for one kilometre of dikes are presented in the next section together with estimates for raising all FRFCP Dikes on the Coquitlam River.

9.4. Allocating additional flood storage in Coquitlam Reservoir

The potential for enhancing the flood control operation of Coquitlam Reservoir was considered as a flood mitigation alternative. Currently a flood control buffer of 1 m below spillway level is allocated in the reservoir. When the reservoir elevation is above the buffer elevation, the power flows and the flows from the low level outlet are increased to maximum.

One alternative would be to lower the flood control buffer elevation and thereby provide increased storage for flood control. However, the analysis presented in Section 5 of this report demonstrates that in the design condition, the buffer is largely ineffective. There is insufficient spill capacity in the reservoir to maintain reservoir levels at the buffer elevation when large multi-day inflow events occur.

If the flood control potential of the reservoir were to be increased, the reservoir would have to be operated at lower levels throughout the winter and spring by decreasing the power target elevations (Table 5.1). However, future plans (10 to 20 years) for Coquitlam Reservoir are to increase the water supply use of the reservoir and decrease the power use. Target elevations for future water supply would be similar to current target power elevations. Allocating additional flood control storage would significantly constrain this potential future use and compensation would likely have to be paid annually to BC Hydro/GVRD for the loss of storage capacity. Operating the reservoir at lower levels would mean that refill in the spring might be compromised leaving inadequate storage for water supply over the critical summer months. Based on the constraints with the physical characteristics of the current dam and future water supply needs it is recommended that no further analysis be undertaken with this alternative.

9.5. Increasing the low level outlet capacity

As noted in Section 4.2, the Coquitlam Reservoir was not originally designed as a flood control reservoir and has limited outlet capacity, compared with flood inflows, for maintaining the buffer elevation. Increasing the outlet capacity has been considered at a conceptual design level by BC Hydro.

A flow control valve with capacity for 200 m^3/s could be installed at the inlet to the existing diversion tunnel that was constructed at the time of construction of Coquitlam Dam. The inlet currently comprises three large diameter (3 m wide by 5 m high) concrete diversion culverts that were plugged with concrete after dam construction. The current low-level outlets are located on top of the concrete diversion culverts with a control tower above the outlets. The conceptual design scheme would involve strengthening the middle concrete diversion culvert, removing the concrete plug and installing a permanent maintenance gate and a control valve. According to BC Hydro an initial rough estimate suggests that the approximate construction cost may be in the range of $3 to $5 million. However this figure would need to be confirmed through a preliminary design study.

BC Hydro has advised that:

We concluded that the concept appears to be feasible but further work during a preliminary design study is required to develop a cost estimate suitable for financial authorization. However, we noted that the reliability of our preliminary design estimate would be affected by the limited information that is available for this 80 year old structure and by our inability to fully inspect its condition during preliminary design because of reservoir operation requirements, fish water releases and the accumulated sediment that has partially buried the upstream side of the tower. Engineering expects preliminary design will cost about $170,000 in engineering fees. This estimate does not include costs that may be required or incurred directly by BC Hydro's Generation, Legal, Aboriginal Relations, Community Relations or for environmental assessments. At the end of the preliminary design phase we would provide a brief report summarizing our assessment, the cost estimate to construct the modifications to the low-level outlets and proposed construction schedule, together with an assessment of the engineering risks involved in implementing the modifications to the low-level outlets.

This cost estimate and work scope does not include the work necessary to confirm that 200 m^3/s can be safely passed downstream nor the costs required for reviewing and confirming the feasibility of providing the 200 m^3/s discharge with respect to:

- Reservoir operations, including any associated economic impacts
- First Nations cemetery inundation/impacts
- Downstream property impacts, erosion
- Downstream debris management
- Downstream flood impacts
- Increased risk to public safety
- Emergency communications/preparedness during high flow events
- Liability issues and concerns
- More detailed engineering and social costs for dike upgrades
- Review of BC Hydro's water use plan if, or when, 200 m^3/s discharge can be implemented safely

We repeated the Coquitlam Reservoir routing with the 200-year design storm based on the synthetic 1995 hydrograph using an increased low level outlet capacity of 200 m^3/s using the existing BC Hydro reservoir operating rule that provides a buffer of 1 m below the spillway crest. As shown in Figure 9.1, the increased outlet capacity provides the capability to hold the reservoir at the buffer elevation for a longer duration so the peak spillway outflow is only 239 m^3/s compared with 425 m^3/s without the increased low-level outlet capacity. The peak instantaneous flow at the CP Rail Bridge was found to be 376 m^3/s compared with 632 m^3/s without the increased capacity. The peak instantaneous flow was used in the HEC-RAS model to derive a water surface profile. The corresponding FCL values are shown in Table 9.1, which indicate that this option would only require relatively minor raising of the FRFCP Dikes. However, the Colony Farm dikes, downstream of Section 6.0, would still require raising. Costs for dike raising for this option are discussed in Section 10.7.

This option significantly reduces the required dike upgrading and in some locations eliminates the need for dike upgrading. The Equipment Landing Area for debris control at the CP Rail bridge would still be recommended as the water surface profile intersects the soffit of the bridge. However the extent of the required works would be reduced as the FCL would be lower. This option also reduces the FCL in the floodplain area behind the dikes which would reduce the floodproofing requirement.

This alternative would require no changes to the current reservoir operating rules. The existing operating rule to maintain the buffer elevation would remain in place. When the reservoir is being held at the buffer elevation, the policy is to "pass inflows" and the releases are equal to or less than the natural inflows. The increased low level outlet capacity would allow BC Hydro to maintain the reservoir at the buffer elevation more effectively. It would also increase the safety of operating the reservoir above the buffer elevation in the spring/summer period. If a major storm is forecast, the buffer elevation could be more easily restored with an increased low level outlet capacity.

In view of the advantages of this option, it is recommended that further analysis be undertaken. The assessment should include the sensitivity of the peak outflows to changes in the buffer elevation. A small increase in the size of the flood buffer could reduce the magnitude and frequency of flood outflows without significantly compromising power and water supply benefits.

Table 9.1 - Comparison of Flood Construction Levels with increased low level outlet capacity at Coquitlam Dam

Model River Cross-Section	AE River Stations	Bridge Location	Previous 1976[1] FCL (m)	Previous 2002[1] FCL (m)	Revised 2003 FCL (m)	Revised 2004 with 200m3/s outlet FCLa (m)	Left (east) dike crest/bank elevation [1] (m)	Right (west) dike crest/bank elevation [1] (m)	Left (east) dike excess freeboard	Right (west) dike excess freeboard
0.5				5.0	5.0	5.0	1.5	2.6	-3.5	-2.4
1		Maryhill By-pass	4.87	5.0	5.0	5.0	4.8	4.8	-0.2	-0.2
1.25				5.0	5.0	5.0	4.8	4.8	-0.2	-0.2
1.5				5.0	5.0	5.0	1.3	**4.9**	-3.7	-0.1
1.75				5.0	5.0	5.0	**4.5**	**5.9**	-0.6	0.9
3				5.0	5.0	5.0	**4.5**	**3.9**	-0.5	-1.1
4				5.0	5.0	5.0	**4.4**	**3.9**	-0.7	-1.2
5			5.02	5.1	5.1	5.1	**4.0**	**4.1**	-1.1	-1.0
5.5				5.6	5.2	5.2	**4.1**	**4.3**	-1.1	-0.9
6				5.9	5.3	5.3	**4.2**	4.2	-1.0	-1.1
6.3			5.1	6.1	5.3	5.4	**5.9**	5.4	0.5	0.0
6.5				6.2	5.4	5.5	**5.9**	7.0	0.4	1.5
7			5.93	6.6	5.6	5.8	**6.1**	7.0	0.3	1.2
7.2				6.7	5.7	5.9	**6.5**	8.0	0.7	2.2
8			6.09	6.7	5.7	5.9	**6.1**	8.0	0.2	2.1
9		Pitt River Road	6.24	6.7	5.7	5.8	6.0	6.0	0.2	0.2
9.1				6.8	5.7	5.9	6.0	6.0	0.1	0.1
9.5				7.1	5.9	6.3	**6.2**	10.0	-0.1	3.7
10			6.54	7.3	6.3	6.5	**6.3**	10.0	-0.2	3.5
11			7	7.7	6.9	7.0	**6.8**	8.0	-0.2	1.0
11.1				7.6	6.9	7.0	**7.2**	8.0	0.2	1.0
11.25				8.1	7.1	7.3	**7.5**	8.0	0.1	0.7
12			7.15	8.4	7.1	7.6	**7.5**	**8.0**	-0.1	0.4
12.5				8.5	7.2	7.7	**7.7**	**8.7**	0.0	1.1
13			7.61	8.9	7.6	7.9	**7.9**	**8.6**	0.0	0.7
13.25	6+860			9.8	8.5	8.5	**8.5**	**8.3**	0.0	-0.3
13.5	7+217			10.6	9.3	9.2	**9.5**	**9.4**	0.3	0.1
13.75	7+300			10.7	9.4	9.3	**9.2**	**9.0**	-0.2	-0.4
14	7+389		8.83	10.3	9.5	9.4	11.0	11.0	1.6	1.6
14.1				10.8	9.7	10.0	11.0	11.0	1.1	1.1
14.9				12.2	9.8	10.1	11.5	11.5	1.4	1.4
15		CP Rail	8.83	12.4	9.8	10.2	11.5	11.5	1.4	1.4
15.1	7+470			12.9	9.9	10.4	11.5	11.5	1.2	1.2
15.25	7+589			13.0	10.1	10.5	**10.2**	8.3	-0.3	-2.2
15.5	7+780			13.1	10.2	10.6	**10.3**	8.8	-0.3	-1.8
16	7+860		10.05	13.1	10.3	10.7	**11.7**	11.7	1.0	1.1
16.1				13.1	10.3	10.7	**11.8**	11.9	1.2	1.2
16.9				13.1	10.3	10.7	**11.9**	11.9	1.2	1.2
17		Lougheed Hwy	10.2	13.1	10.3	10.7	**11.9**	**12.0**	1.3	1.3
17.25				13.3	11.2	11.2	**13.0**	**13.3**	1.8	2.1
17.5				13.8	12.1	12.0	**14.4**	**14.6**	2.4	2.6
17.75				14.2	12.8	12.8	**14.5**	**15.3**	1.8	2.5
18			15.23	15.2	14.6	14.6	16.8	17.4	2.2	2.9
18.5				17.1	15.9	16.1	19.7	19.9	3.6	3.8
19				21.4	20.6	20.3	27.0	22.0	6.7	1.7
20				24.8	23.9	23.8	24.8	26.2	1.0	2.4
21				28.3	27.4	27.0	27.9	28.7	1.0	1.7
22				30.0	28.7	28.7	28.6	29.0	-0.1	0.3
23				30.1	29.0	28.9	29.7	30.4	0.8	1.5
24				31.2	30.2	30.3	31.7	33.6	1.4	3.3
25				35.1	34.8	34.1	38.0	37.4	3.9	3.3
26				38.3	37.5	37.0	42.5	46.3	5.5	9.3
27				39.8	38.5	39.0	42.1	39.1	3.1	0.2
28				41.4	41.3	40.4	44.0	41.0	3.6	0.6
29				43.4	42.7	42.6	44.0	43.1	1.4	0.5
30				46.1	45.7	45.1	50.0	44.5	4.9	-0.6
31				48.1	47.3	47.0	50.0	48.0	3.0	1.0
32				50.3	49.3	49.2	49.8	49.8	0.6	0.6
33				52.3	51.3	51.5	55.7	55.0	4.2	3.6
34				54.9	53.9	53.7	55.0	56.0	1.3	2.3
35				57.0	56.2	56.2	57.9	62.3	1.6	6.1
36				58.0	57.3	57.3	60.0	60.0	2.8	2.8

[1]From Northwest Hydraulics (2002)

[a]Based on partial blockage at Kingway Bridge and without raising bed level

Highest of Instantaneous peak +0.3 and Daily Peak +0.6

14.40 Dyke crest

10. DIKE RAISING

10.1. Introduction

Prior to 1989, the Coquitlam River flood control works were comprised of a series of informal dikes constructed to various standards. In 1989, construction started on a formal diking system that extended from the southern end of Kwikwetlem I.R. No.2 to the Lougheed Highway Bridge. The dike construction was completed in 1994. This work was completed under the Canada-British Columbia Fraser River Flood Control Program (FRFCP) 1968 Agreement.

The locations of the FRFCP Dikes are shown on the floodplain maps. FRFCP Dikes exist on the left bank from Lougheed Highway (XS 16) downstream to the southern boundary of I.R. No.2 (XS 6.3). FRFCP Dikes exist on the right bank from Kingsway Avenue (XS 14) downstream to Scott Creek (XS 12), and continuing upstream on the left bank of Scott Creek to (XS 56.0).

The increased flood levels described in Section 8 result in approximately 6 km of the FRFCP Dikes being deficient in elevation. This section describes design work on 1 km of the FRFCP Dikes and provides an extrapolation of costs to the entire 6 km length of FRFCP Dikes. Upgrades to the Colony Farm dikes and other informal dikes are not included in the cost estimates.

The project team completed field survey, base mapping compilation, and design work on the flood mitigation projects in January 2004.

Monitoring of future sediment aggradation is recommended as the dikes may require raising again in the future until an equilibrium condition is reached between the sediment loading and river flushing.

10.2. Left Bank Through Lions Park

Existing Conditions

The existing dike through Lions Park follows the river bank for the north half of the park, then follows a set-back alignment for the southern portion. This set-back alignment takes the form of one of the park elevated pathways and the access road paralleling the existing parking lot. The dike ties in to high ground in the CPR right-of-way at the south end of the park. This original set-back alignment was selected due to concerns that the river bank immediately upstream of the CPR bridge would be susceptible to bank erosion. The existing river bank along the southern half of the park has an informal dike, which includes the Traboulay Poco Trail along its crest. This informal dike is at a higher elevation than the official dike. However, the construction is not to Fraser River Flood Control Standards and there is a low point in the informal dike, where the Traboulay Poco Trail dips under the CPR bridge.

Issues and Constraints

Upgrading the dike system through Lions Park presents many challenges. These challenges are largely the result of the significant increase in dike crest elevation required to meet the latest approved Flood Construction Level (FCL). The dike crest elevation must be increased more than 2.5 m at the southern end of Lions Park, immediately upstream of the CPR bridge. Maintaining the existing Traboulay Poco Trail system along the river bank and through the CPR and Kingsway bridge openings will require modification due to the change in dike crest elevation.

Survey of the CPR railway embankment confirms that the railway grade is below the FCL. Although not specifically included as a design upgrade in this preliminary dike upgrade submission, a continuous tie-in will be required. Conceptually, this may entail constructing a vertical barrier wall parallel to the railway tracks that could be tied to the upstream face of the railway bridge. This is a significant outstanding issue that must be addressed prior to detailed design of the adjacent dike sections.

Recommended Upgrade

We recommend the upgraded dike through Lions Park follow the river bank alignment for its entire length. Upgrading the official dike through the southern half of Lions Park would require raising the existing pathway over 3 m from its existing elevation. This would have a significant negative impact on the aesthetics of the park. Upgrading the dike to follow the river bank allows us to take advantage of the increased elevation of the river bank, and results in less disruption to the park setting. The new dike cross section will include a continuous impervious blanket on the riverside face and a sand and gravel drainage layer as per the drawings 032502-0-104, -109 and -110.

Cost Estimate

The estimated cost for the flood mitigation works through Lions Park is approximately $249,000 excluding engineering, contingencies and taxes. A detailed breakdown of the costs is provided in Appendix B just d/s of CPR bridge

10.3. Left Bank between Kingsway Bridge and McAllister Pedestrian Bridge

≈200m d/s of Kingsway Br

Existing Conditions

The existing dike downstream of the Kingsway bridge is composed of a riprap armoured slope as per the design from 1990. The Traboulay Poco Trail follows the wide dike crest and joins with the pedestrian bridge at McAllister Avenue.

Issues and Constraints

The significant issues considered along this section of dike involve integrating the Traboulay Poco Trail with the upgraded dike crest.

Recommended Upgrade

We recommend extending the riprap armouring to the new Q_{200} elevation, and extending the dike to the new FCL using the same location and configuration of the existing dike. The new dike cross section will include a continuous impervious blanket on the riverside face and a sand and gravel drainage layer as per drawings 032502-0-103, -107 and –108. At the northern end the dike will tie into the Kingsway bridge approach fill.

Cost Estimate

The estimated cost for the flood mitigation works on the left bank between the Kingsway bridge and the McAllister pedestrian bridge is approximately $141,000 excluding engineering, contingencies and taxes. A detailed breakdown of the costs is provided in Appendix B.

10.4. Right Bank between Kingsway Bridge and McAllister Pedestrian Bridge

Existing Conditions

The existing dike system downstream of the Kingsway bridge is composed of a riprap armoured slope as per the design from 1990. In addition, a 100-m long section of Lock Block wall exists along the top of the river bank in this area. The wall, also part of the 1990 design, is located in an area where a garage on private property is located too close to the river to allow for a standard dike cross section.

Issues and Constraints

The existing dike/floodwall must be raised by up to 1.5 m along this section. This results in a longer section of floodwall being required to meet the FCL without disrupting the trail system near the Kingsway bridge. The increased height of the Lock Block wall, up to four blocks high, requires a wider cross section due to the backfill required to stabilize the wall. This wider cross section may result in property impacts along portions of this alignment.

Recommended Upgrade

We recommend extending the riprap armouring to the new Q_{200} elevation, and extending the dike to the new FCL using the same alignment as of the existing dike. The new dike cross section will include a continuous impervious blanket on the riverside face and a sand and gravel drainage layer as per drawings 032502-102, -106, -107, -301, -302 and -303 . In addition, we recommend constructing approximately 150 m of floodwall extending from approximately 40 m upstream of the McAllister pedestrian bridge and tying in to high ground at Kingsway. The floodwall will consist of both a Lock Block wall and a section of cast-in-place concrete wall to tie in to the new dike cross section. The structural design of the floodwall has been completed using the limited geotechnical information available. Further geotechnical investigation is required to confirm our design assumptions prior to construction.

Cost Estimate

The estimated cost for the flood mitigation works on the right bank between the Kingsway bridge and the McAllister pedestrian bridge is approximately $255,000 excluding engineering, contingencies and taxes. A detailed breakdown of the costs is provided in Appendix B.

10.5. Right Bank Downstream of McAllister Pedestrian Bridge

Existing Conditions

The existing dike system downstream of the McAllister pedestrian bridge is a composed standard 3H:1V side slopes as per the design from 1990.

Issues and Constraints

The significant issues considered along this section of dike involve integrating the trail system with the McAllister pedestrian bridge.

Recommended Upgrade

We recommend increasing the elevation of the dike crest by extending the 3H:1V side slopes to meet the new FCL. The new dike cross section will include a continuous impervious blanket on the riverside face and a sand and gravel drainage layer as per drawings 032502-0-102, -105 and -106.

Cost Estimate

The estimated cost for the flood mitigation works on the right bank downstream of the McAllister pedestrian bridge is approximately $172,000 excluding engineering, contingencies and taxes. A detailed breakdown of the costs is provided in Appendix B.

10.6. Cost Summary for Dike Raising

The cost estimate for raising the 1 km of dike included in this project is approximately $1,351,000, as described in the previous sections. A detailed breakdown of the 4 different sections of dike upgrade is provided in Appendix B. Each of these sections has unique characteristics and site specific works are required at each location.

We considered a variety of factors to develop a unit cost for dike upgrades to the formal dikes outside of the 1 km designed as part of this project. This information included the following:

- The difference in elevation between the existing dike crest and the new FCL.
- The existing cross-sections of the dikes included in the 1 km of design for this project.
- The existing cross-sections of the dikes based on design drawings from 1988-1990 for dikes outside of the 1 km included in this project.

Based on a review of these cross-sections and the differences in the existing dike crest and the new FCLs, we consider a unit cost of $1330/m suitable estimate for dike upgrades to the formal dikes on the left bank of the Coquitlam River between XS 8 and XS 13.5.

Significant dike upgrades are required on the left bank from XS 8 to XS 13.5 (2310 m in length) and 140 m of dike on the right bank from the beginning of our design downstream to XS 13.25. Minor dike upgrades are required along the southern boundary of I.R. No.2 (850 m in length) and along the left bank from XS 6.3 to XS 8 (1230 m in length).

Therefore, we applied a unit cost of $1330/m to the 2450 m requiring significant dike upgrades and a unit cost of $345/m for the 2080 m of dike requiring only minor upgrades. This results in a total estimated cost of approximately $3,976,000 for upgrading the formal dikes outside of the 1 km designed as part of this project. Combining this cost with the estimated cost for the 1 km of dike ($1,351,000) gives a total estimated cost of approximately $5,327,000 to upgrade all of the deficient FRFCP Dikes on the Coquitlam River. Scott Creek is not included in this estimate.

Additional areas where dike construction or upgrading would be required include:

- Colony Farm area
- On the west bank between Lougheed Highway and the CP Rail bridge
- Near Hockaday Street

10.7. Cost Summary for Dike Raising with Increased Capacity at the Coquitlam Dam Low-Level Outlet

We completed a cost estimate for dike raising based on reduced FCLs obtained from modeling the design flood with increased low-level outlet capacity at the Coquitlam Dam. The estimated cost for flood mitigation works for the 1 km of dike upgrades included in this project is approximately $302,000. A detailed breakdown of the costs is provided in Appendix B.

We considered the same information as we did for the original 2004 FCLs to develop a unit cost based on the FCLs from the low-level outlet alternative. This unit cost was used for dike upgrades to the formal dikes outside of the 1 km designed as part of this project.

We consider a unit cost of $345/m a suitable estimate for these type of minor dike upgrades to the FRFCP Dikes on the Coquitlam River.

Minor dike upgrades are required on the left bank from XS 9.1 to XS 11.1 (1030 m in length) and 140 m of dike on the right bank from the beginning of our design downstream to XS 13.25.

Therefore, we applied a unit cost of $345/m for the 1170 m of dike requiring these minor upgrades. This results in a total estimated cost of approximately $404,000 for upgrading the formal dikes outside of the 1 km designed as part of this project. Combining this cost with the estimated cost presented earlier, $302,000, gives a total estimated cost of approximately $706,000 to upgrade all of the deficient FRFCP Dikes on the Coquitlam River. Scott Creek is not included in this estimate.

11. EQUIPMENT LANDING AREA FOR DEBRIS CONTROL

Significant floating debris accumulation at the CPR bridge is considered likely during a Q_{200} flood event. The CPR bridge, and the Kingsway bridge immediately downstream, are located on a channel bend and constrict the Coquitlam River at this location. In addition to the narrowed channel width, the bridge soffit elevations are both lower than the Q_{200} peak water surface elevation. The constricted channel opening, along with the channel bend will likely cause debris to accumulate upstream of the CPR bridge and be directed toward the left river bank.

Issues and Constraints

The most feasible location from which to remove floating debris from the upstream side of the CPR bridge is the left bank, which is located in Lions Park. The right bank at this location is extremely flat and will not permit equipment to operate close enough to the river during a Q_{200} flood event. The left bank of the Coquitlam River at Lions Park would be upgraded as part of the flood mitigation works described earlier. Therefore, the dike upgrades through Lions Park will be integrated into the layout of the Equipment Landing Area. Other significant design considerations involve maintaining the integrity of the Traboulay Poco Trail through Lions Park and limiting the overall footprint of the Equipment Landing Area.

We recently became aware of the Port Coquitlam Parks Department intention to develop a Youth Park within Lions Park. Our understanding is that this Youth Park will encompass the entire existing parking lot, and possibly extend a small distance beyond the parking lot towards the Coquitlam River. The Equipment Landing Area and the equipment access required, must be able to co-exist with the planned Youth Park. Resolving this potential conflict will require coordination with the planning of both projects.

Recommended Upgrade

In December 2003, we investigated 7 different debris removal alternatives. These alternatives were as follows:

- A cable system with a grapple or clamshell.
- A crane on high rails working from the CPR bridge.

- A long-reach excavator with a hydraulic thumb working from a structural platform.
- A series of piles driven into the riverbed upstream of the CPR bridge.
- A system involving a cable and baffles across the river.
- A long-reach excavator with a hydraulic thumb working from the top of bank.
- A mobile crane with grapple working from the top of bank.

Use of the CP Rail bridge as the major component of the floating debris removal system has the following concerns:

- The estimated Q200 water surface elevation is approximately 600 mm above the existing railway grade (the CPR bridge will be submerged during a Q200 flood event).

- Operator safety would be a concern when working from the bridge during a significant flood event.

- Forcing jammed floating debris under the bridge structure would likely only be possible during the smaller flood events.

- There would still be a major impact on Lions Park without the elevated Equipment Landing Area, as the dike would still be required at the same elevation (FCL = 12.6 m). However, with due consideration for other infrastructure and land use activities, a set-back alignment is possible.

- A staging area (on land) would still be required, albeit at a possibly reduced elevation from the FCL.

- Site access would still be required for debris handling and removal.

- The rate of material removal will be much slower than that achieved by a land-based crane. Each piece of debris, regardless of shape, will need to be placed on a flatbed railcar. Conceptually, an excavator equipped with a hydraulic thumb would be positioned between two flatbed railcars. The debris could be placed on the two adjacent railcars until capacity is reached. Capacity is expected to be reached quickly due to the awkward shape of the unprocessed debris. Operators would then trim overhanging branches. The railcars would be powered to the adjacent land-based staging area for further processing and disposal. This process is expected to be slow and would not be efficient at removing the large volumes of debris expected during the design flood event.

- An agreement with CPR is required. As a minimum, the agreement must include access to the tracks, operator certification, access to equipment (flatbed railcars, excavator, and engine), and indemnity issues.

The mobile crane with grapple alternative located on a landing area on the east bank was selected at the December 17, 2003 Advisory Committee Meeting. This alternative was selected mainly because it minimizes long-term maintenance, does not further constrict the bridge opening, and provides a long reach across the river.

We designed the Equipment Landing Area to have an elevation matching the new FCL and to have a working area large enough to accommodate the required equipment. A Lock Block retaining wall will serve both as a floodwall and support for the Equipment Landing Area. The wall follows a staggered alignment, as shown on the drawings, to allow for integration with the existing Traboulay Poco Trail that passes under the CPR and Kingsway bridges. The re-aligned Traboulay Poco Trail will follow the base of wall as it transitions from the low elevation under the bridges until it reaches the dike crest. The structural design of the wall has been completed using the limited geotechnical information available. Further geotechnical investigation is required to confirm our design assumptions prior to construction.

The proposed Equipment Landing Area is split into 2 levels, and covers an area approximately 40 m x 40 m. The top level includes the crane platform and integration with the dike crest. The lower level, 1 m below crane platform, sits behind the dike crest and includes a working area with allowances for truck turnarounds. We designed the working area to be lower than the crane platform and dike crest to decrease the overall footprint of the Equipment Landing Area in Lions Park. Access to the working area and crane platform will need to originate from Lions Way and follow an access road through the park. Drawing 032502-0-112 shows a detailed layout of the Equipment Landing Area.

We recommend using a conventional truck crane with a minimum capacity of 60 tons and a minimum boom length of 150 ft. Accordingly, we designed the crane platform with an area of 10 m wide by 20 m long to provide sufficient area for the crane to be stabilized in position with the outriggers extended and set. The size of the crane platform will also accommodate larger cranes or other equipment if desired.

Cost Estimate

The estimated cost for the Equipment Landing Area is approximately $421,000. A detailed breakdown of the costs is provided in Appendix B.

12. EROSION PROTECTION WORKS

Existing Conditions

An erosion area had been identified on the right bank of the Coquitlam River approximately 500 m upstream of Lougheed Highway. This erosion area currently extends for a length of 20 m, with a maximum height of 2.5 m. Rock riprap slope armouring exists along the river bank immediately downstream of the erosion area. However, the existing riprap slope is over-steepened to approximately 1.2H:1V with loose rock in places.

Issues and Constraints

The most direct access to the site is to follow the existing dike crest from Lougheed Highway. There is an existing access ramp down the river bank immediately upstream of the erosion area. This access ramp will allow construction equipment to access the toe of the erosion area without significant disruption to the river bank and existing vegetation.

Recommended Upgrade

We recommend placing rock riprap slope protection through the erosion area as shown on drawing 032502-0-115. Our design indicates that the new riprap must be keyed in to the native ground and form a smooth transition with the existing riprap downstream of the erosion area. The new riprap will be placed up to the elevation of the existing downstream riprap, with vegetative planting undertaken from the top of the riprap to the new Q_{200} elevation. The new riprap slope protection must transition between a bank slope of 1.2H:1V at the downstream side to 2.5H:1V at the upstream side.

Cost Estimate

The estimated cost for the Erosion Protection Works is approximately $53,000. A detailed breakdown of the costs is provided in Appendix B.

13. CONCLUSIONS, OUTSTANDING ISSUES AND RECOMMENDATIONS

13.1. Conclusions

Flood flows in the Coquitlam River are sensitive to the assumptions made regarding the volume distribution of the reservoir flood inflow hydrograph and the timing of downstream tributary flows. Coquitlam River flood flows are not sensitive to initial reservoir conditions when multi-day inflow design hydrographs are used for periods of 110 days.

The selected design flood in the Coquitlam River has an instantaneous peak flow of 632 m^3/s at the CP Rail bridge and a mean daily flow of 448 m^3/s. Previous studies had used a higher peak daily flow of 573 m^3/s but had not considered peak instantaneous flow. When freeboard is considered for defining the Flood Construction Level (FCL), the FCL using the peak instantaneous flow was found to be higher than the FCL using the peak daily flow at most river sections. The FCL values defined in this report were the higher of the two sets of values.

A partial debris jam for 1 m below the soffit of the CP Rail bridge was included in the modelling because of the historical evidence of floating debris in the river during flood events and the fact that the water surface profile is higher than the bridge soffit. A partial debris jam was also included at the Kingsway Bridge, which blocked the right bank opening between the bank and the first bridge pier.

The existing dikes were found to be generally deficient in crest elevation by up to 1.61 m. Immediately upstream of the CP Rail bridge, the dikes are up to 2.52 m too low as a result of the constriction of the bridge and the debris jams.

Alternative flood mitigation alternatives were considered including allocating additional flood storage in the Coquitlam Reservoir, increasing the low level outlet capacity at Coquitlam Dam, channel excavation, modifying bridges and raising existing dikes. It was concluded that two options would provide cost-effective feasible alternatives with the least environmental impact:

1. Dike raising in combination with increasing the capacity of the Coquitlam Dam low level outlet would cost about $4 to 6 million. There would be no requirement for additional diking except for upgrading the Colony Farm dikes; however, it is noted that the dikes for Colony Farm were designed and built with a lower left bank elevation to reduce flood levels on the right bank by allowing high flood flows behind

the left dike. Flood Construction Levels in floodplain areas would be reduced with this option, which would reduce the costs of development in the floodplain.

2. Raising the FRFCP Dikes without increasing the low level outlet capacity at Coquitlam Dam would cost a total of about $5.3 million. In addition there would be new areas requiring diking, for example on the right bank of the Coquitlam River between the Lougheed Highway Bridge and the CP Rail Bridge, and near Hockaday Street.

13.2. Outstanding Issues

As a result of the rapid survey and design schedule, several outstanding issues remain to be resolved as listed below:

- Further consideration of the alternative to increase the low-level outlet capacity at Coquitlam Dam will require an engineering preliminary design study and a process to address several operational, social and economic key issues to ensure that 200 m^3/s can be passed safely downstream.

- The increased flood profiles based on the dike raising option result in significant dike raising. As well, high retaining walls are required adjacent to the debris landing area in Lions Park and on the right bank downstream of the Kingsway Bridge. The project team has designed these structures based on limited geotechnical information and conservative soils assumptions. We recommend that specialist geotechnical reviews and/or investigations be completed prior to detailed design.

- Survey of the CPR railway embankment confirms that the railway grade is below the FCL. Although not specifically included as a design upgrade in this preliminary dike upgrade submission, a continuous tie-in will be required if the dike raising option is selected. Conceptually, this may entail constructing a vertical barrier wall parallel to the railway tracks that could be tied to the upstream face of the railway bridge. This is a significant outstanding issue that must be addressed prior to detailed design of the adjacent dike sections.

- The debris landing area and dike upgrades will have a significant impact on Lions Park. It is noted that the dike upgrades for the dike raising option would be more significant than for the low level outlet option. Other planning initiatives, including a planned Youth Park at the southern end of the park must be coordinated with the proposed flood control works.

- Private infrastructure is impacted, particularly on the right bank downstream of the Kingsway Bridge. A garage must be removed based on the current design. As well, dike fills extend into a private housing complex. Additional property impacts should be assessed at the detailed design stage.

- During the course of the study, another erosion area was identified on the left bank, upstream of the right bank erosion area. No action has been taken under this assignment to address the newly identified erosion area.

- The revised flood profiles confirm that approximately 6 km of FRFCP Dikes are deficient in elevation. As well, there are areas where no FRFCP Dikes exist that are deficient including:

    - Colony Farm area
    - On the west bank between Lougheed Highway and the CP Rail bridge
    - Near Hockaday Street

13.3. Recommendations

1. That further consideration be given to design investigations to increase the capacity of the low level outlet at the Coquitlam Dam. This would reduce the 200-year design flow, significantly reduce dike raising costs and impacts, and result in lower Flood Construction Levels throughout the floodplain area. It would also increase the safety of operating the reservoir above the flood buffer in the spring/summer. To ensure that 200 m^3/s can be safely passed downstream, operational and downstream issues must be addressed.
2. That raising the FRFCP Dikes be considered as a fallback flood mitigation alternative if the low level outlet option is not achievable.
3. That the floodplain maps provided in this report be used for the designation of Flood Construction Levels in the Coquitlam River floodplain until the final design flood flow is selected.
4. That the final design of the dikes include geotechnical investigations.
5. That an equipment landing area be developed on the east bank of the Coquitlam River at Lions Park so that a mobile crane with grapple can be used for debris removal during flood events.
6. That erosion protection works be constructed on the right bank of the Coquitlam River about 500 m upstream of the Lougheed Highway based on the design presented in this report.
7. That sediment monitoring be continued by periodic cross section surveys every five years and the design capacity of the channel verified.

REFERENCES

Associated Engineering Services Ltd. 1988 Coquitlam River Flood Control Works. First Stage Final Design Report.

BC Hydro, 2004 Coquitlam-Buntzen Project Water Use Plan, 19 May 2004.

Bland Engineering Ltd. 2001a Coquitlam River Cross Sections. Report to BC Hydro

Bland Engineering Ltd. 200 b Coquitlam River Downstream of Pitt River Road: Report on Navigation and Flood Protection issues relating to First Nation Lands. Report to BC Hydro

Greater Vancouver Regional District, 1998 Or Creek Watershed Restoration Program Level 1 Assessment. GVRD Watershed Management.

Northwest Hydraulic Consultants, 2002 Coquitlam River Dike Study: Hockaday Street to Fraser River. Report to the Cities of Coquitlam and Port Coquitlam

U.S. Army Corps of Engineers, 1994, Engineering and Design – Hydraulic Design of Flood Control Channels. Publication Number EM 1110-2-1601

US Army Corps of Engineers, 1997 Engineering Manual EM 110-2-1420- Hydrologic Engineering Design Requirements for Reservoirs Chapter 10, Flood Control Storage

Water Investigations Branch 1976, Coquitlam River Hydrology: High Flows. British Columbia Ministry of Environment. File: 0256957

Water Investigations Branch 1978, Coquitlam River Water Management Study. British Columbia Ministry of Environment.

Water Rights Branch 1961, Preliminary Report on Coquitlam River Flooding, Volumes 1 and 2. R. A. Pollard.

Water Management Consultants March 2003, Coquitlam River Hydrology Review

APPENDIX A
OCTOBER 16, 2003
WATER SURFACE PROFILE

Table 1

City of Port Coquitlam
Flood Hazard Mitigation Options - Coquitlam River
Flood Profile Survey - Documentation of Water Levels from October 16-17, 2003 Storm Event

Point ID	Dwg	Elev (m)	Location	Date	Time	Photo #	Description
PWL-1	2N	3.20	Red Bridge	10/17/03	2:00 PM	101	Hub on east bank, approximately 1 m downslope from MH, 10 m downstream of bridge.
HWM-1	2N	4.14	Red Bridge	10/22/03	4:30 PM	102-103	Hub on east bank at debris line upslope from MH, 10 m downstream of bridge.
PWL-2	2N	3.27	Red Bridge	10/17/03	2:00 PM	104	Hub on east bank, approximately 3 m downslope from pier at upstream side of bridge.
HWM-2	2N	4.19	Red Bridge	10/17/03	2:00 PM	105	Painted line on base of bridge abutment at upstream side of bridge (east bank).
PWL-3	2N	3.25	Red Bridge	10/17/03	2:10 PM	106-107	Flagging on vegetation/painted line on ground 20 m upstream of bridge (east bank). No HWM at this location.
PWL-4	3S	4.38	Reeve Park	10/17/03	2:25 PM	108-111	Hub on river bank adjacent to parking lot at Reeve Park (West of Wilson Street cul-de-sac)
HWM-4	3S	5.64	Reeve Park	10/22/03	3:30 PM	108-111	Hub on river bank adjacent to parking lot at Reeve Park (West of Wilson Street cul-de-sac)
PWL-5	3S	5.65	McAllister Ped. Brg. - East End	10/17/03	2:50 PM	113-117	Hub on east bank at upstream side of McAllister pedestrian bridge. Hub disturbed prior to survey.
HWM-5	3S	6.79	McAllister Ped. Brg. - East End	10/22/03	2:30 PM	113-117	Paint on rocks on east bank at upstream side of McAllister pedestrian bridge. Rough estimate of HWM only.
PWL-6	3S	5.50	McAllister Ped. Brg. - West End	10/17/03	3:00 PM	113-117	Hub on west bank at upstream side of McAllister pedestrian bridge.
HWM-6	3S	6.47	McAllister Ped. Brg. - West End	10/22/03	2:45 PM	113-117	Paint on rocks on west bank at upstream side of McAllister pedestrian bridge. Better estimate than east bank.
PWL-7	3S	6.16	D/S of Kingsway Bridge	10/17/03	3:20 PM	118-120	Hub on east bank, 30 m downstream of blue gate on downstream side of Kingsway bridge.
HWM-7	3S	7.10	D/S of Kingsway Bridge	10/17/03	3:20 PM	118-120	Paint on rocks on east bank, 30 m downstream of blue gate on downstream side of Kingsway bridge.
PWL-8	3S	6.34	between Kingsway & CPR Brgs.	10/17/03	3:25 PM	121-124	Hub on east bank between Kingsway and CPR bridges.
HWM-8	3S	7.29	Kingsway Bridge	10/22/03	1:30 PM	121-124	Hub on east bank, 3 m downstream of Kingsway bridge.
PWL-9	3S	6.41	U/S of CPR Bridge	10/17/03	4:00 PM	125-126	Hub on east bank, 35 m upstream of CPR bridge.
HWM-9	3S	7.37	U/S of CPR Bridge	10/17/03	4:00 PM	125-126	Hub on east bank, 35 m upstream of CPR bridge.
PWL-10	3N	6.49	middle of Lions Park	10/17/03	4:05 PM	n/a	Hub on east bank near the middle of Lions Park (historic plaques), near intersection of paved paths.
HWM-10	3N	7.55	middle of Lions Park	10/22/03	1:00 PM	n/a	Hub on east bank near the middle of Lions Park (historic plaques), near intersection of paved paths.
PWL-11	3N	7.41	D/S of Lougheed Bridge	10/17/03	4:15 PM	n/a	Hub on east bank, 25 m downstream of Lougheed bridge.
HWM-11	3N	8.51	D/S of Lougheed Bridge	10/17/03	4:15 PM	n/a	Paint on rocks on east bank, 25 m downstream of Lougheed bridge. HWM difficult to estimate.
PWL-12	3N	7.73	U/S of Lougheed Bridge	10/17/03	4:25 PM	n/a	Hub on east bank, 25 m upstream of Lougheed bridge.
HWM-12	3N	8.59	U/S of Lougheed Bridge	10/17/03	4:25 PM	n/a	Paint on rocks on east bank, 25 m upstream of Lougheed bridge.
PWL-13	4S	10.25	300 m D/S of Patricia Brg.	10/22/03	11:45 AM	127	Hub on east bank, approx. 300 m downstream of Patricia ped. brg. Significant bank erosion immediately D/S.
HWM-13	4S	11.83	300 m D/S of Patricia Brg.	10/17/03	5:15 PM	n/a	Hub on east side of walking path (path was submerged), approx. 300 m downstream of Patricia ped. brg.
PWL-14	4S	13.04	D/S of Patricia Ped. Bridge	10/17/03	5:00 PM	n/a	Hub on east bank on downstream side of Patricia pedestrian bridge.
HWM-14	4S	13.72	D/S of Patricia Ped. Bridge	10/17/03	5:00 PM	n/a	Painted line on east bank on downstream side of Patricia pedestrian bridge.
PWL-15	5N	38.47	Hockaday Street	10/22/03	10:00 AM	201-203	Hub on north bank, south of cul-de-sac at end of Hockaday Street. Level gauge here read approx. 38.4 m.
HWM-15	5N	39.27	Hockaday Street	10/22/03	10:00 AM	201-203	Hub at debris line on north bank, south of cul-de-sac at end of Hockaday Street.
PWL-16	5N	51.83	Gallette Avenue	10/17/03	5:40 PM	n/a	0.5 m below top of headwall at storm outfall on west bank.
HWM-16	5N	52.83	Gallette Avenue	10/17/03	5:40 PM	n/a	Paint on rocks on west bank, immediately upstream of storm outfall.

Notes:
1. Indicates data that may be unreliable
2. Photos #101-127 were taken on October 17, 2003, photos #201-203 were taken on October 22, 2003.
3. Point ID refers to either PWL = Present Water Level, or HWM = High Water Mark.
4. Drawing numbers are referenced to the set of 5 drawings, "Coquitlam River Floodplain Maps: Fraser River to Hockaday Street", provided my Water Management Consultants Inc. (Example: Dwg 3S refers to the south half of drawing #3, and Dwg 3N refers to the north half of drawing #3)

APPENDIX B
COST ESTIMATES

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE

	Unit	Unit Price	Quantity	Total Cost
FLOOD MITIGATION ALTERNATIVES				
General				
Mob/Demob	LS			$10,000
Fences/Gates	LS			$11,250
Utilities	LS			$15,000
Asphalt Removal	m^2	$4.00	2800	$11,200
Stripping of Topsoil	m^3	$8.50	1532	$13,018
Seeding	m^2	$1.00	12000	$12,000
Miscellaneous	LS			$18,750
Subtotal				$91,218
Left Bank Through Lions Park				
Impervious Fill Material	m^3	$20.00	2267	$45,344
Imported Bulk Dyke Fill	m^3	$20.00	7149	$142,977
Sand and Gravel Drainage Material	m^3	$30.00	1200	$36,012
Topsoil	m^3	$0.80	767	$614
Surface Gravel	m^3	$40.00	207	$8,295
Excavation	m^3	$10.00	1534	$15,340
Subtotal				$248,581
Left Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$20.00	803	$16,066
Imported Bulk Dyke Fill	m^3	$20.00	2414	$48,273
Sand and Gravel Drainage Material	m^3	$30.00	638	$19,131
Topsoil	m^3	$0.80	192	$153
Surface Gravel	m^3	$40.00	112	$4,465
Rock Riprap	m^3	$50.00	740	$36,989
Rock Filter	m^3	$50.00	247	$12,355
Excavation	m^3	$10.00	383	$3,831
Subtotal				$141,263
Right Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$20.00	188	$3,756
Imported Bulk Dyke Fill	m^3	$20.00	538	$10,756
Sand and Gravel Drainage Material	m^3	$30.00	125	$3,750
Topsoil	m^3	$0.80	38	$30
Surface Gravel	m^3	$40.00	24	$952
Rock Riprap	m^3	$50.00	180	$8,990
Rock Filter	m^3	$50.00	57	$2,837
Excavation	m^3	$10.00	75	$753
Lock Block Wall	LS			$43,000
Cast-in-Place Concrete Wall	LS			$180,000
Subtotal				$254,823

Right Bank - Downstream of McAllister				
Impervious Fill Material	m^3	$20.00	1422	$28,435
Imported Bulk Dyke Fill	m^3	$20.00	4979	$99,570
Sand and Gravel Drainage Material	m^3	$30.00	919	$27,584
Topsoil	m^3	$0.80	535	$428
Surface Gravel	m^3	$40.00	133	$5,307
Excavation	m^3	$10.00	1071	$10,707
Subtotal				$172,032
Subtotal Flood Mitigation Works				**$907,918**
Engineering (15%)				$136,188
Subtotal Construction of Flood Mitigation Works				**$1,044,105**
Contingency (25%)				$226,979
GST (7%)				$79,443
Total Cost Estimate for Flood Mitigation Works				**$1,350,527**

Note: The Equipment Landing Area includes costs for Flood Mitigation Works (Left Bank Through Lions Park) for Cross Sections 7+479L to 7+503L.

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE

	Unit	Unit Price	Quantity	Total Cost
EQUIPMENT LANDING AREA				
Impervious Fill Material	m^3	$20.00	280	$5,602
Imported Bulk Dyke Fill	m^3	$20.00	7148	$142,964
Sand and Gravel Drainage Material	m^3	$30.00	478	$14,329
Topsoil	m^3	$0.80	45	$36
Surface Gravel	m^3	$40.00	330	$13,196
Excavation	m^3	$10.00	91	$907
Lock Block Wall	LS			$75,650
Mob/Demob	LS			$10,000
Fences/Gates	LS			$3,750
Utilities	LS			$4,600
Asphalt Removal	m^2	$4.00	400	$1,600
Stripping of Topsoil	m^3	$8.50	45	$386
Seeding	m^2	$1.00	4000	$4,000
Miscellaneous	LS			$5,750
Subtotal				$282,770
Subtotal Equipment Landing Area				**$282,770**
Engineering (15%)				$42,416
Subtotal Construction of Equipment Landing Area				**$325,186**
Contingency (25%)				$70,693
GST (7%)				$24,742
Total Cost Estimate for Equipment Landing Area				**$420,621**

Note: The Equipment Landing Area includes costs for Flood Mitigation Works (Left Bank Through Lions Park) for Cross Sections 7+479L to 7+503L.

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE

	Unit	Unit Price	Quantity	Total Cost
EROSION PROTECTION WORKS				
Sandy Organic Fill Material	m^3	$30.00	31	$923
Vegetative Planting	m^2	$20.00	125	$2,500
Rock Riprap	m^3	$50.00	335	$16,750
Rock Filter	m^3	$50.00	100	$5,013
Mob/Demob	LS			$10,000
Miscellaneous	LS			$500
Subtotal				$35,685
Subtotal Erosion Protection Works				**$35,685**
Engineering (15%)				$5,353
Subtotal Construction of Erosion Protection Works				**$41,038**
Contingency (25%)				$8,921
GST (7%)				$3,122
Total Cost Estimate for Erosion Protection Works				**$53,081**

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE - CHANNEL EXCAVATION OPTION

	Unit	Unit Price	Quantity	Total Cost
FLOOD MITIGATION ALTERNATIVES				
Left Bank Through Lions Park				
Impervious Fill Material	m^3	$20.00	1859	$37,182
Imported Bulk Dyke Fill	m^3	$20.00	4933	$98,654
Sand and Gravel Drainage Material	m^3	$30.00	1044	$31,330
Topsoil	m^3	$0.80	660	$528
Surface Gravel	m^3	$40.00	207	$8,295
Excavation	m^3	$10.00	1319	$13,193
Subtotal				$189,181
Left Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$20.00	659	$13,174
Imported Bulk Dyke Fill	m^3	$20.00	1665	$33,308
Sand and Gravel Drainage Material	m^3	$30.00	555	$16,644
Topsoil	m^3	$0.80	165	$132
Surface Gravel	m^3	$40.00	112	$4,465
Rock Riprap	m^3	$50.00	614	$30,701
Rock Filter	m^3	$50.00	203	$10,131
Excavation	m^3	$10.00	329	$3,295
Subtotal				$111,850
Right Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$20.00	154	$3,080
Imported Bulk Dyke Fill	m^3	$20.00	371	$7,422
Sand and Gravel Drainage Material	m^3	$30.00	109	$3,262
Topsoil	m^3	$0.80	32	$26
Surface Gravel	m^3	$40.00	24	$952
Rock Riprap	m^3	$50.00	149	$7,461
Rock Filter	m^3	$50.00	47	$2,327
Excavation	m^3	$10.00	65	$647
Lock Block Wall	LS			$43,000
Cast-in-Place Concrete Wall	LS			$180,000
Subtotal				$248,176
Right Bank - Downstream of McAllister				
Impervious Fill Material	m^3	$20.00	1166	$23,317
Imported Bulk Dyke Fill	m^3	$20.00	3435	$68,704
Sand and Gravel Drainage Material	m^3	$30.00	800	$23,998
Topsoil	m^3	$0.80	460	$368
Surface Gravel	m^3	$40.00	133	$5,307
Excavation	m^3	$10.00	921	$9,208
Subtotal				$130,902

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE - CHANNEL EXCAVATION OPTION

	Unit	Unit Price	Quantity	Total Cost
EQUIPMENT LANDING AREA				
Impervious Fill Material	m^3	$20.00	230	$4,594
Imported Bulk Dyke Fill	m^3	$20.00	4932	$98,645
Sand and Gravel Drainage Material	m^3	$30.00	416	$12,466
Topsoil	m^3	$0.80	39	$31
Surface Gravel	m^3	$40.00	330	$13,196
Excavation	m^3	$10.00	78	$780
Lock Block Wall	LS			$75,650
Subtotal				$205,363
EROSION PROTECTION WORKS				
Sandy Organic Fill Material	m^3	$30.00	31	$923
Vegetative Planting	m^2	$20.00	125	$2,500
Rock Riprap	m^3	$50.00	335	$16,750
Rock Filter	m^3	$50.00	100	$5,013
Subtotal				$25,185
GENERAL				
Mob/Demob	LS			$10,000
Fences/Gates	LS			$15,000
Utilities	LS			$20,000
Asphalt Removal	m^2	$4.00	3200	$12,800
Stripping of Topsoil	m^3	$8.50	1577	$13,404
Seeding	m^2	$1.00	15000	$15,000
Miscellaneous	LS			$25,000
Subtotal				$111,204
CHANNEL EXCAVATION				
Channel Excavation	m^3	$15.00	175000	$2,625,000
Subtotal Coquitlam River Works				**$3,646,861**
Engineering (15%)				$547,029
Subtotal Construction				**$4,193,890**
Contingency (25%)				$911,715
GST (7%)				$319,100
Total Cost Estimate				**$5,424,706**

Note: The Equipment Landing Area includes costs for Flood Mitigation Works (Left Bank Through Lions Park) for Cross Sections 7+479L to 7+503L.

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE - 200 m^3/s LOW-LEVEL OUTLET IN DAM ALTERNATIVE

	Unit	Unit Price	Quantity	Total Cost
FLOOD MITIGATION ALTERNATIVES				
General				
Mob/Demob	LS			$10,000
Fences/Gates	LS			$11,250
Utilities	LS			$15,000
Asphalt Removal	m^2	$4.60	2800	$12,880
Stripping of Topsoil	m^3	$9.78	368	$3,593
Seeding	m^2	$1.15	4000	$4,600
Miscellaneous	LS			$18,750
Subtotal				$76,073
Left Bank Through Lions Park				
Impervious Fill Material	m^3	$23.00		$0
Imported Bulk Dyke Fill	m^3	$23.00	848	$19,494
Sand and Gravel Drainage Material	m^3	$34.50		$0
Topsoil	m^3	$0.92	114	$105
Surface Gravel	m^3	$46.00	207	$9,539
Excavation	m^3	$11.50	496	$5,702
Subtotal				$34,839
Left Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$23.00		$0
Imported Bulk Dyke Fill	m^3	$23.00	430	$9,891
Sand and Gravel Drainage Material	m^3	$34.50		$0
Topsoil	m^3	$0.92	81	$74
Surface Gravel	m^3	$46.00	112	$5,135
Rock Riprap	m^3	$57.50	196	$11,259
Rock Filter	m^3	$57.50	51	$2,946
Excavation	m^3	$11.50	393	$4,525
Subtotal				$33,830
Right Bank - Kingsway to McAllister				
Impervious Fill Material	m^3	$23.00		$0
Imported Bulk Dyke Fill	m^3	$23.00	273	$6,279
Sand and Gravel Drainage Material	m^3	$34.50		$0
Topsoil	m^3	$0.92	23	$22
Surface Gravel	m^3	$46.00	24	$1,094
Rock Riprap	m^3	$57.50	49	$2,803
Rock Filter	m^3	$57.50	17	$987
Excavation	m^3	$11.50	97	$1,112
Lock Block Wall	LS			$15,000
Cast-in-Place Concrete Wall	LS			
Subtotal				$27,297

Right Bank - Downstream of McAllister				
Impervious Fill Material	m^3	$23.00		$0
Imported Bulk Dyke Fill	m^3	$23.00	918	$21,115
Sand and Gravel Drainage Material	m^3	$34.50		$0
Topsoil	m^3	$0.92	150	$138
Surface Gravel	m^3	$46.00	133	$6,103
Excavation	m^3	$11.50	302	$3,470
Subtotal				$30,826
Subtotal Flood Mitigation Works				**$202,866**
Engineering (15%)				$30,430
Subtotal Construction of Flood Mitigation Works				**$233,296**
Contingency (25%)				$50,717
GST (7%)				$17,751
Total Cost Estimate for Flood Mitigation Works				**$301,763**

Note: The Equipment Landing Area includes costs for Flood Mitigation Works (Left Bank Through Lions Park) for Cross Sections 7+479L to 7+503L.

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

Note: The Unit Prices for dike works in this Cost Estimate have been increased by 15% over the original Cost Estimate to account for small quantities.

CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE - 200 m^3/s LOW-LEVEL OUTLET IN DAM ALTERNATIVE

	Unit	Unit Price	Quantity	Total Cost
EQUIPMENT LANDING AREA				
Impervious Fill Material	m^3	$23.00		$0
Imported Bulk Dyke Fill	m^3	$23.00	1538	$35,363
Sand and Gravel Drainage Material	m^3	$34.50		$0
Topsoil	m^3	$0.92	22	$20
Surface Gravel	m^3	$46.00	330	$15,176
Excavation	m^3	$11.50	302	$3,473
Lock Block Wall	LS			$50,000
Mob/Demob	LS			$10,000
Fences/Gates	LS			$3,750
Utilities	LS			$4,600
Asphalt Removal	m^2	$4.60	400	$1,840
Stripping of Topsoil	m^3	$9.78	22	$211
Seeding	m^2	$1.15	1000	$1,150
Miscellaneous	LS			$5,750
Subtotal				$131,332
Subtotal Equipment Landing Area				**$131,332**
Engineering (15%)				$19,700
Subtotal Construction of Equipment Landing Area				**$151,032**
Contingency (25%)				$32,833
GST (7%)				$11,492
Total Cost Estimate for Equipment Landing Area				**$195,357**

Note: The Equipment Landing Area includes costs for Flood Mitigation Works (Left Bank Through Lions Park) for Cross Sections 7+479L to 7+503L.

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

Note: The Unit Prices for dike works in this Cost Estimate have been increased by 15% over the original Cost Estimate to account for small quantities.

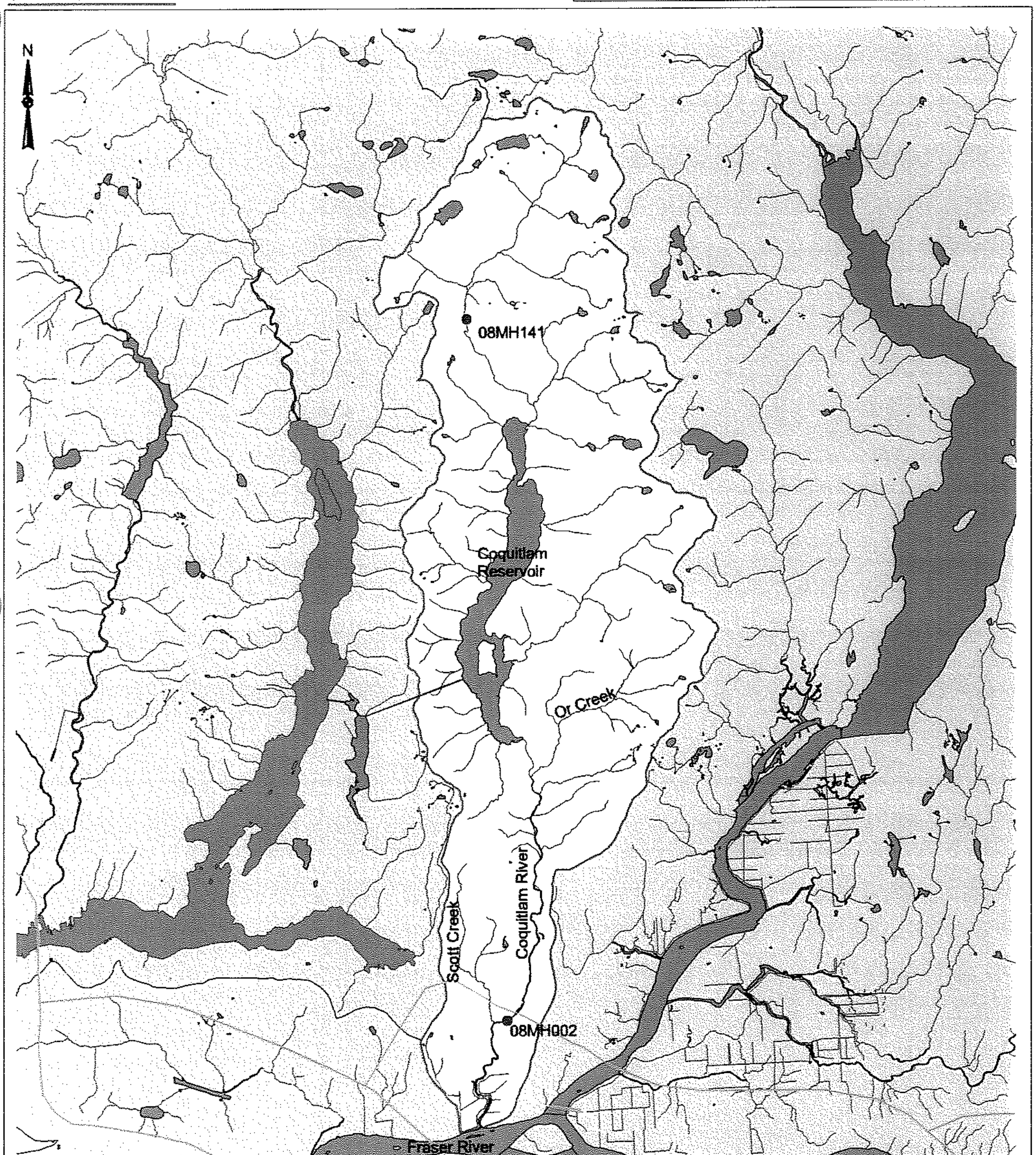
CITY OF PORT COQUITLAM
FLOOD HAZARD MITIGATION OPTIONS - COQUITLAM RIVER
COST ESTIMATE - 200 m^3/s LOW-LEVEL OUTLET IN DAM ALTERNATIVE

	Unit	Unit Price	Quantity	Total Cost
EROSION PROTECTION WORKS				
Sandy Organic Fill Material	m^3	$30.00	31	$923
Vegetative Planting	m^2	$20.00	125	$2,500
Rock Riprap	m^3	$50.00	335	$16,750
Rock Filter	m^3	$50.00	100	$5,013
Mob/Demob	LS			$10,000
Miscellaneous	LS			$500
Subtotal				$35,685
Subtotal Erosion Protection Works				**$35,685**
Engineering (15%)				$5,353
Subtotal Construction of Erosion Protection Works				**$41,038**
Contingency (25%)				$8,921
GST (7%)				$3,122
Total Cost Estimate for Erosion Protection Works				**$53,081**

Note: The Mob/Demob cost may be reduced if all 3 projects components are completed concurrently.

FIGURES

Figure 1.1 Coquitlam River Watershed



WATER MANAGEMENT CONSULTANTS

Figure 3.1 - 200 Yr Regional Flood Frequency Discharge vs. Drainage Area
Pearson Type III

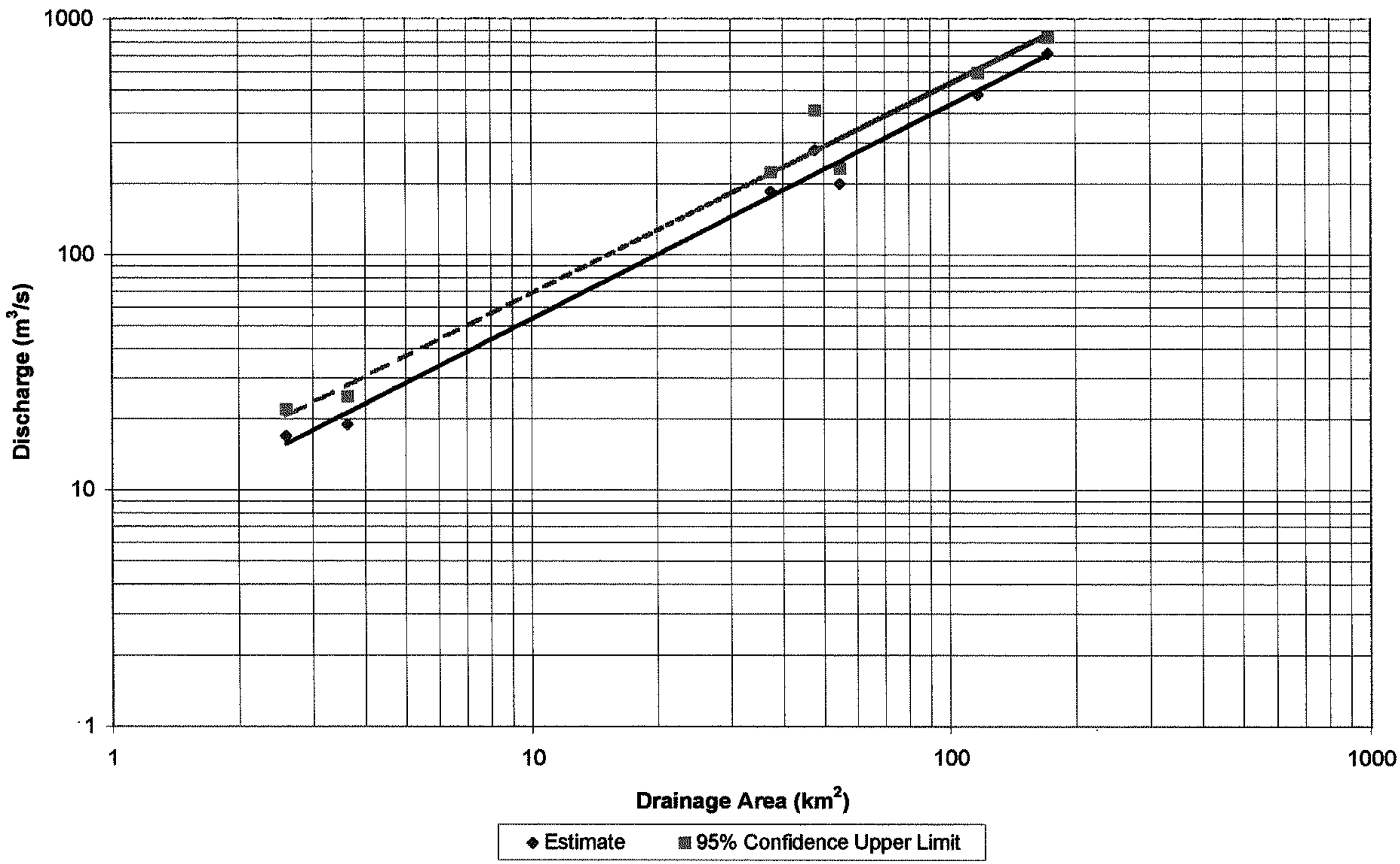


Figure 3.2 200-year Inflow Volumes

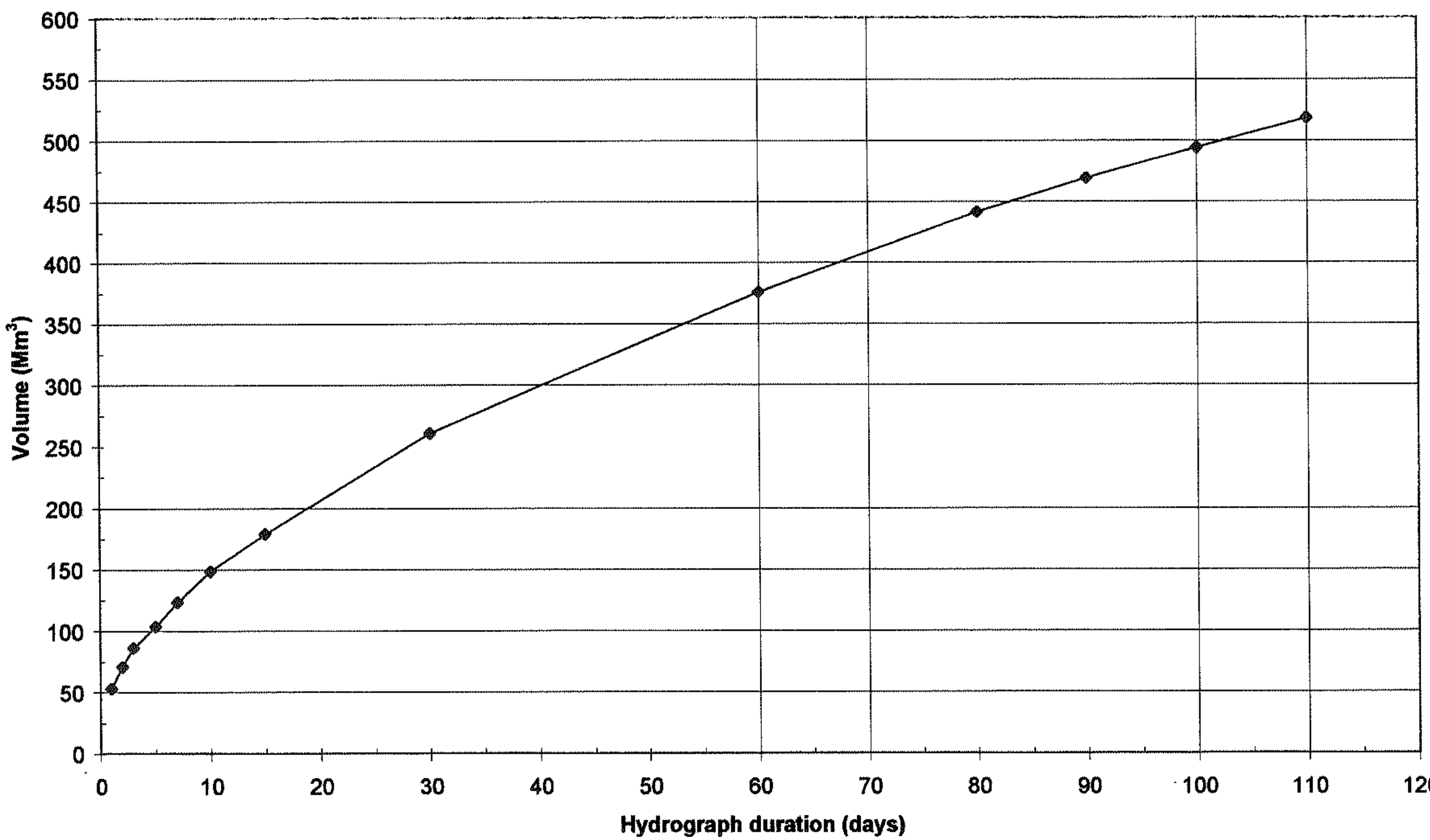
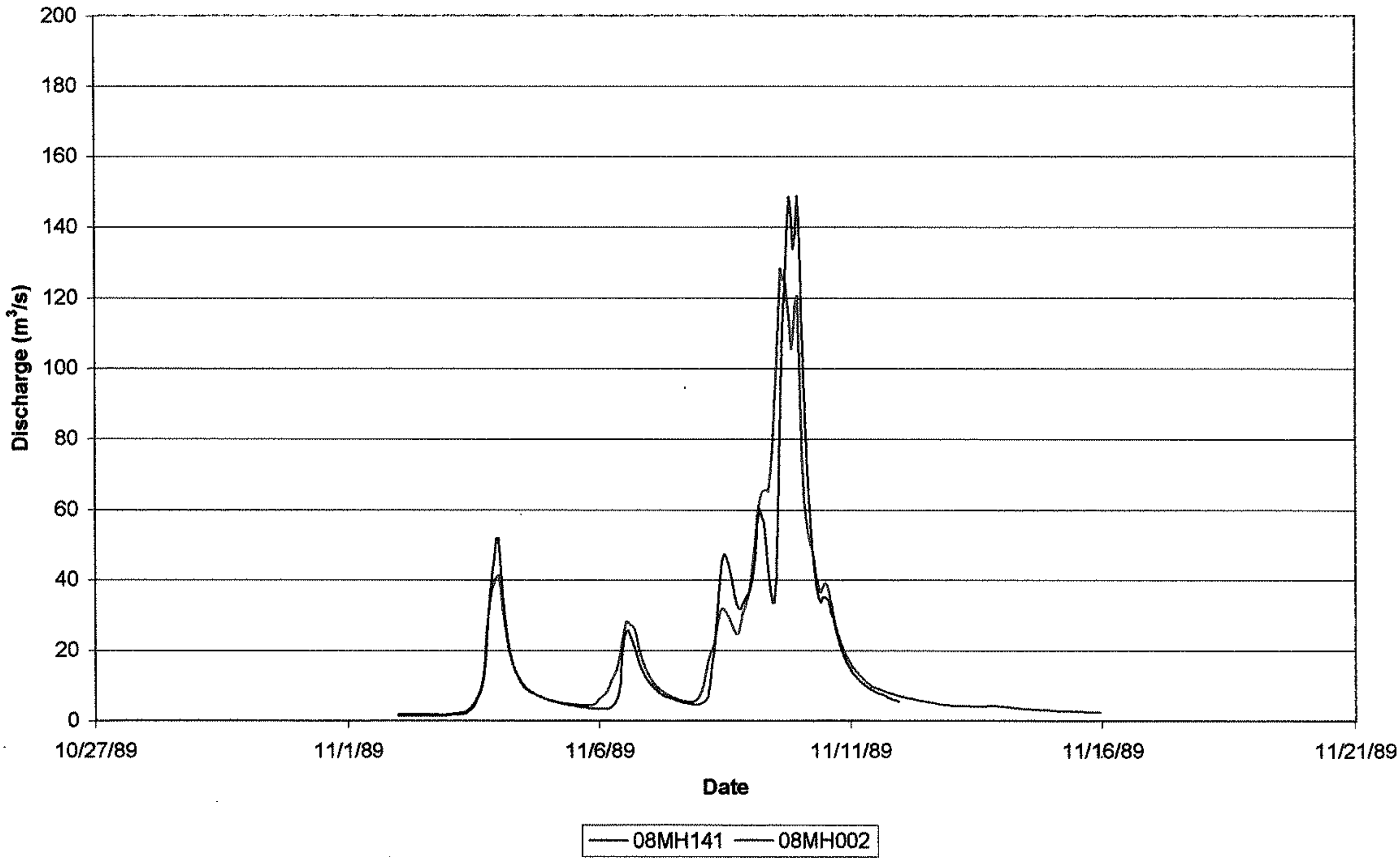


Figure 4.1 - 1989 Storm Hydrographs



WATER MANAGEMENT CONSULTANTS

Figure 4.2 - 1995 Storm Hydrographs

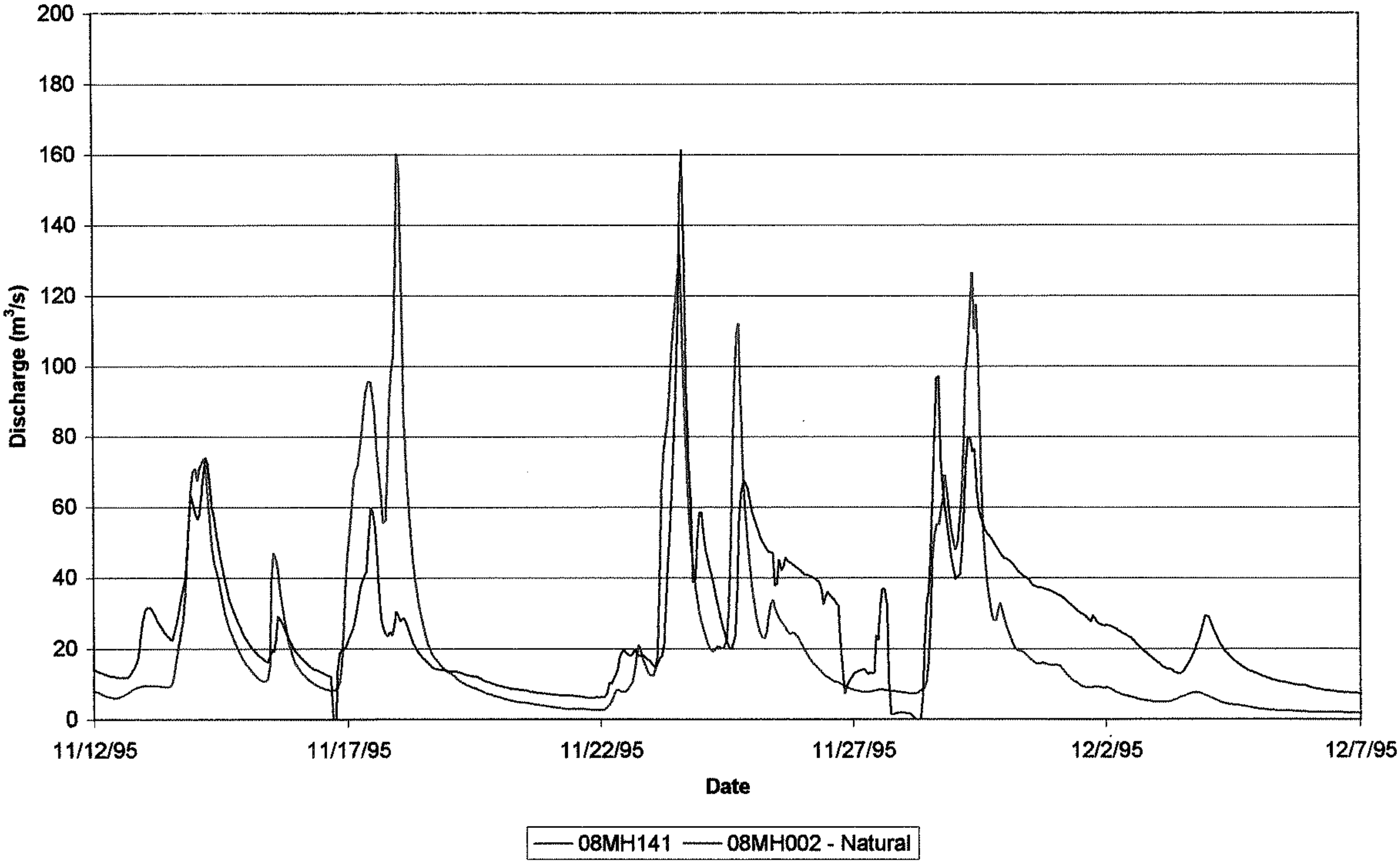
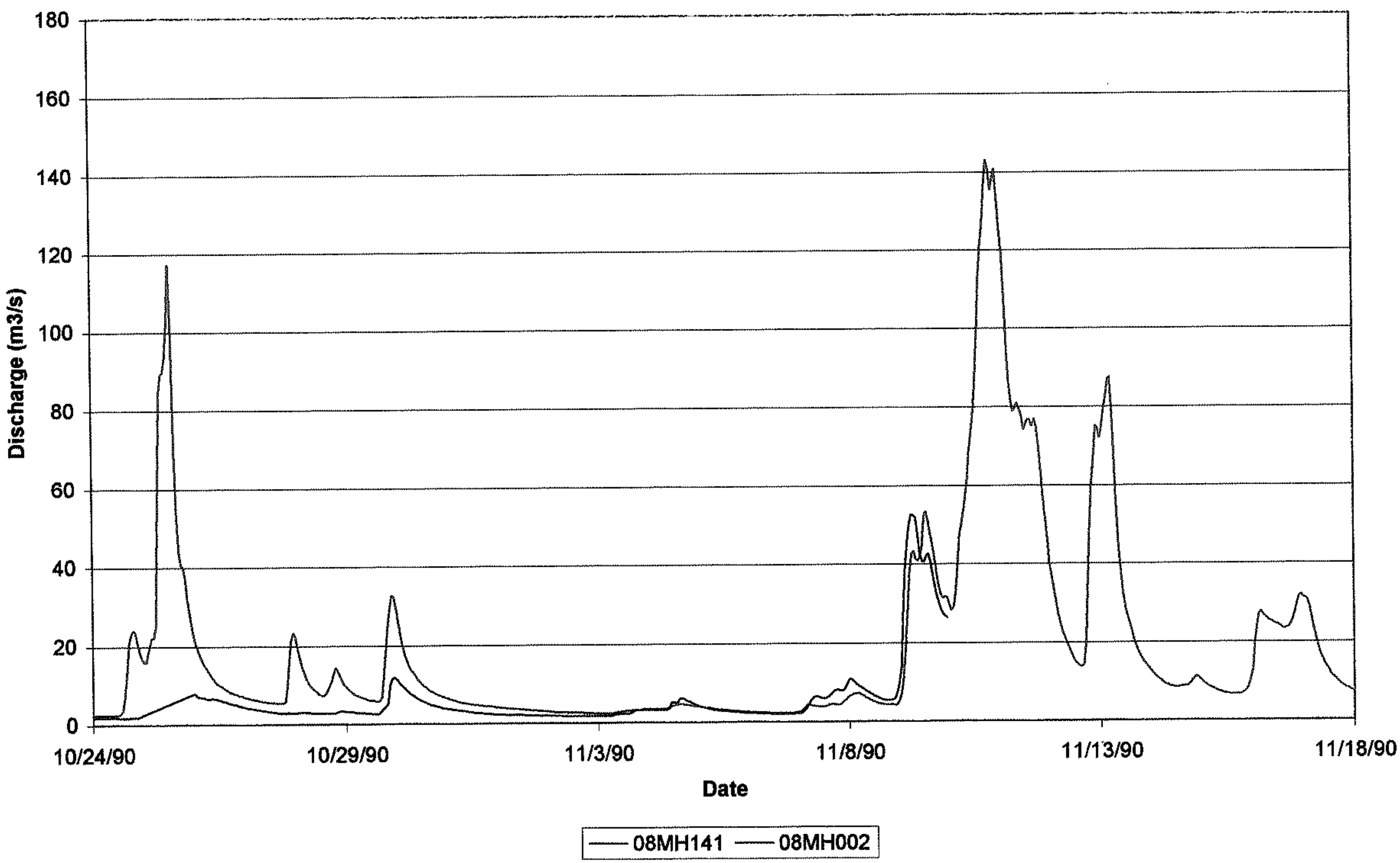
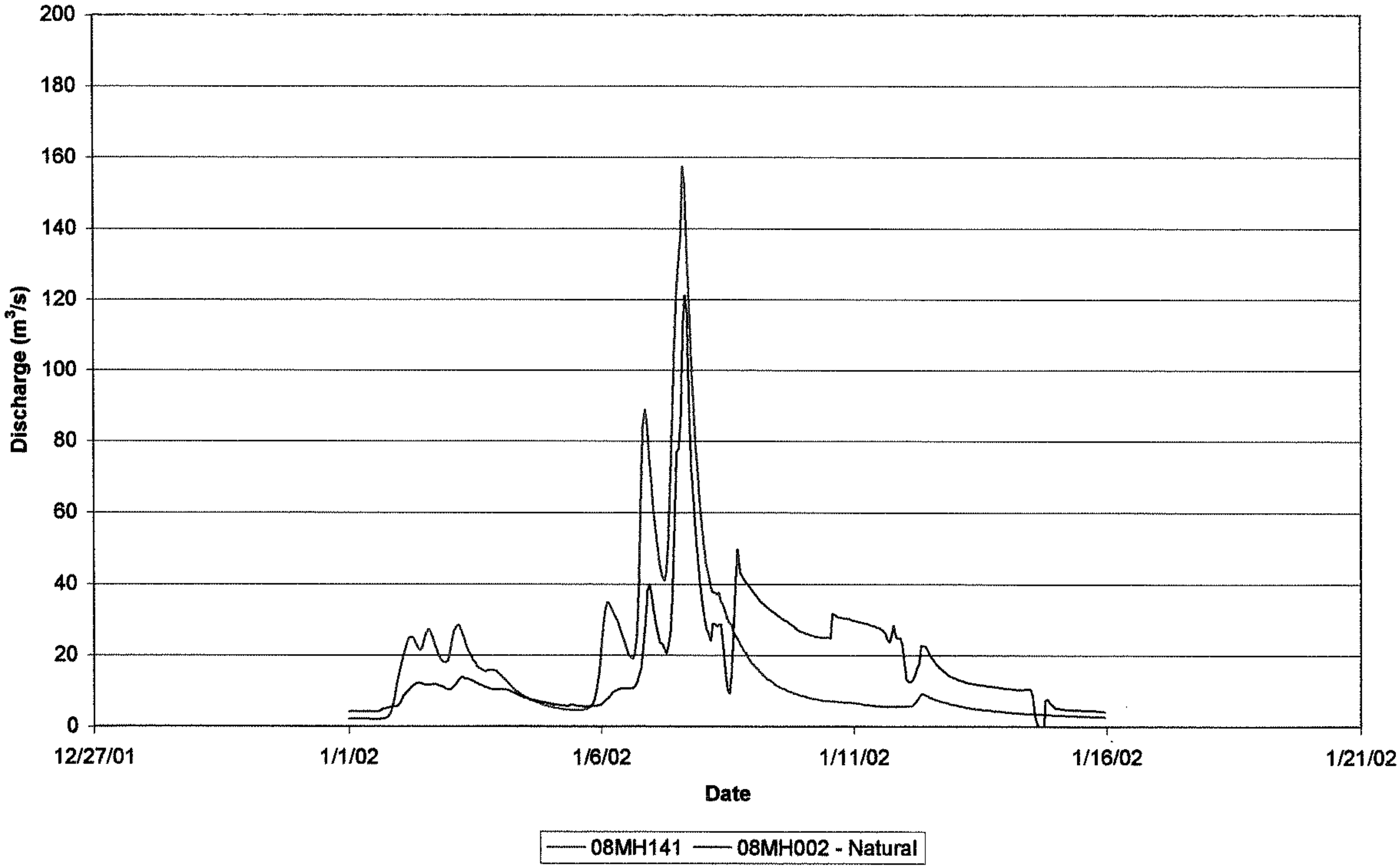


Figure 4.3 - 1990 Storm Hydrographs



WATER MANAGEMENT CONSULTANTS

Figure 4.4 - 2002 Storm Hydrographs



WATER MANAGEMENT CONSULTANTS

Figure 4.5 - 2003 Storm Hydrographs

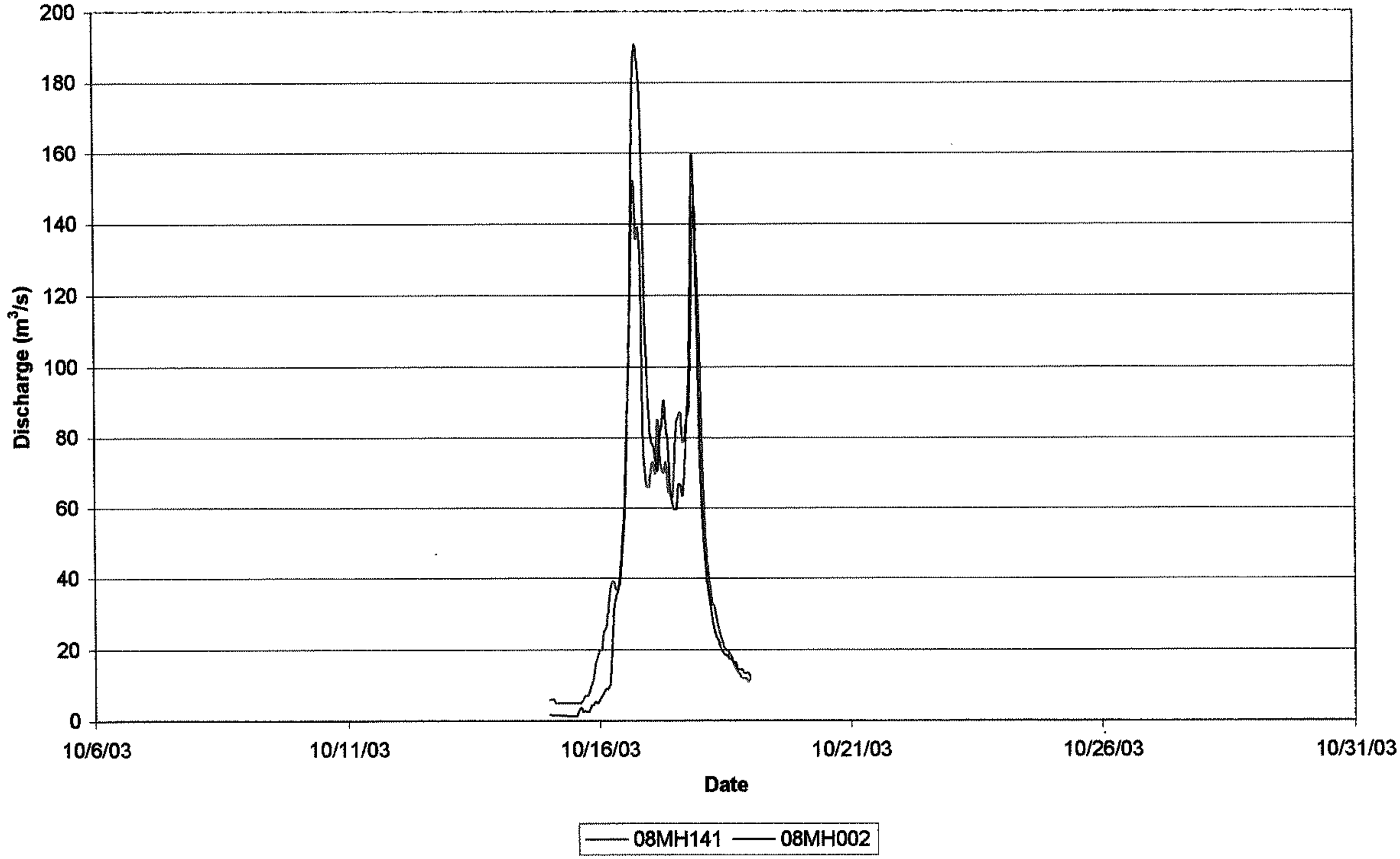
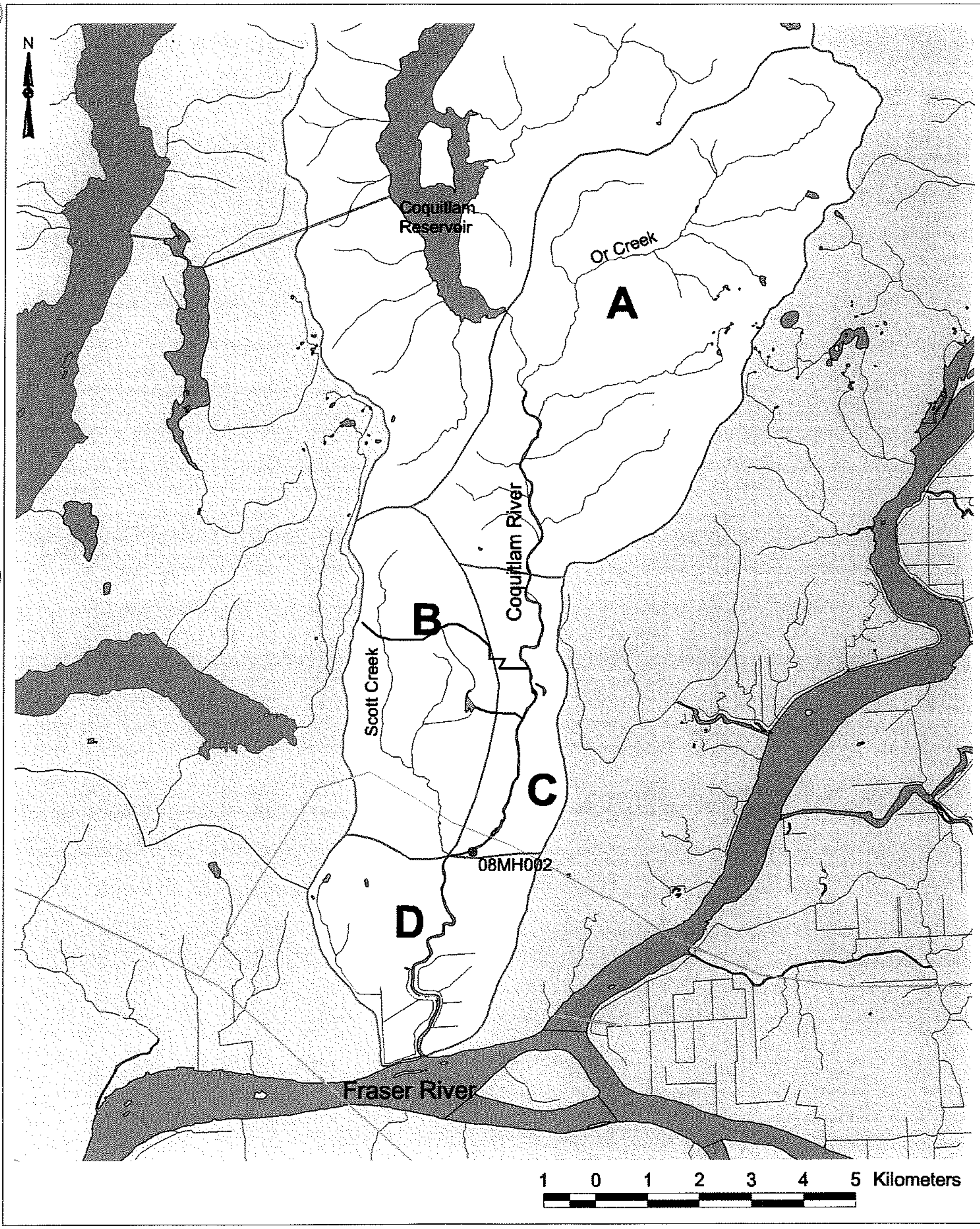
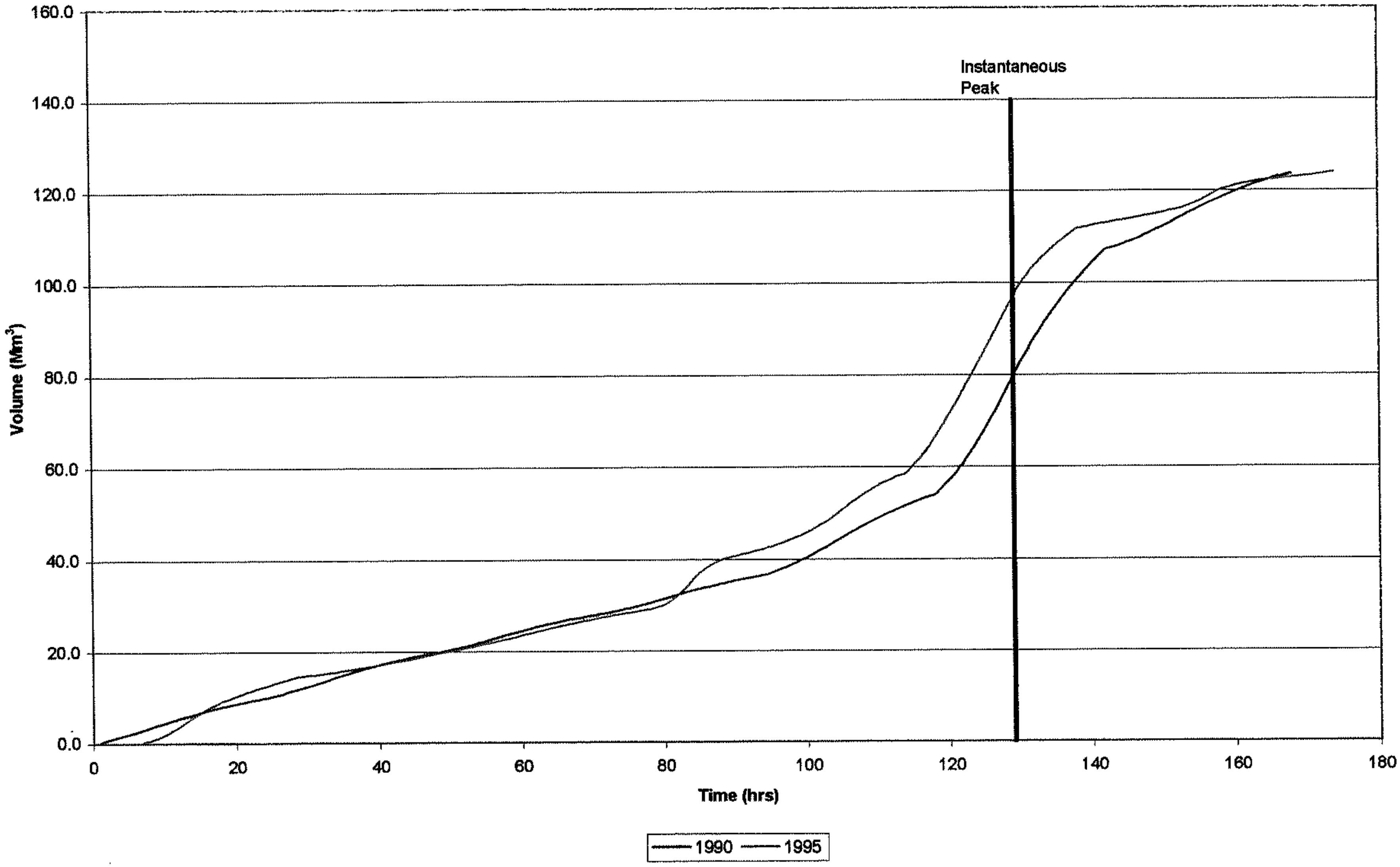


Figure 4.6 Catchment Areas Downstream of Coquitlam Dam



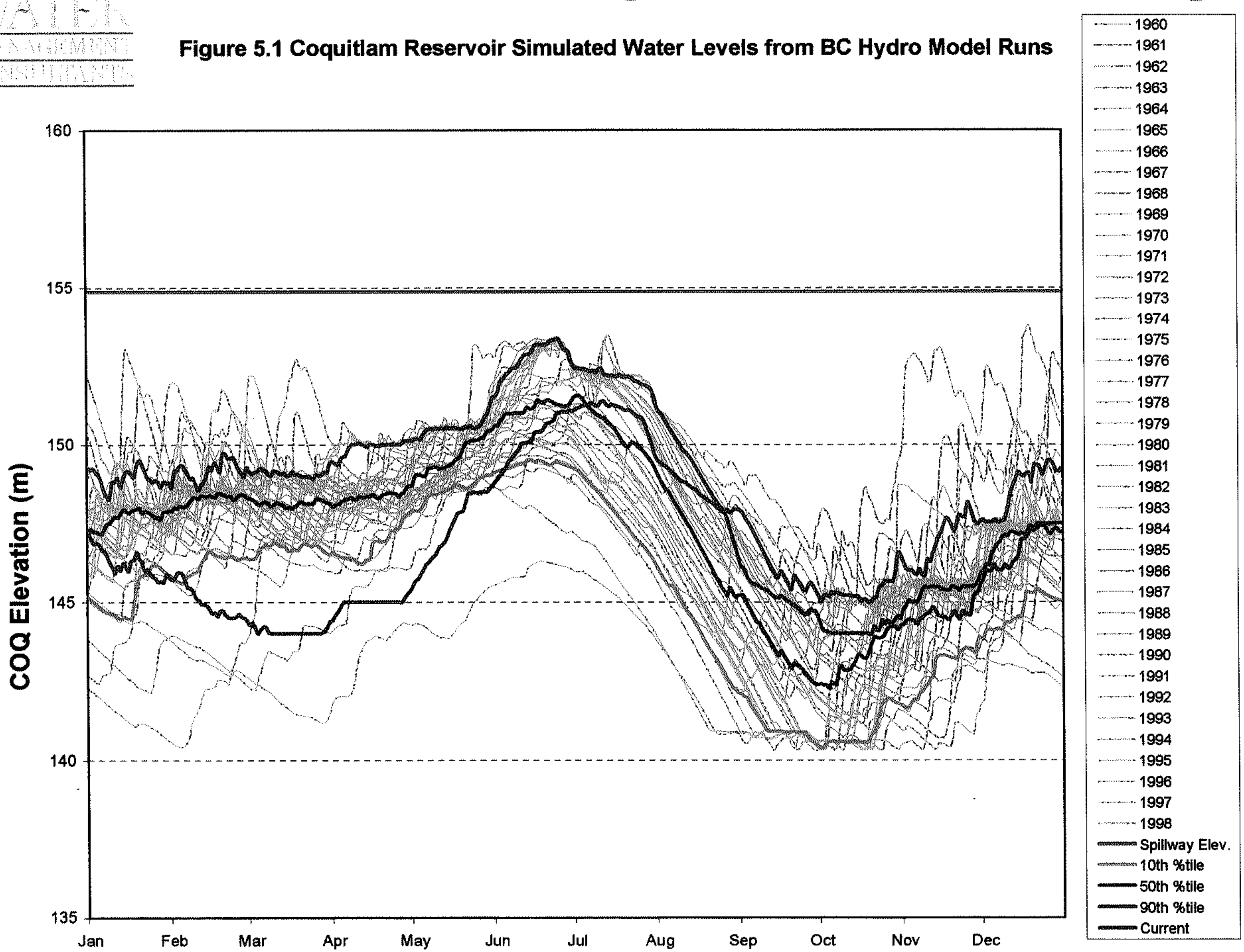
WATER MANAGEMENT CONSULTANTS

Figure 4.7 - Cumulative Volumes for Synthetic 1990 and 1995 Hydrographs



Water Management Consultants

Figure 5.1 Coquitlam Reservoir Simulated Water Levels from BC Hydro Model Runs



WATER MANAGEMENT CONSULTANTS

Figure 5.2 Coquitlam Reservoir Starting Water Level Elevation-Duration Curves for High Inflows

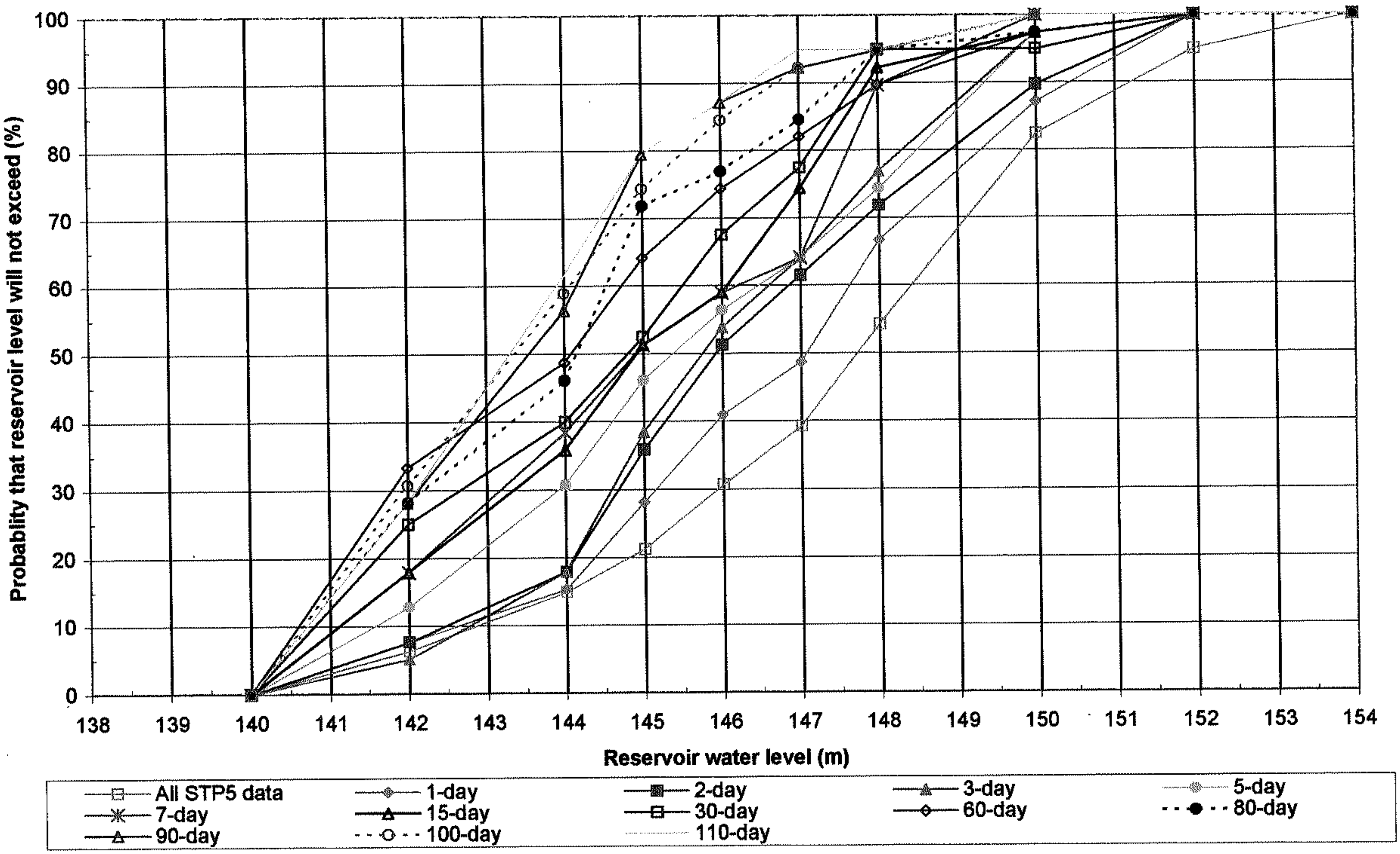


Figure 5.3 Starting reservoir water levels (50% probability of occurrence)

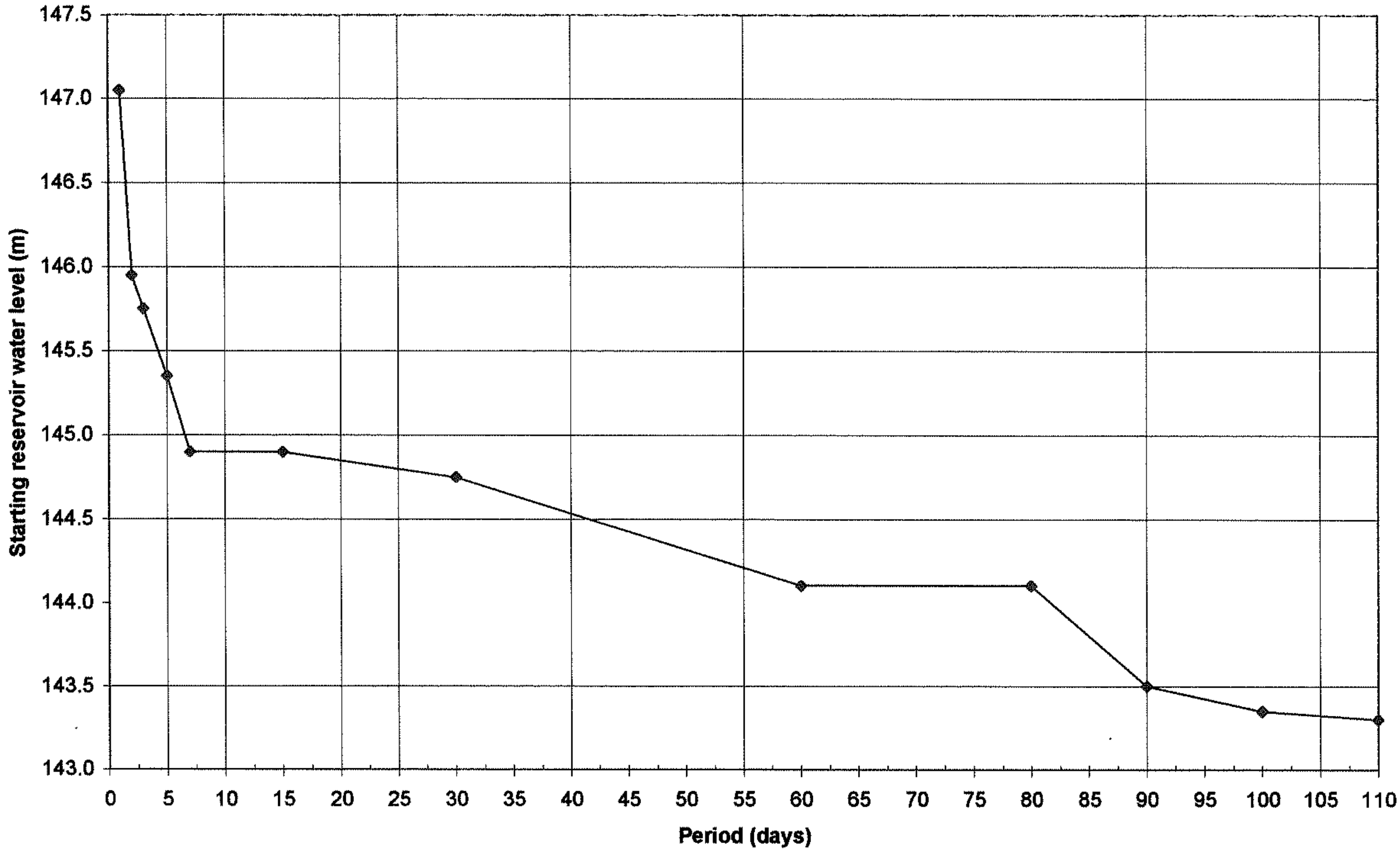
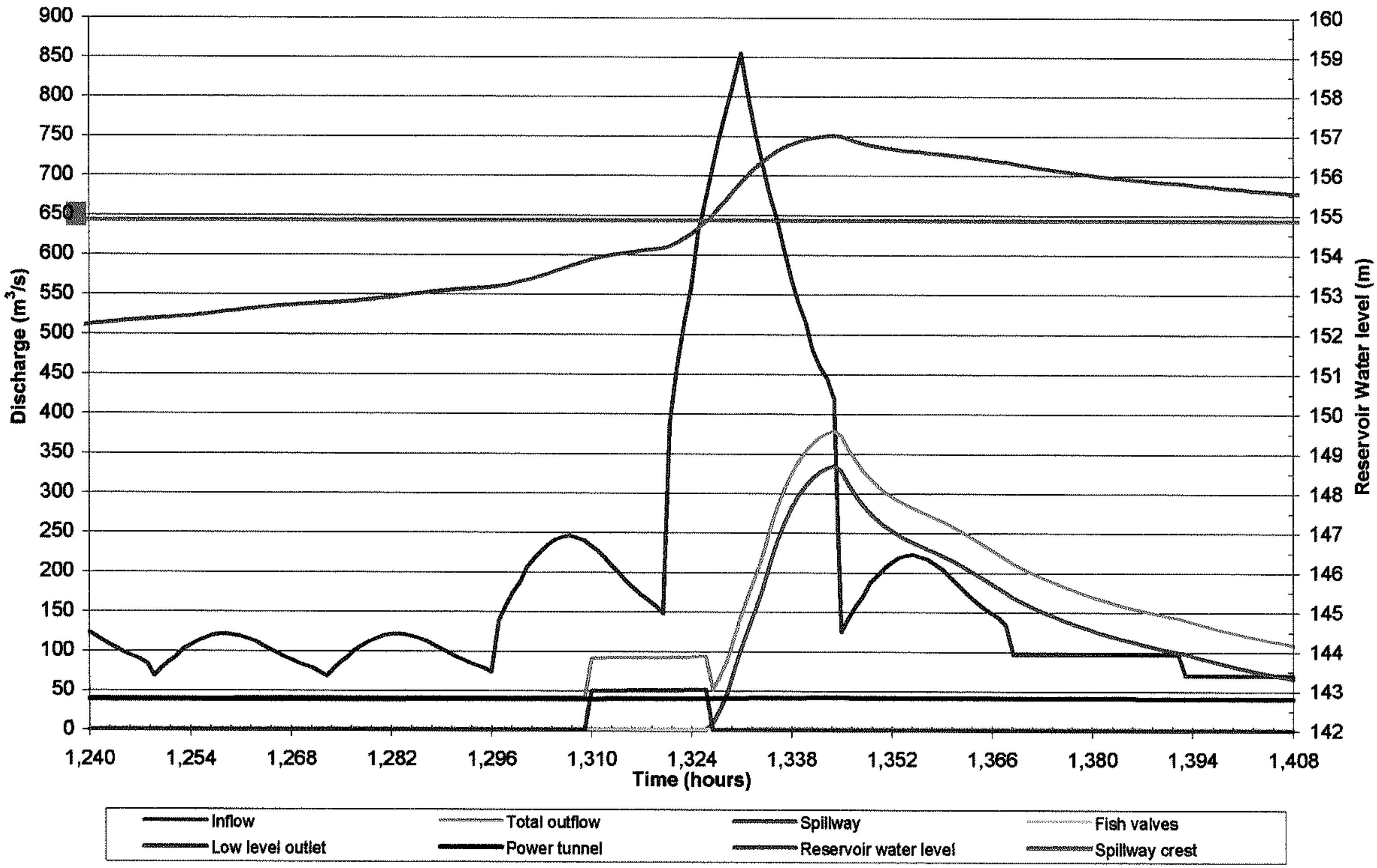
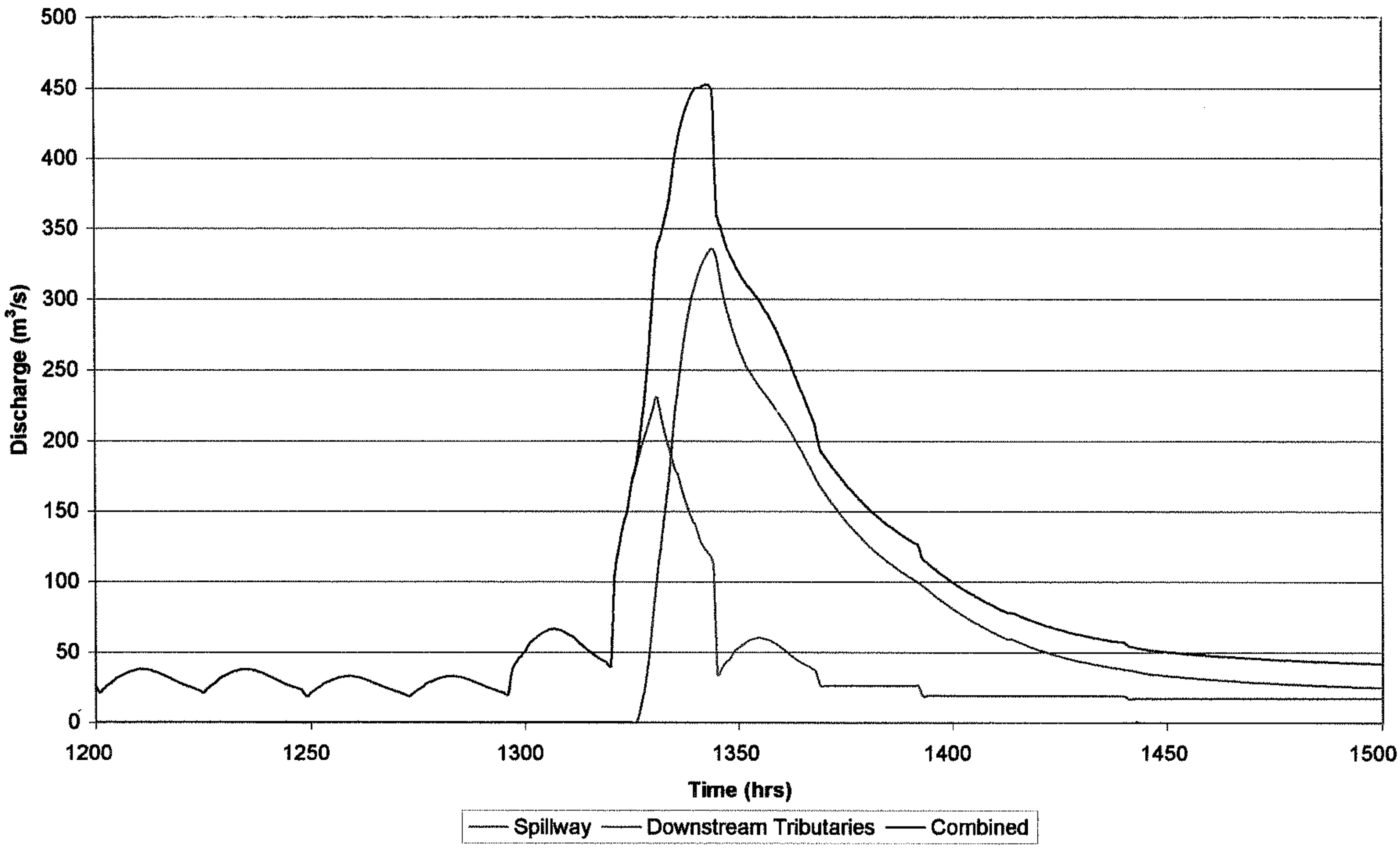


Figure 6.1 - Synthetic (1990) 200-Year Reservoir Routing Hydrographs
110-day hydrograph (Peak Period) - Starting reservoir WL 143.3 m



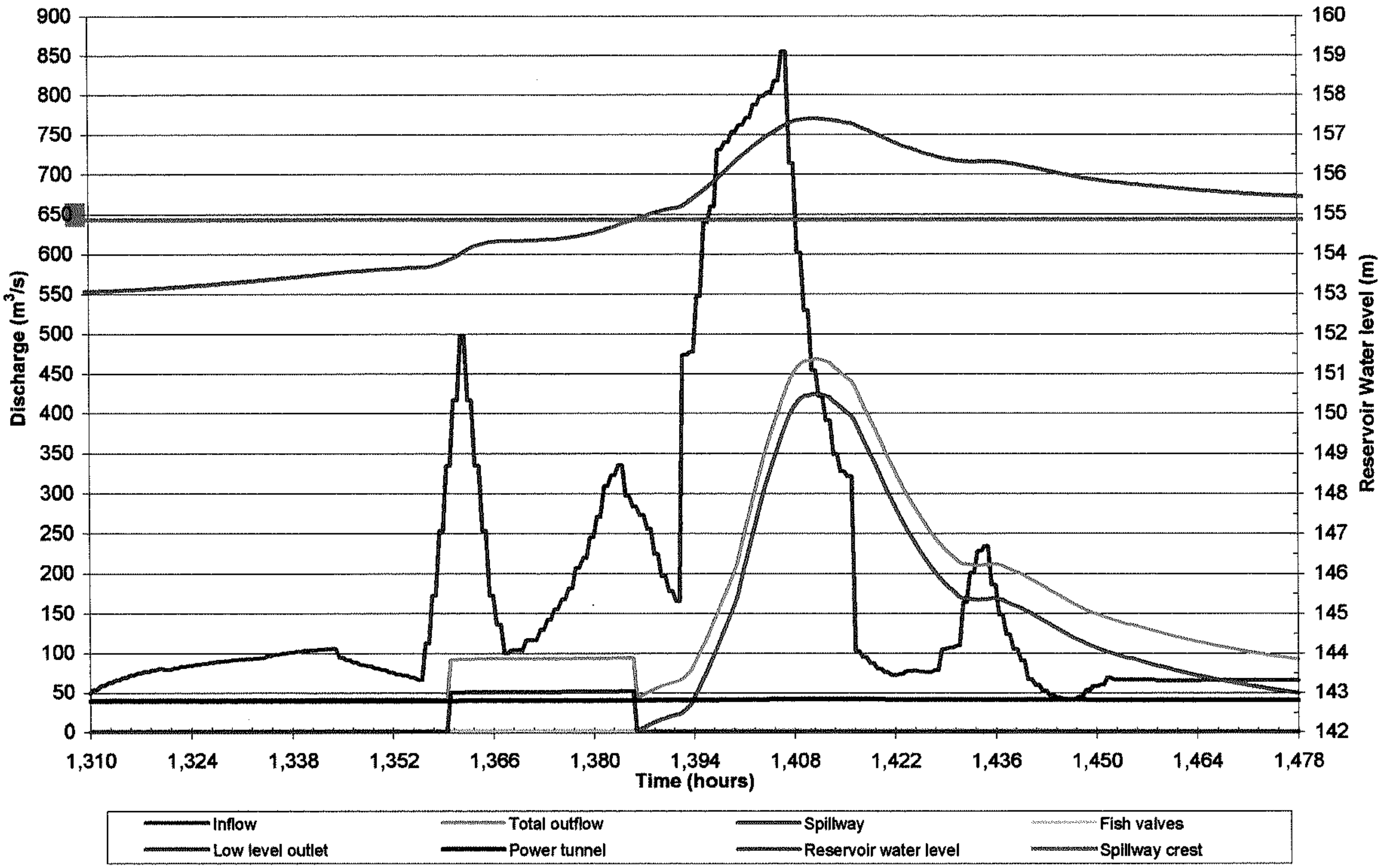
WATER MANAGEMENT CONSULTANTS

Figure 6.2 - Synthetic (1990) 200-Year Spillway Outflow and Downstream Tributary Hydrographs



WATER MANAGEMENT CONSULTANTS

Figure 6.3 - Synthetic (1995) 200-Year Reservoir Routing Hydrographs
110-day hydrograph (Peak Period) - Starting reservoir WL 143.3 m



WATER MANAGEMENT CONSULTANTS

Figure 6.4 - Synthetic (1995) 200-Year Spillway Outflow and Downstream Tributary Hydrographs

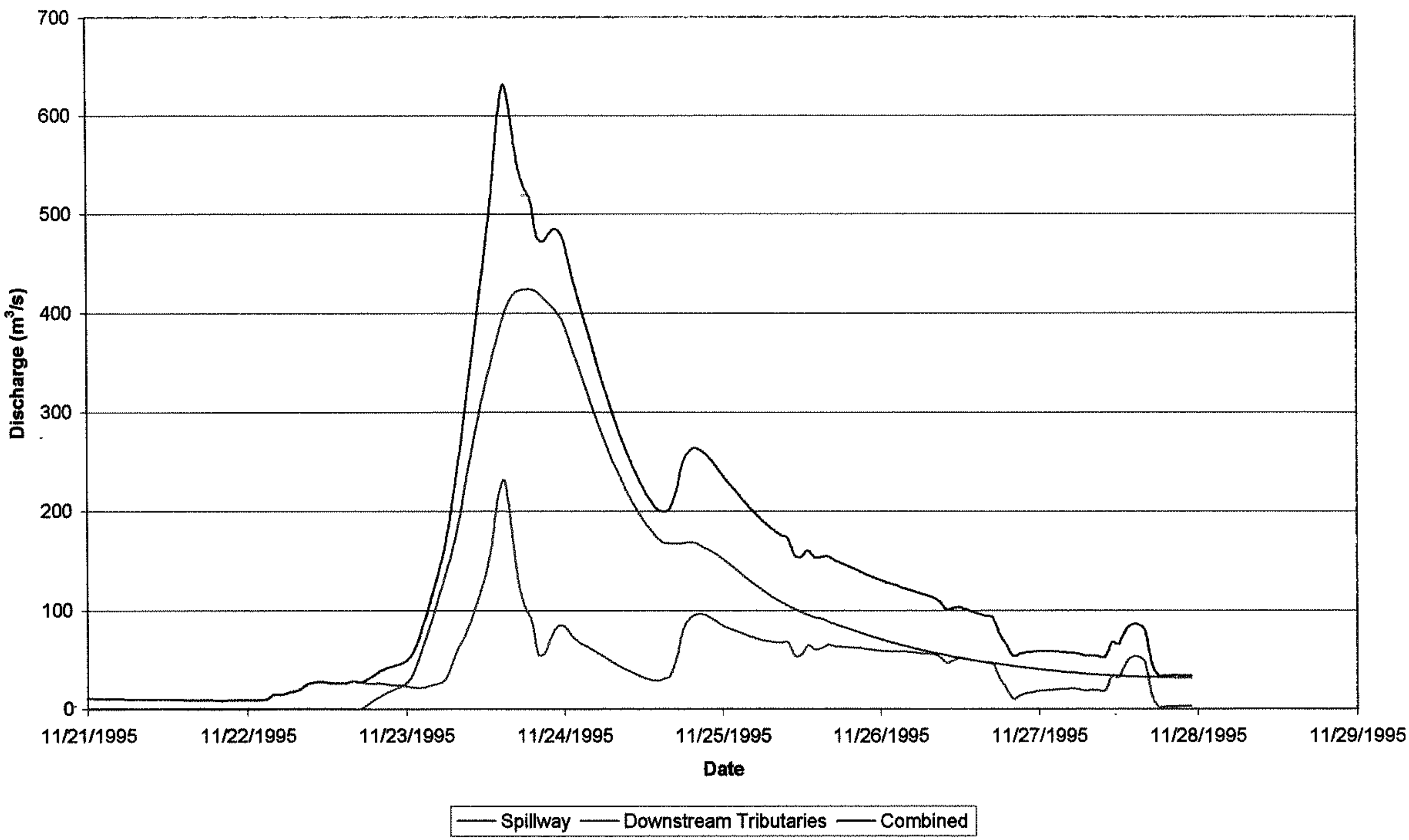


Figure 6.5 - 2003 Spillway Outflow and Downstream Tributary Hydrographs

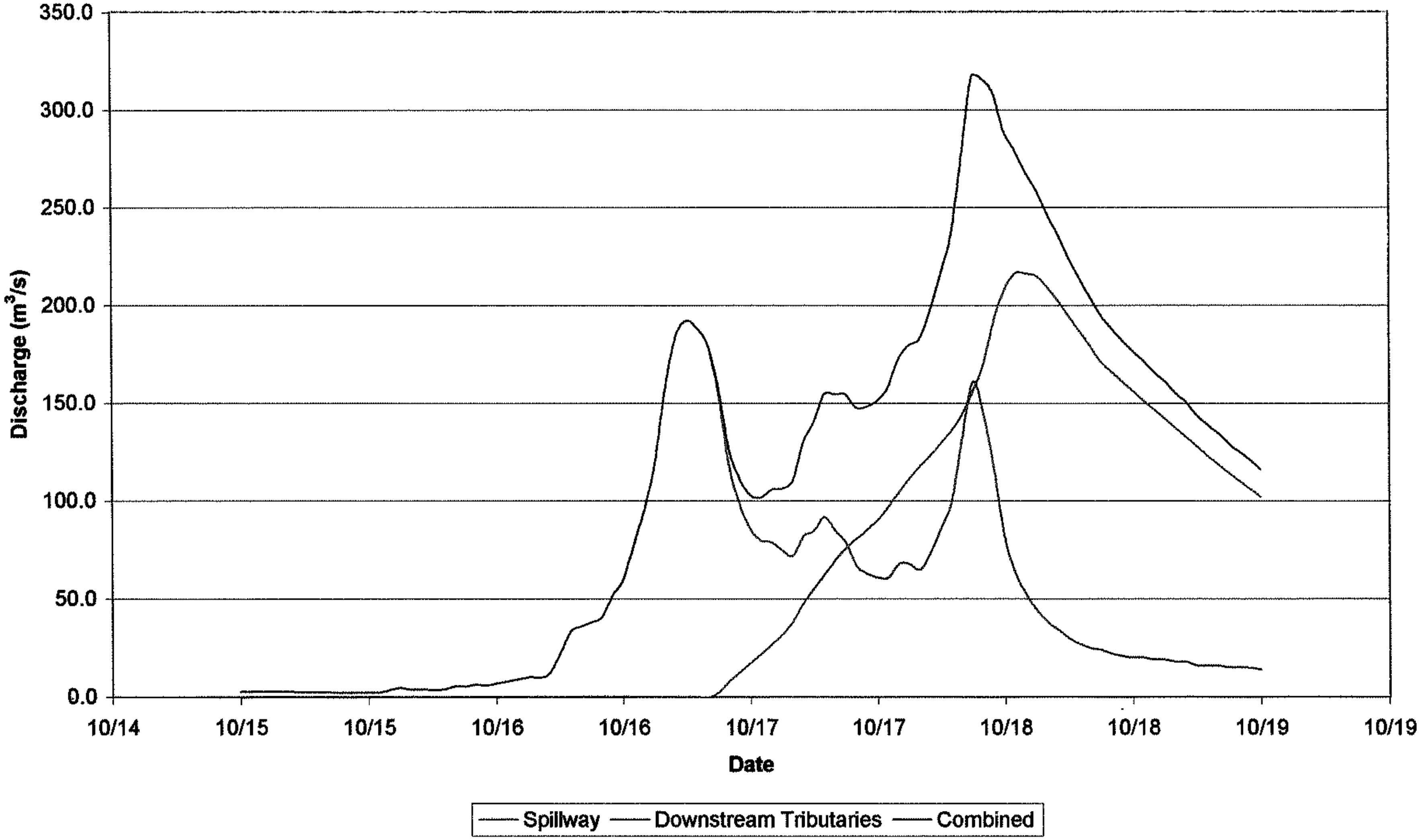
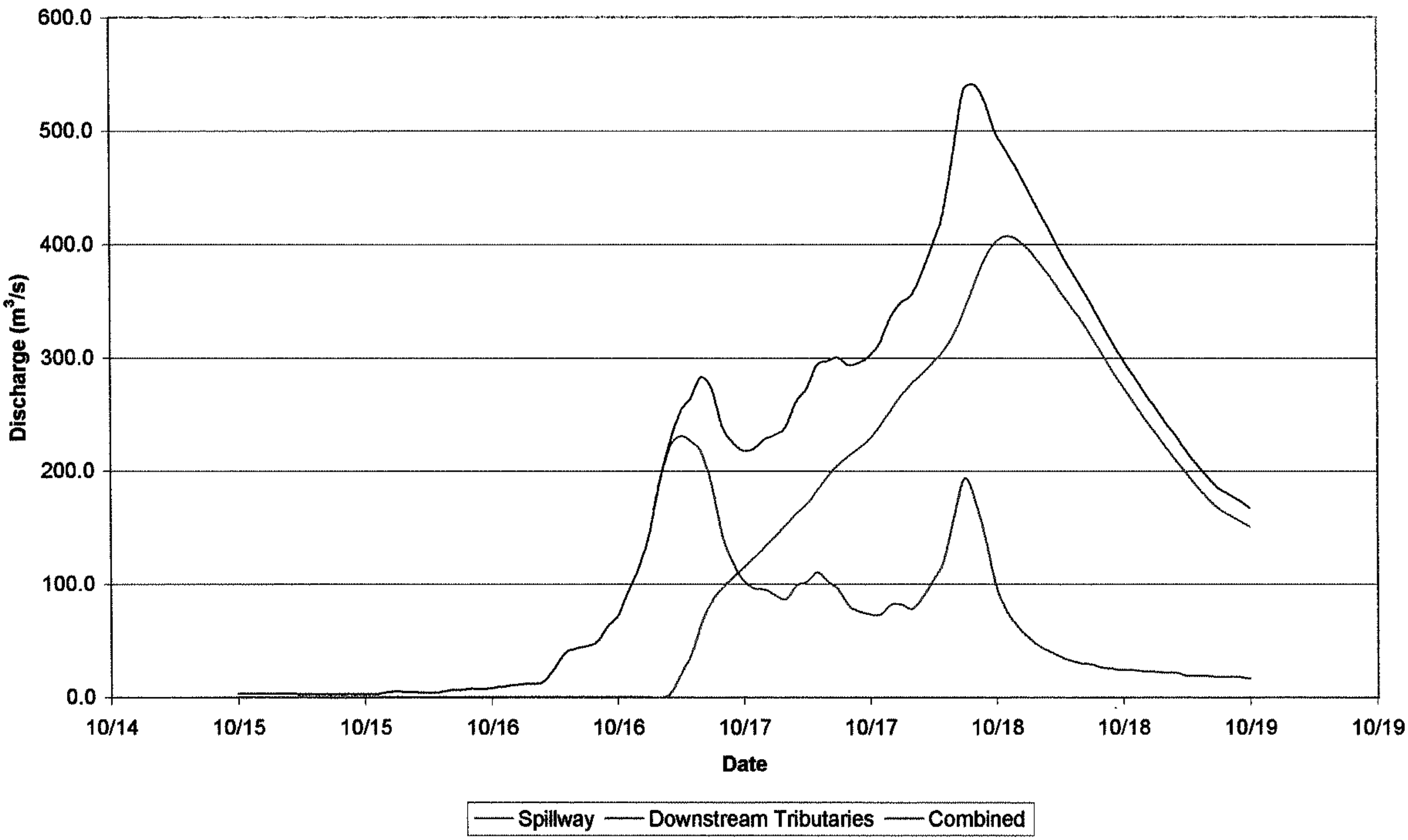


Figure 6.6 - Synthetic (2003) 200-Year Spillway Outflow and Downstream Tributary Hydrographs



WATER MANAGEMENT CONSULTANTS

Figure 6.7 - Synthetic (1990) 200-Year Reservoir Routing Hydrographs
Complete 110-day hydrograph - Starting reservoir WL 143.3m

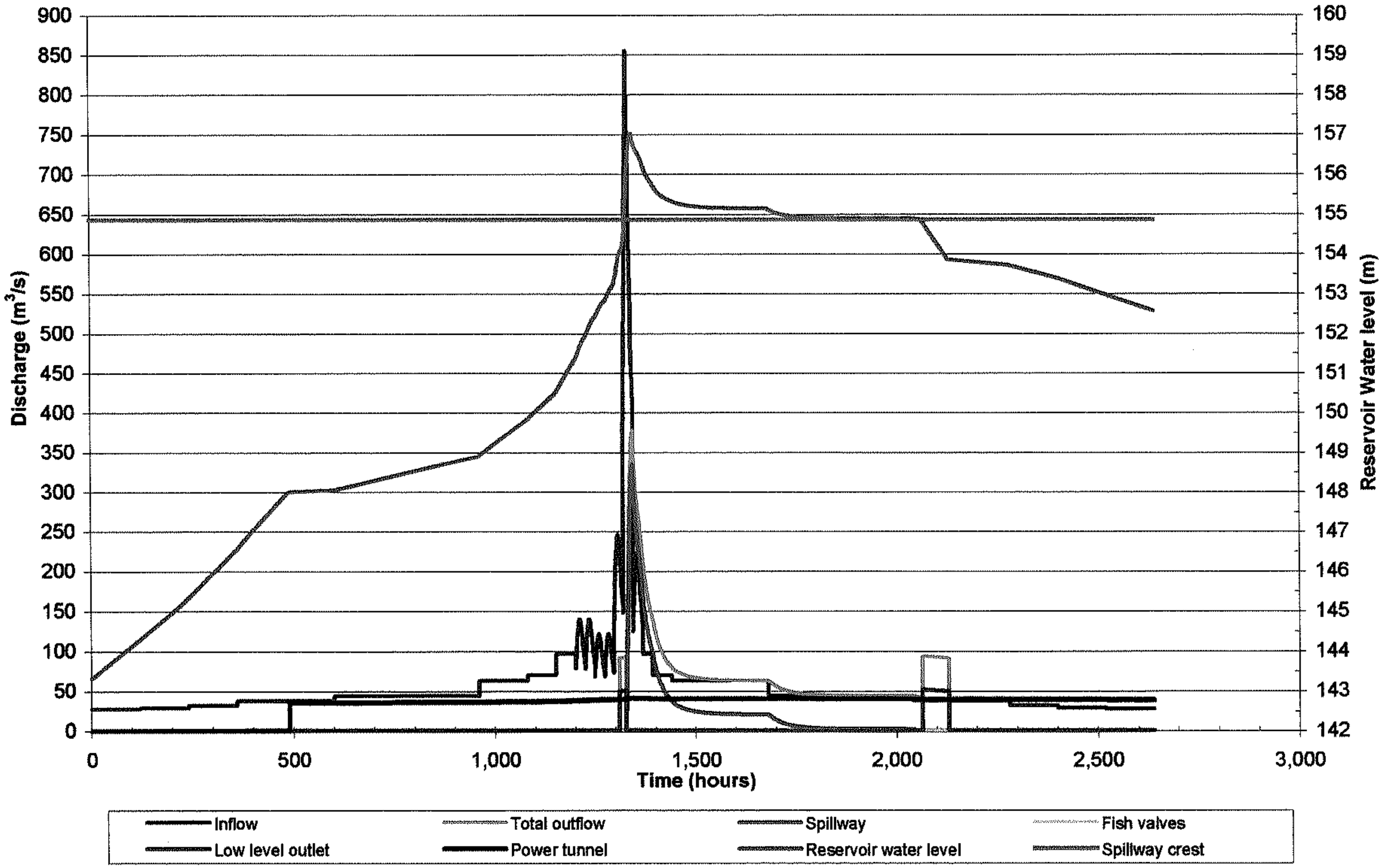
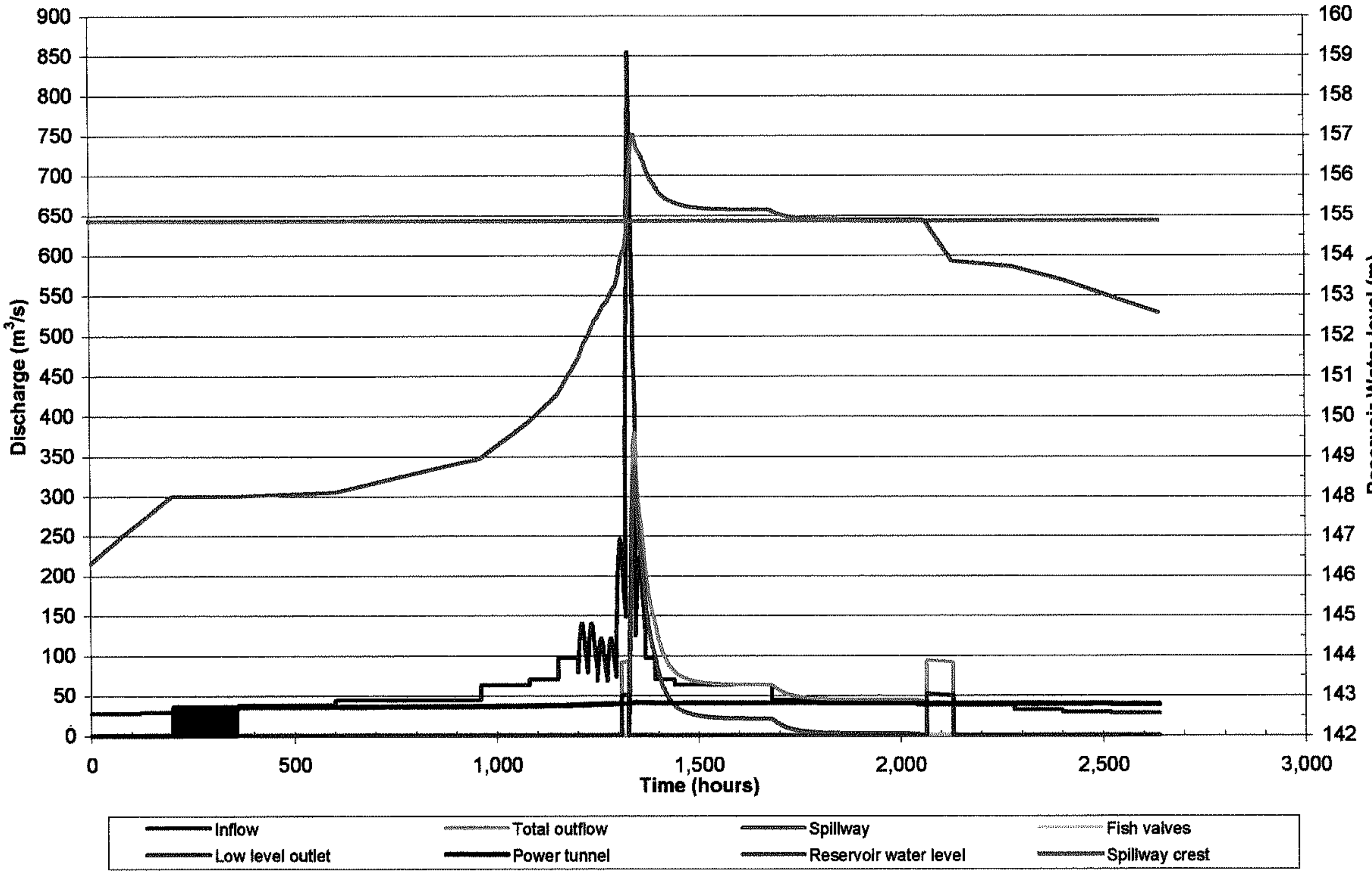
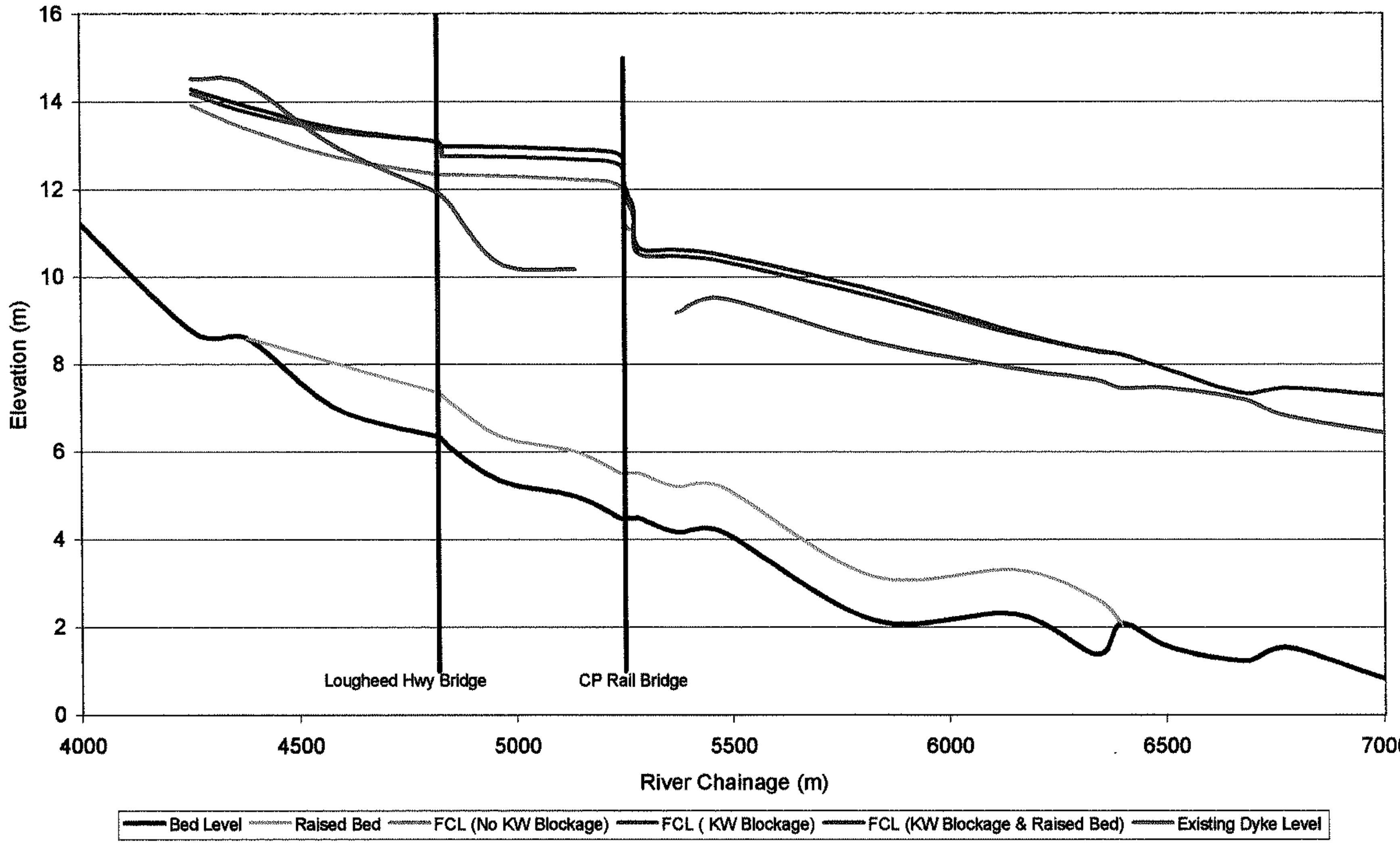


Figure 6.8 - Synthetic (1990) 200-Year Reservoir Routing Hydrographs
Complete 110-day hydrograph - Starting reservoir WL 146.3m



WATER MANAGEMENT CONSULTANTS

Figure 6.9 - Water Surface Profile Sensitivity



Note: KW - Kingway Bridge

WATER MANAGEMENT CONSULTANTS

Figure 8.1 - Modelled Winter and Spring 200-Year Water Level Profiles

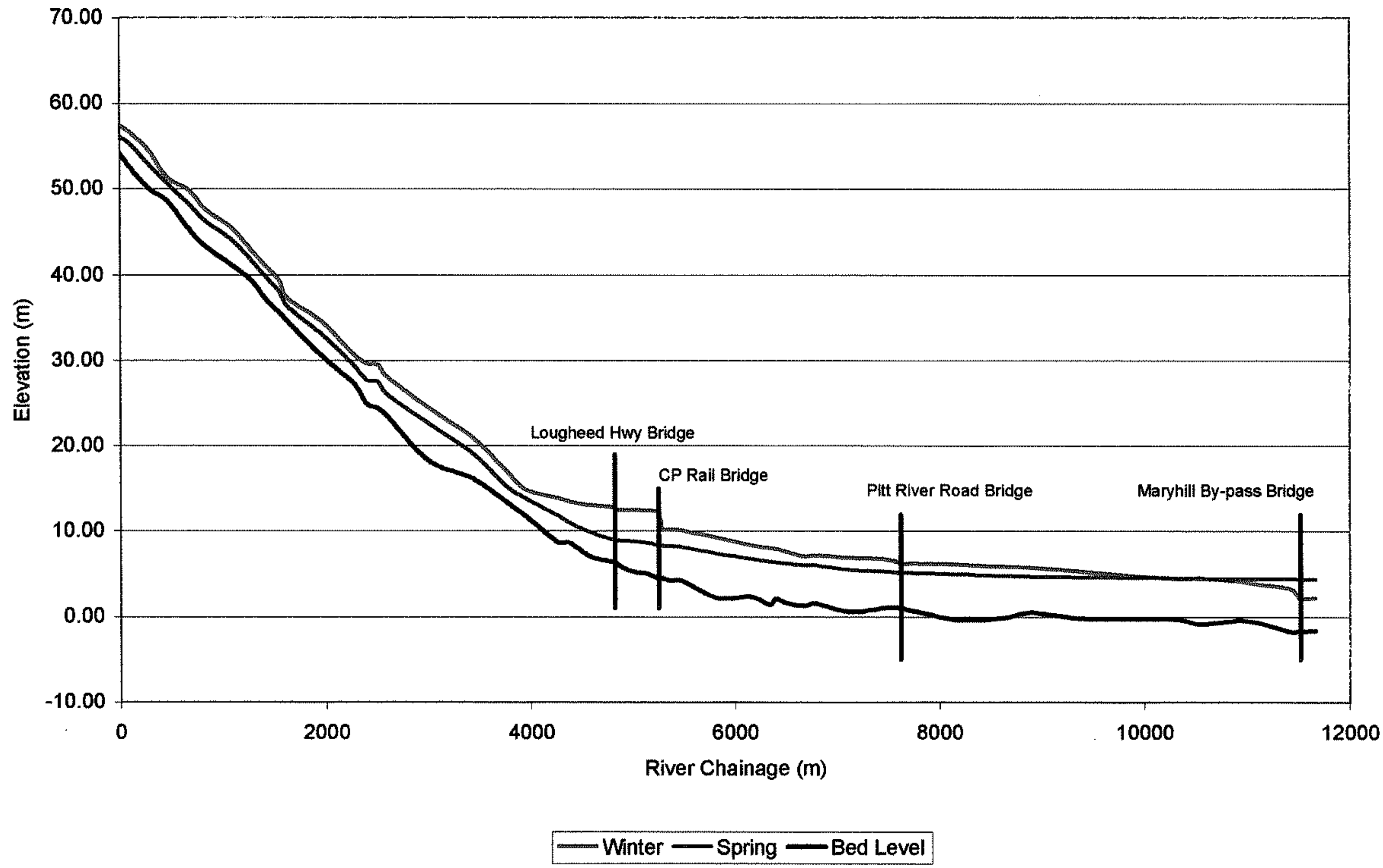
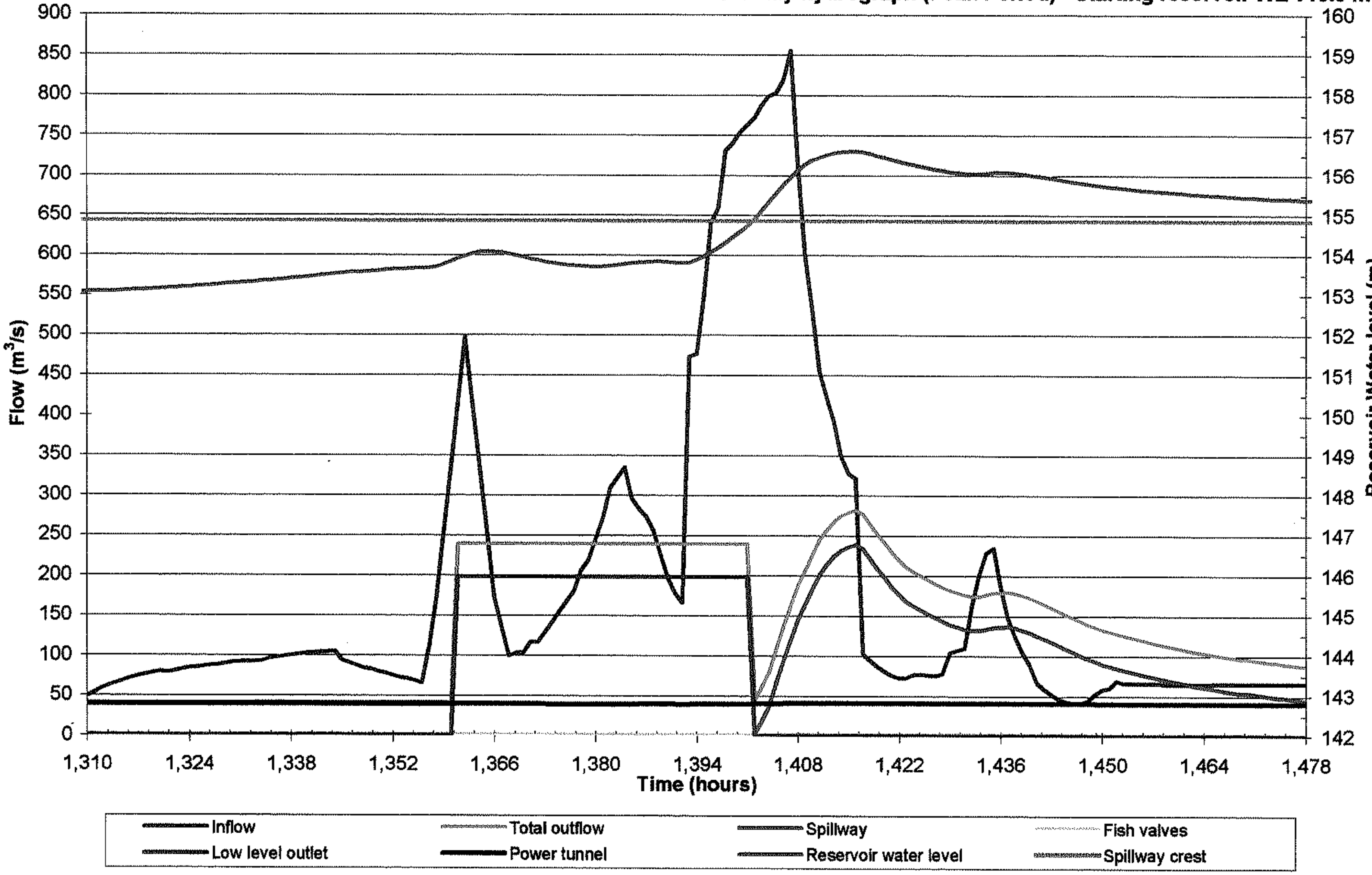
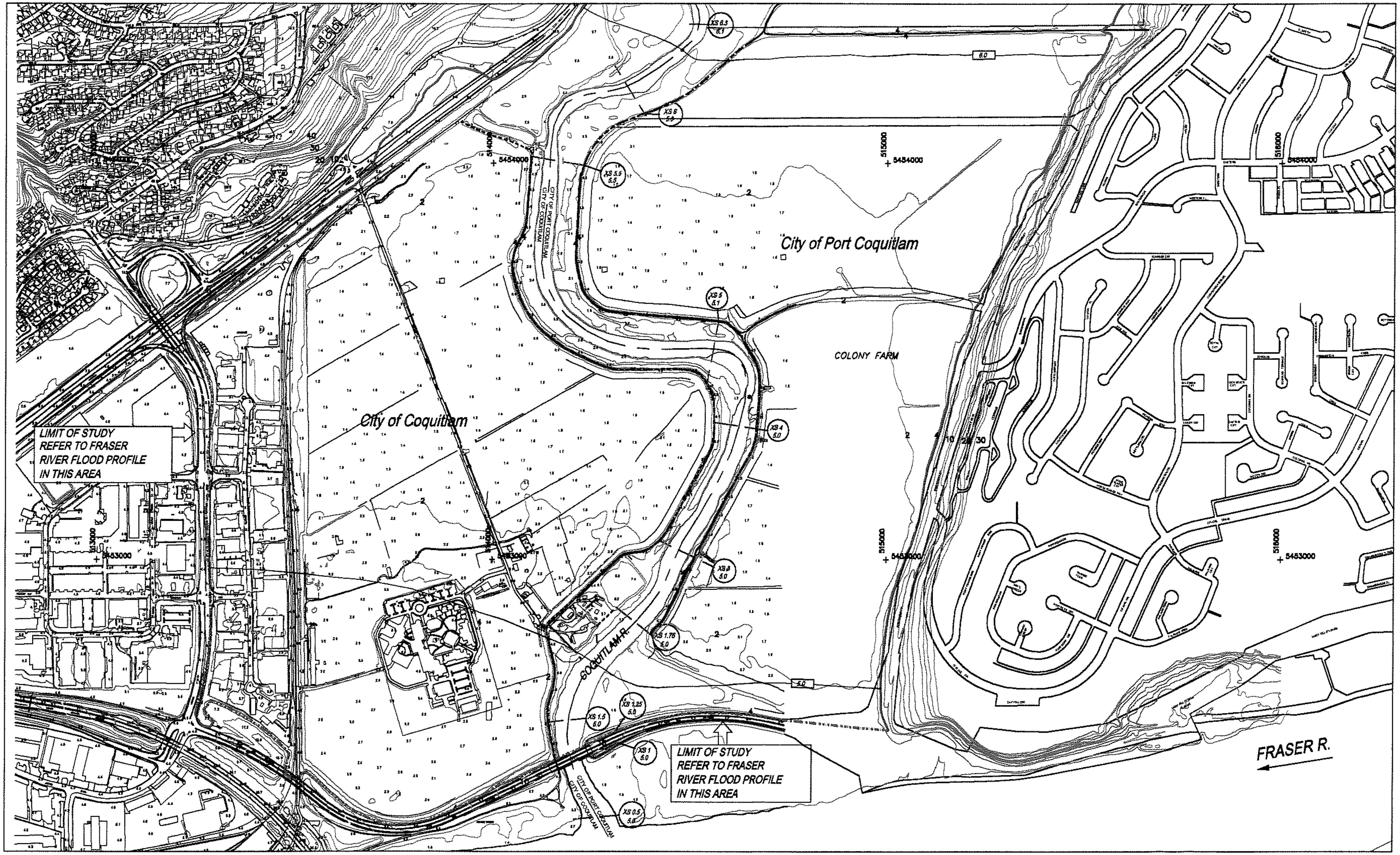


Figure 9.1 - Increased low level outlet capacity: Synthetic (1995) 200-Year Reservoir Routing Hydrographs 110-day hydrograph (Peak Period) - Starting reservoir WL 143.3 m



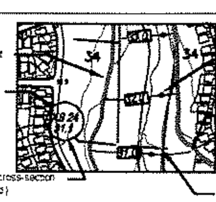
DRAWINGS



FLOODPLAIN DATA

1. These maps are available from the Cities of Coquitlam and Port Coquitlam
2. Flooding may still occur outside of the designated floodplain areas. The Cities of Coquitlam and Port Coquitlam and the Province do not assume any liability by reason of the failure to designate areas on this map
3. There is a one in 200 chance that the Designated flood could be equaled or exceeded in any one year
4. The base mapping has been produced by combining 2 m contour interval maps supplied by the Cities of Coquitlam and Port Coquitlam. Grid is to NAD 27 zone 10. Elevations refer to geodetic datum
5. The flood levels were computed using the HEC-RAS computer program version 3.1, and river cross sections surveyed in October-December 2000 and November 2001
6. The floodplain limits assume that dikes can fail
7. The floodplain limits and flood levels include a freeboard allowance
8. The floodplain limits are not established on the ground by legal survey. Building elevations should be based on field survey and established bench marks
9. The floodplain limits are not delineated for side streams and tributaries
10. The required setback of buildings from the natural boundaries, lakes and watercourses to allow for the passage of floodwaters and possible bank erosion is not shown. This information is available either through local municipalities or the Ministry of Water, Land and Air Protection

LEGEND



Study limits

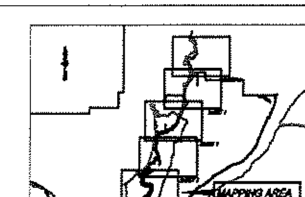
Non-standard dike

Standard dike

4 Contour line

4 Spot elevation

KEY MAP



REVISIONS			
REV	DATE	BY	DESCRIPTION

PROJECT	7095
ISSUE OF MAPPING	March 2004
DRAWN	MG
ENGINEER	M.Garrett
CHECKED	
APPROVED	D.Sellars

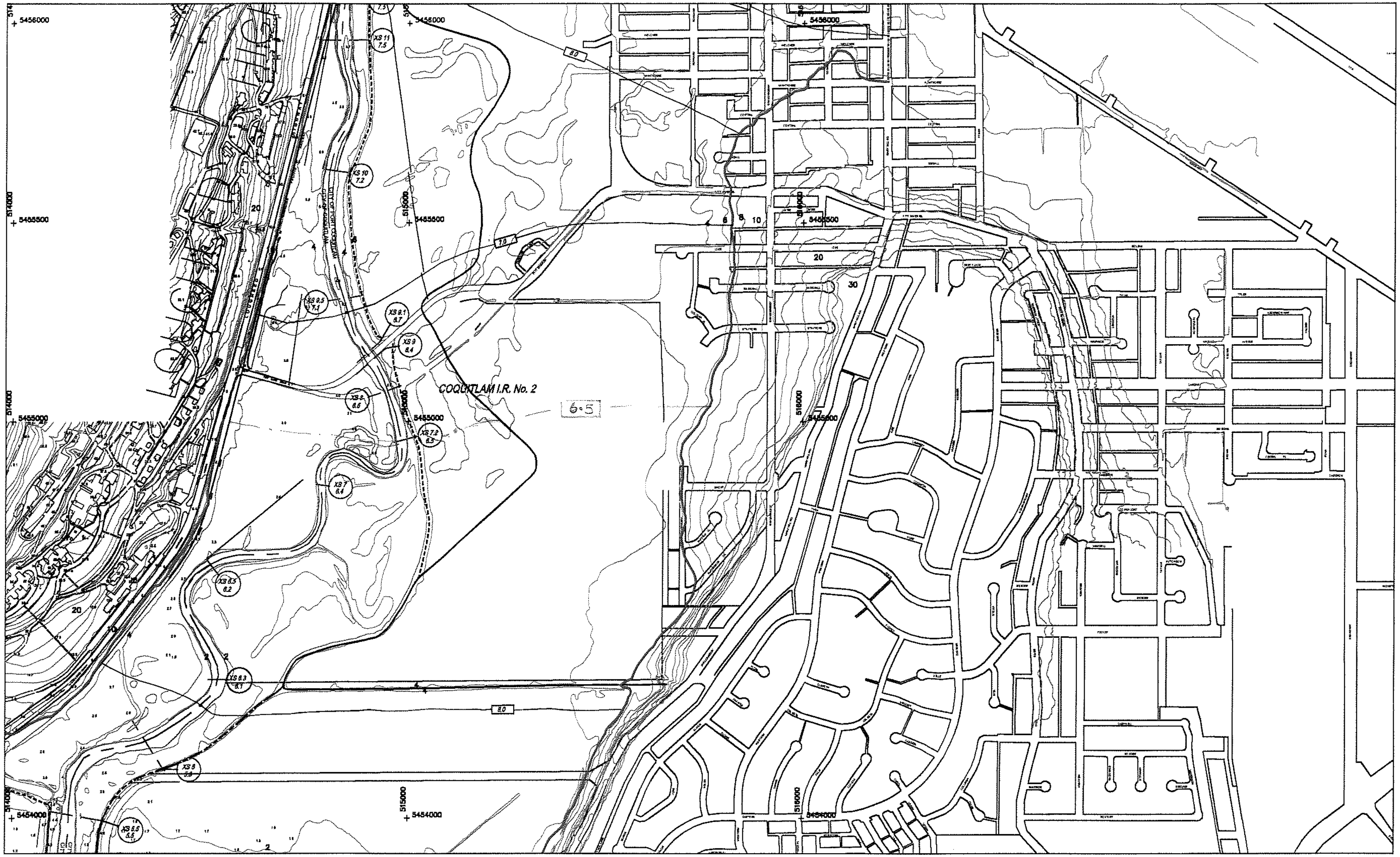
CITY OF PORT COQUITLAM

Distance in Metres

COQUITLAM RIVER FLOODPLAIN MAPS:
FRASER RIVER TO HOCKADAY STREET

Water Management Consultants Inc.
130-10691 Shellbridge Way,
Richmond, B.C. Canada V6X 2W8

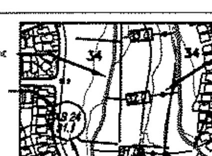
DRAWING NO.	7095 - 2
SHEET	1 OF 5
REVISION	



FLOODPLAIN DATA

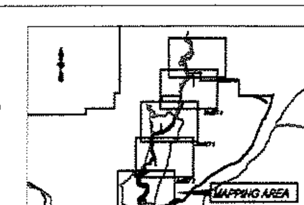
1. These maps are available from the Cities of Coquitlam and Port Coquitlam
2. Flooding may still occur outside of the designated floodplain areas. The Cities of Coquitlam and Port Coquitlam and the Province do not assume any liability by reason of the failure to designate areas on this map
3. There is a one in 200 chance that the Designated flood could be equaled or exceeded in any one year
4. The base mapping has been produced by combining 2 m contour interval maps supplied by the Cities of Coquitlam and Port Coquitlam. Grid is to NAD 27 zone 10. Elevations refer to geodetic datum.
5. The flood levels were computed using the HEC-RAS computer program version 3.1, and river cross sections surveyed in October-December 2000 and November 2001
6. The floodplain limits assume that dikes can fail
7. The floodplain limits and flood levels include a freeboard allowance
8. The floodplain limits are not established on the ground by legal survey. Building elevations should be based on field survey and established bench marks
9. The floodplain limits are not delineated for side streams and tributaries
10. The required setback of buildings from the natural boundaries, lakes and watercourses to allow for the passage of floodwaters and possible bank erosion is not shown. This information is available either through local municipalities or the Ministry of Water, Land and Air Protection

LEGEND



	Study limits
	Non-standard dike
	Standard dike
4	Contour line
4	Spot elevation

KEY MAP



REVISIONS

REV	DATE	BY	DESCRIPTION

PROJECT	7095
ISSUE OF MAPPING	March 2004
DRAWN	MG
ENGINEER	M.Garrett
CHECKED	
APPROVED	D.Sellars

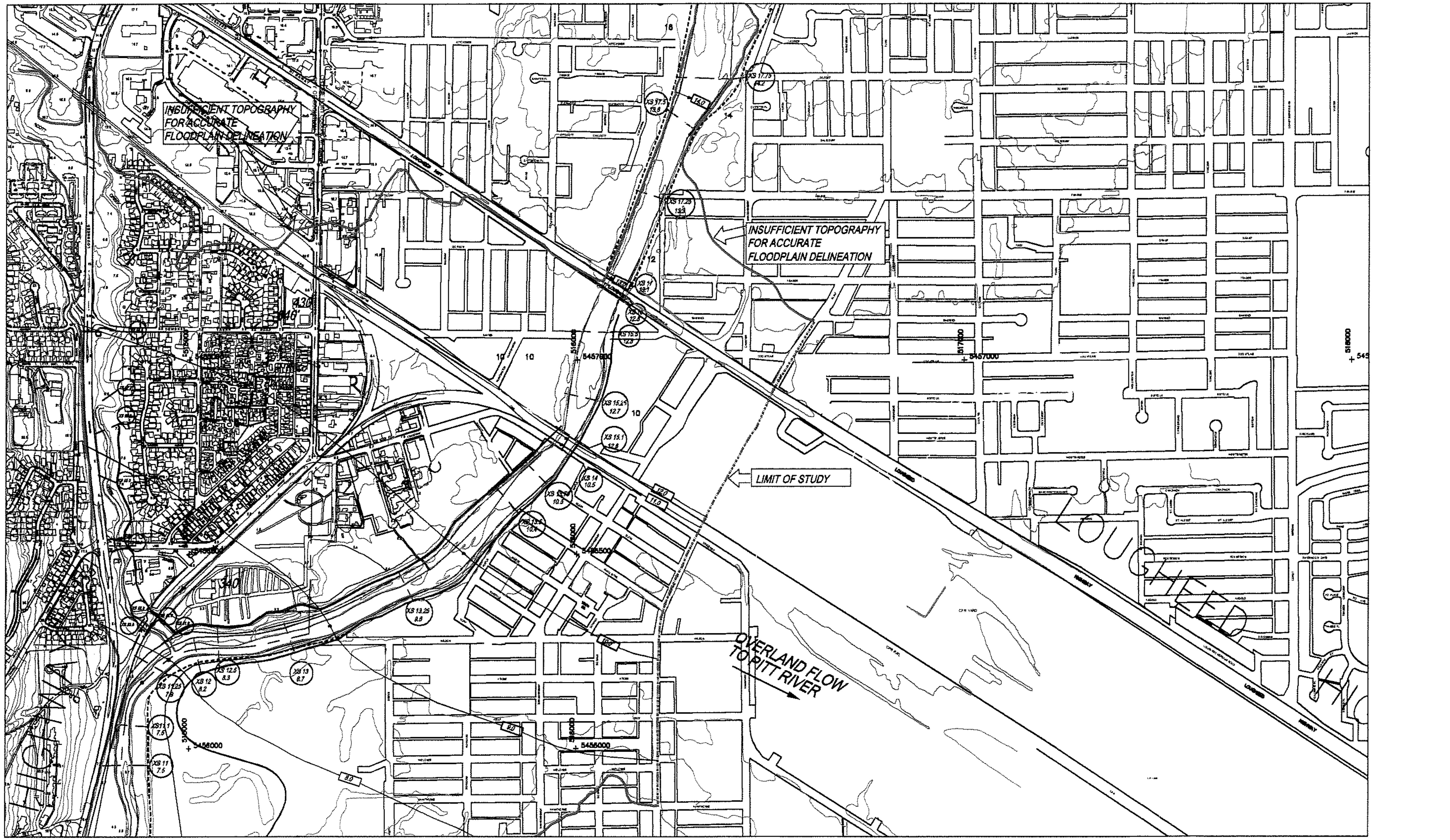
CITY OF PORT COQUITLAM

Distance in Metres

COQUITLAM RIVER FLOODPLAIN MAPS:
FRASER RIVER TO HOCKADAY STREET

Water Management Consultants Inc.
130-10691 Shellbridge Way,
Richmond, B.C. Canada V6X 2W8

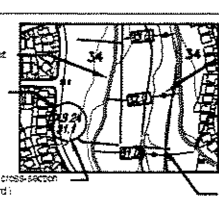
DRAWING NO.	7095 - 2
SHEET	2 OF 5
REVISION	



FLOODPLAIN DATA

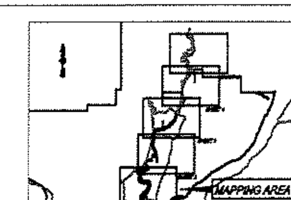
1. These maps are available from the Cities of Coquitlam and Port Coquitlam
2. Flooding may still occur outside of the designated floodplain areas. The Cities of Coquitlam and Port Coquitlam and the Province do not assume any liability by reason of the failure to designate areas on this map
3. There is a one in 200 chance that the Designated flood could be equaled or exceeded in any one year
4. The base mapping has been produced by combining 2m contour interval maps supplied by the Cities of Coquitlam and Port Coquitlam. Grid is to NAD 27 zone 10. Elevations refer to geodetic datum
5. The flood levels were computed using the HEC-RAS computer program version 3.1, and river cross sections surveyed in October-December 2000 and November 2001
6. The floodplain limits assume that dikes can fail
7. The floodplain limits and flood levels include a freeboard allowance
8. The floodplain limits are not established on the ground by legal survey. Building elevations should be based on field survey and established bench marks
9. The floodplain limits are not delineated for side streams and tributaries
10. The required setback of buildings from the natural boundaries, lakes and watercourses to allow for the passage of floodwaters and possible bank erosion is not shown. This information is available either through local municipalities or the Ministry of Water, Land and Air Protection

LEGEND



	Study limits
	Non-standard dike
	Standard dike
4	Contour line
4	Spot elevation

KEY MAP



REVISIONS

REV	DATE	BY	DESCRIPTION

PROJECT 7095

March 2004

MG

M.Garrett

D.Sellars

CITY OF PORT COQUITLAM

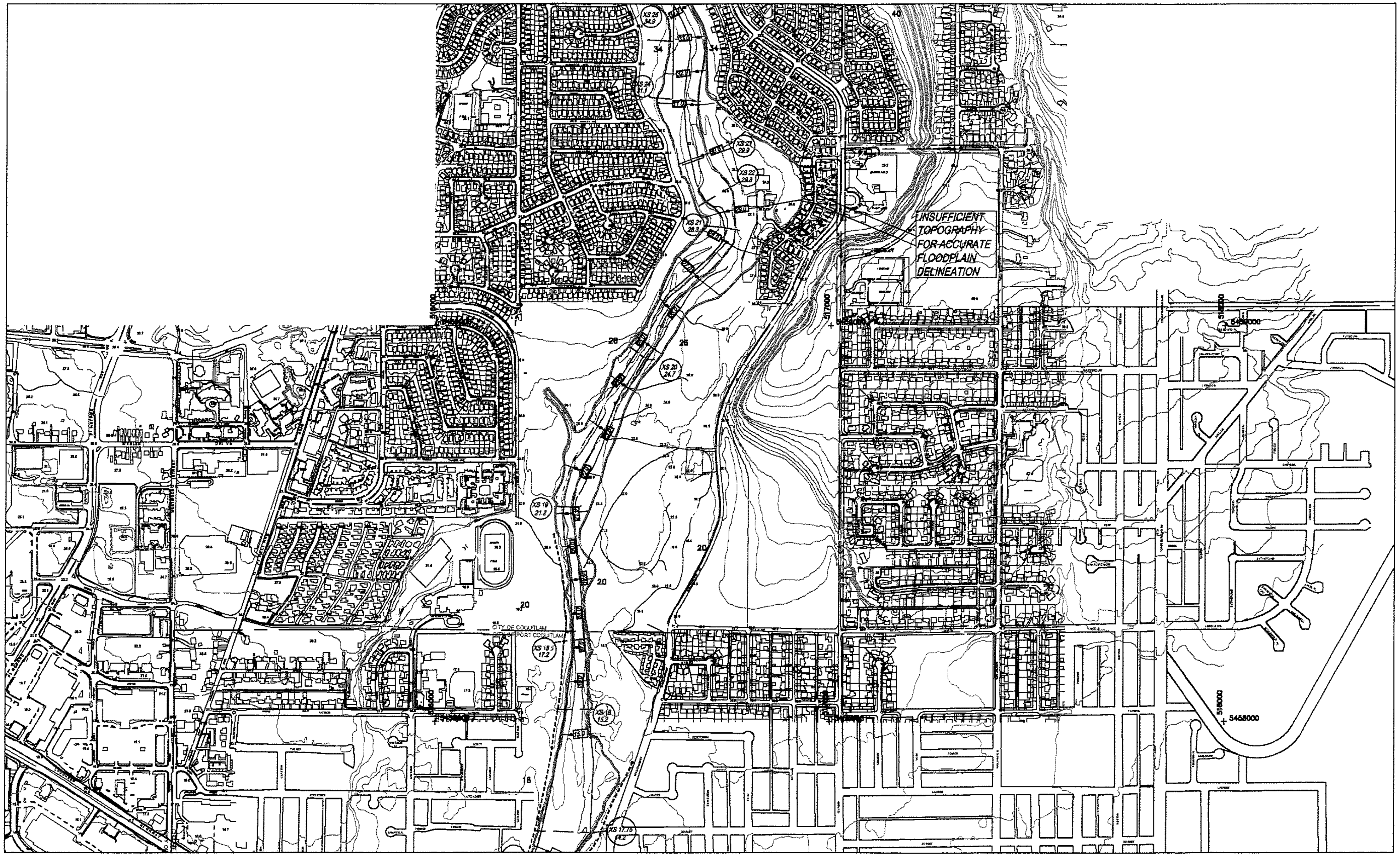
Distance in Metres

COQUITLAM RIVER FLOODPLAIN MAPS: FRASER RIVER TO HOCKADAY STREET

Water Management Consultants Inc.
130-10691 Shellbridge Way,
Richmond, B.C. Canada V6X 2W8

7095 - 2

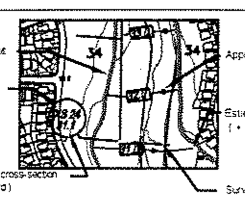
3 OF 5



FLOODPLAIN DATA

1. These maps are available from the Cities of Coquitlam and Port Coquitlam
2. Flooding may still occur outside of the designated floodplain areas. The Cities of Coquitlam and Port Coquitlam and the Province do not assume any liability by reason of the failure to designate areas on this map
3. There is a one in 200 chance that the Designated flood could be equaled or exceeded in any one year
4. The base mapping has been produced by combining 2 m contour interval maps supplied by the Cities of Coquitlam and Port Coquitlam. Grid is to NAD 27 zone 10. Elevations refer to geodetic datum
5. The flood levels were computed using the HEC-RAS computer program version 3.1, and river cross sections surveyed in October-December 2000 and November 2001
6. The floodplain limits assume that dikes can fail
7. The floodplain limits and flood levels include a freeboard allowance
8. The floodplain limits are not established on the ground by legal survey. Building elevations should be based on field survey and established bench marks
9. The floodplain limits are not delineated for side streams and tributaries
10. The required setback of buildings from the natural boundaries, lakes and watercourses to allow for the passage of floodwaters and possible bank erosion is not shown. This information is available either through local municipalities or the Ministry of Water, Land and Air Protection

LEGEND



Study limits

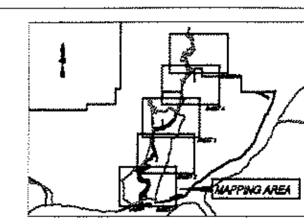
Non-standard dike

Standard dike

4 Contour line

4 Spot elevation

KEY MAP



REVISIONS

REV	DATE	BY	DESCRIPTION

PROJECT 7095

ISSUE OF MAPPING March 2004

DRAWN MG

ENGINEER M.Garrett

CHECKED

APPROVED D.Sellars

CITY OF PORT COQUITLAM

0 40 80 120 160 200
Distance in Metres

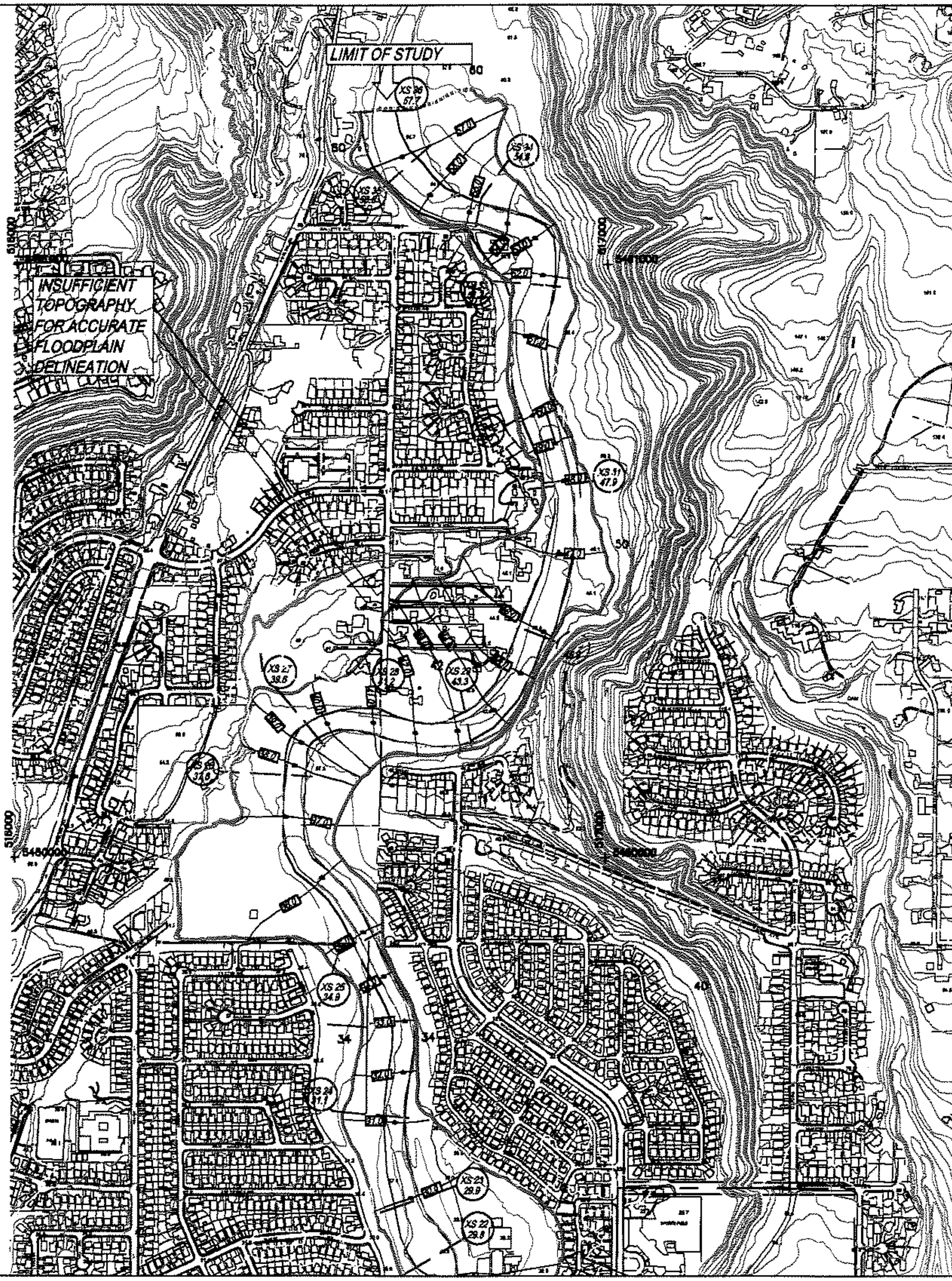
COQUITLAM RIVER FLOODPLAIN MAPS:
FRASER RIVER TO HOCKADAY STREET

Water Management Consultants Inc.
130-10691 Shellbridge Way,
Richmond, B.C. Canada V6X 2W8

DRAWING NO. 7095 - 2

SHEET 4 OF 5

REVISION



518000
+ 5461000

518000
+ 5460000

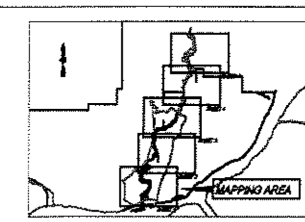
FLOODPLAIN DATA

1. These maps are available from the Cities of Coquitlam and Port Coquitlam
2. Flooding may still occur outside of the designated floodplain areas. The Cities of Coquitlam and Port Coquitlam and the Province do not assume any liability by reason of the failure to designate areas on this map
3. There is a one in 200 chance that the Designated flood could be equaled or exceeded in any one year
4. The base mapping has been produced by combining 2 m contour interval maps supplied by the Cities of Coquitlam and Port Coquitlam. Grid is to NAD 27 zone 10. Elevations refer to geodetic datum
5. The flood levels were computed using the HEC-RAS computer program version 3.1, and river cross sections surveyed in October-December 2000 and November 2001
6. The floodplain limits assume that dikes can fail
7. The floodplain limits and flood levels include a freeboard allowance
8. The floodplain limits are not established on the ground by legal survey. Building elevations should be based on field survey and established bench marks
9. The floodplain limits are not delineated for side streams and tributaries
10. The required setback of buildings from the natural boundaries, lakes and watercourses to allow for the passage of floodwaters and possible bank erosion is not shown. This information is available either through local municipalities or the Ministry of Water, Land and Air Protection

LEGEND

200-year Floodplain Limit
Cross-section number
200-year flood level at cross-section (+ 0.6 metre freeboard)
Approximate thalweg location
Estimated 200-year flood level (+ 0.6 metre freeboard)
Surveyed river cross-section
Study limits
Non-standard dike
Standard dike
Contour line
Spot elevation

KEY MAP



REVISIONS

REV	DATE	BY	DESCRIPTION

PROJECT: 7095
DATE OF MAPPING: March 2004
DRAWN: MG
ENGINEER: M.Garrett
CHECKED:
APPROVED: D.Sellars

CITY OF PORT COQUITLAM

0 40 80 120 160 200
Distance in Metres

COQUITLAM RIVER FLOODPLAIN MAPS:
FRASER RIVER TO HOCKADAY STREET

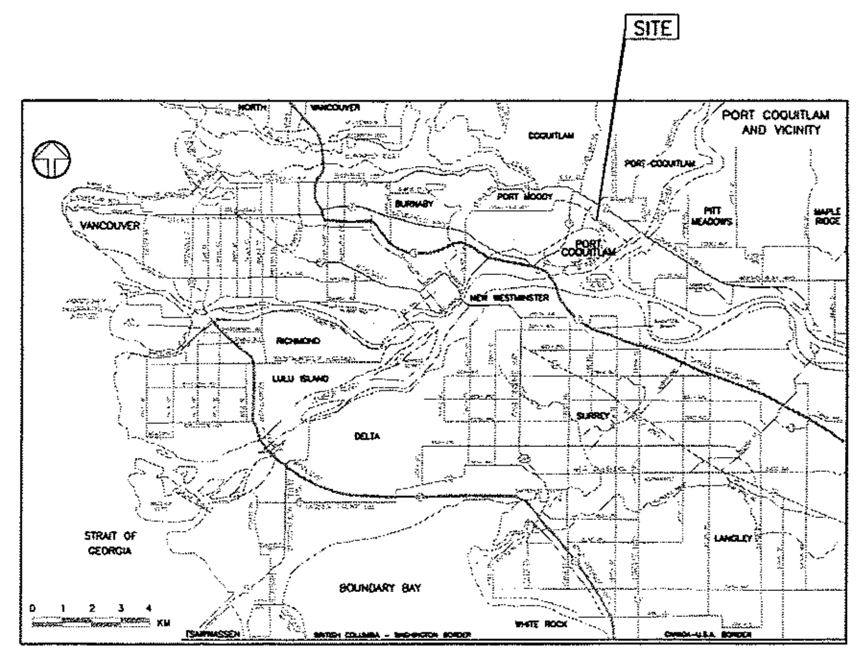
Water Management Consultants Inc.
130-10691 Shellbridge Way,
Richmond, B.C. Canada V6X 2W8

DRAWING ID: 7095 - 2
SHEET: 5 OF 5

CITY OF PORT COQUITLAM

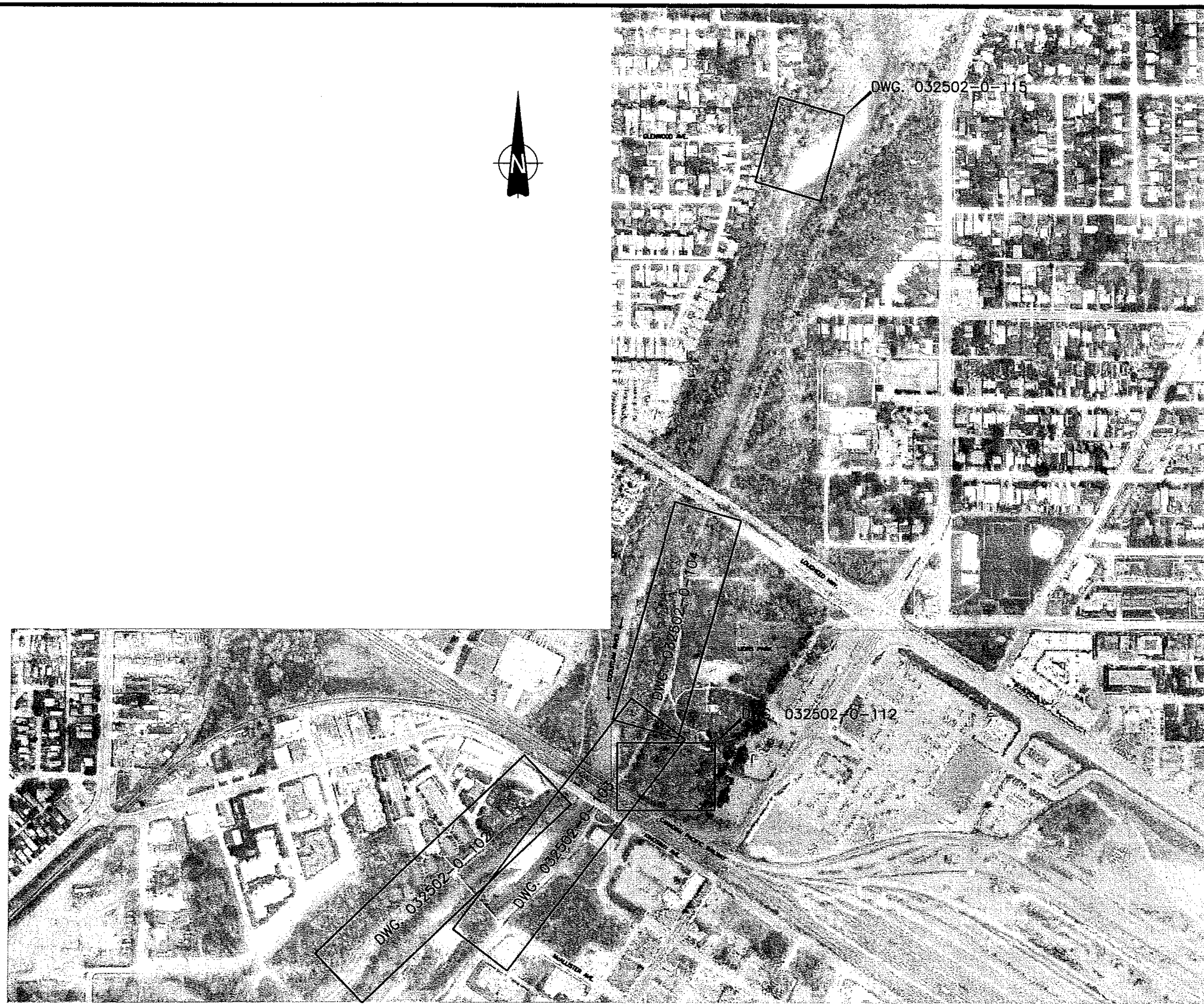
FLOOD HAZARD MITIGATION OPTIONS

COQUITLAM RIVER



LOCATION PLAN
N.T.S.

ASSOCIATED ENGINEERING



NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	C.O.	PRELIMINARY DESIGN
2	JAN. 2004	J.F.	C.O.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 ——— 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	C.O.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

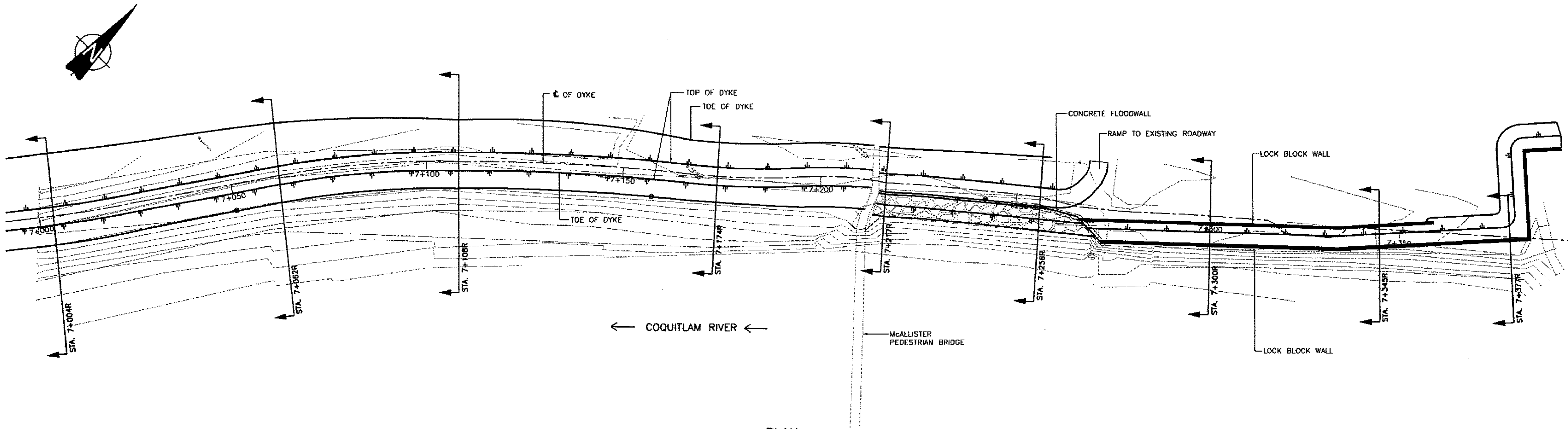
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
KEY PLAN

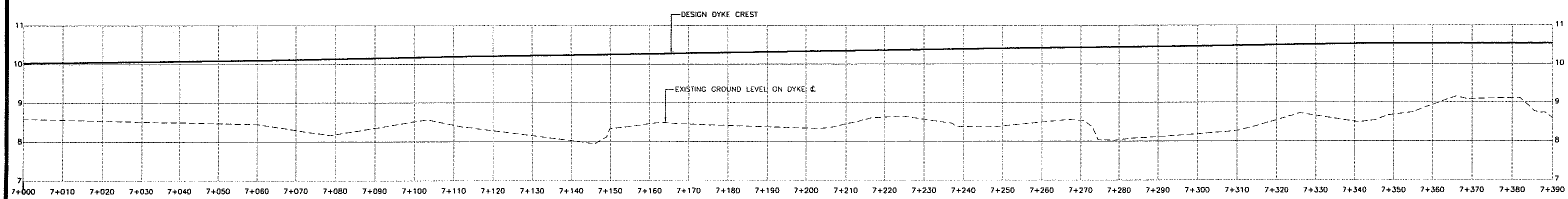
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-101	2	

This Drawing Is For The Use Of The Client ... Project Indicated
No Representations Of Any Kind Are Made ... Other Parties



PLAN
SCALE: 1:1000



PROFILE OF RIGHT BANK
SCALE: H: 1:1000 V: 1:100

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

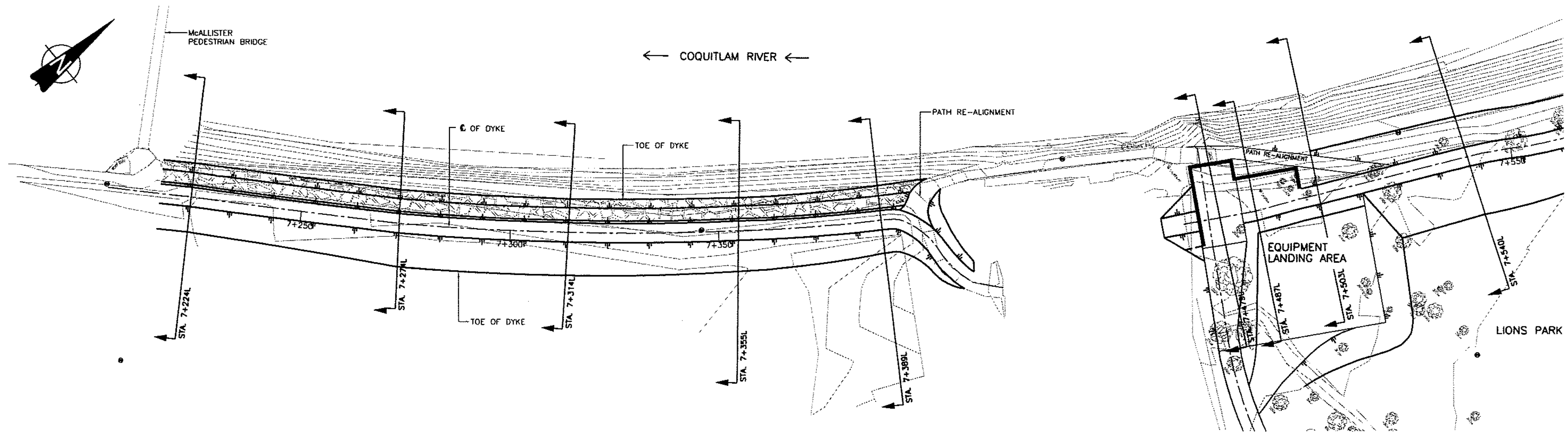
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
PLAN AND PROFILE

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

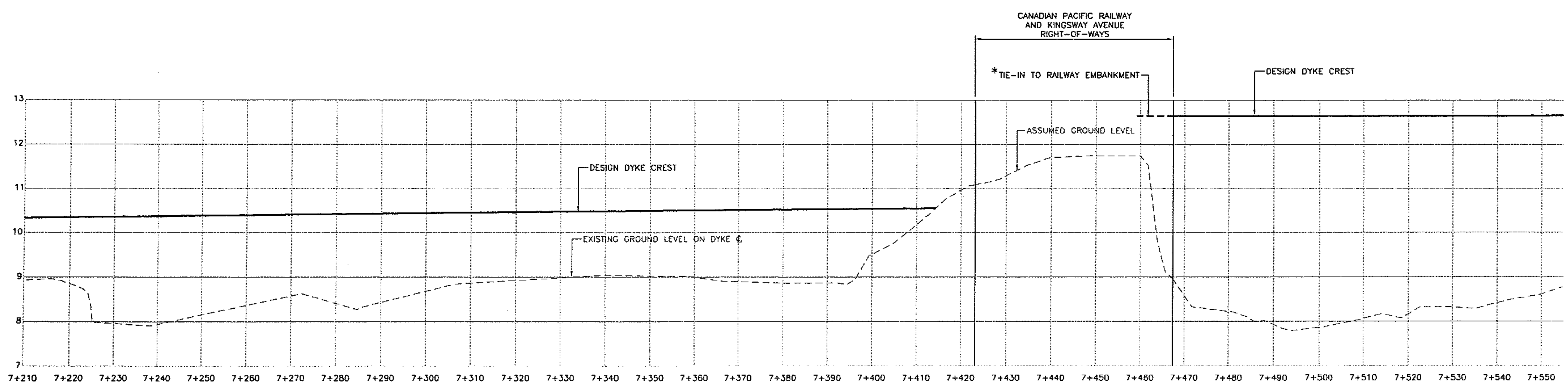
DRAWING NUMBER	REV. NO.	SHEET
032502-0-102	2	

This Drawing Is For The Use Of The Client Project Indicated
No Representations Of Any Kind Are Made To Other Parties



PLAN

SCALE: 1:1000



PROFILE OF LEFT BANK

SCALE: H: 1:1000 V: 1:100

*RAILWAY EMBANKMENT IS BELOW FCL AND WILL REQUIRE MODIFICATIONS TO PROVIDE CONTINUOUS DYKE.

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

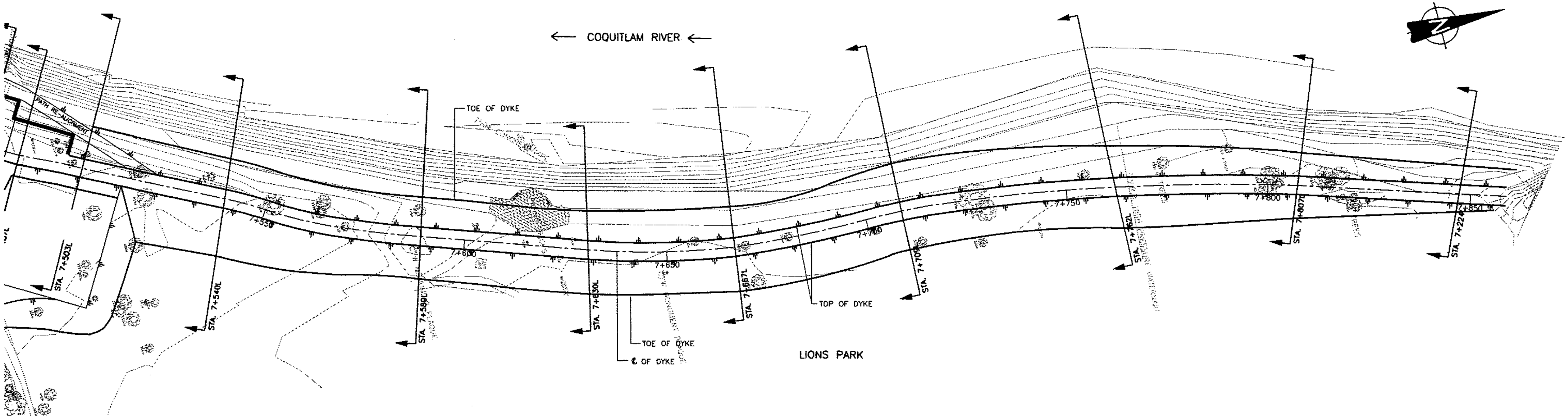
PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
PLAN AND PROFILE

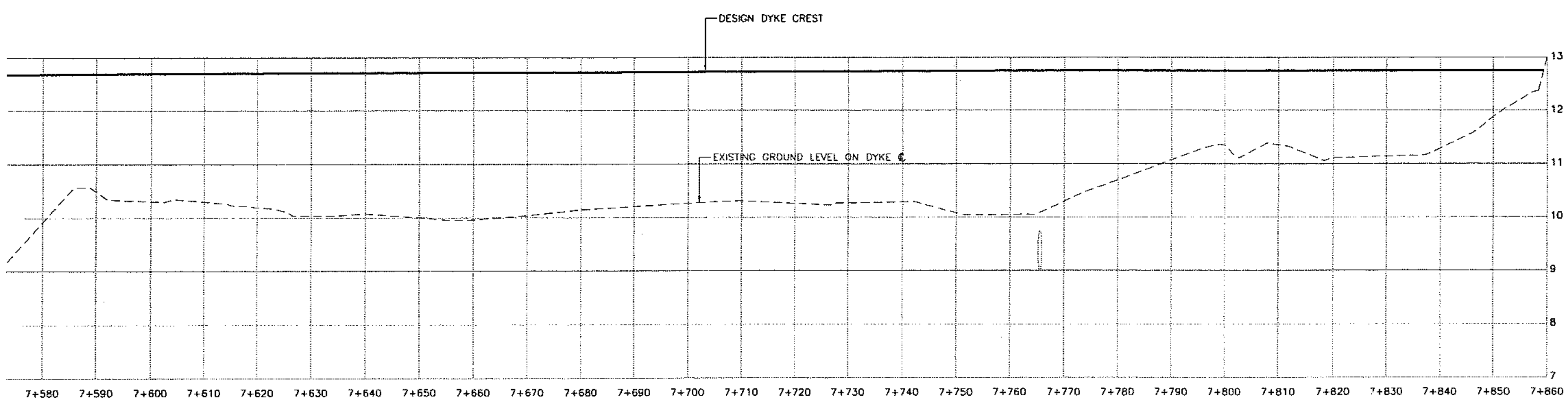
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-103	2	



PLAN

SCALE: 1:1000



PROFILE OF LEFT BANK

SCALE: H: 1:1000 V: 1:100

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

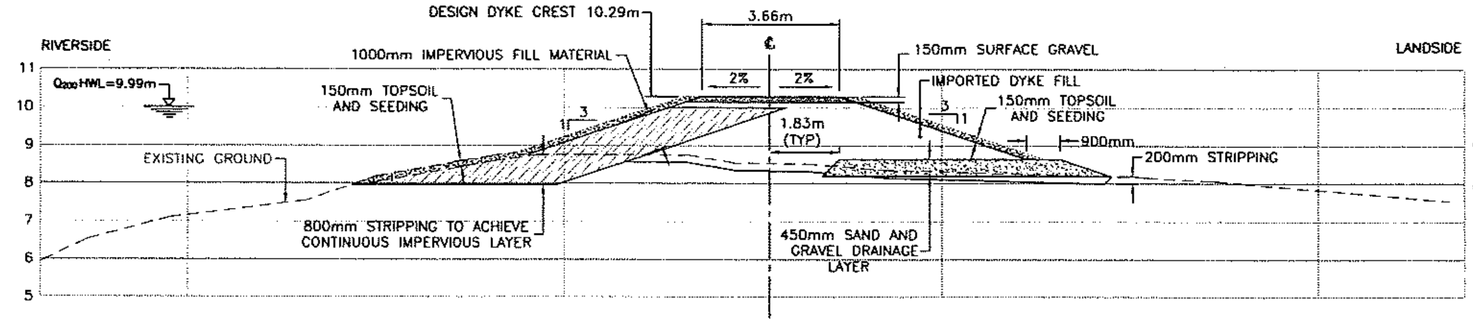
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
PLAN AND PROFILE

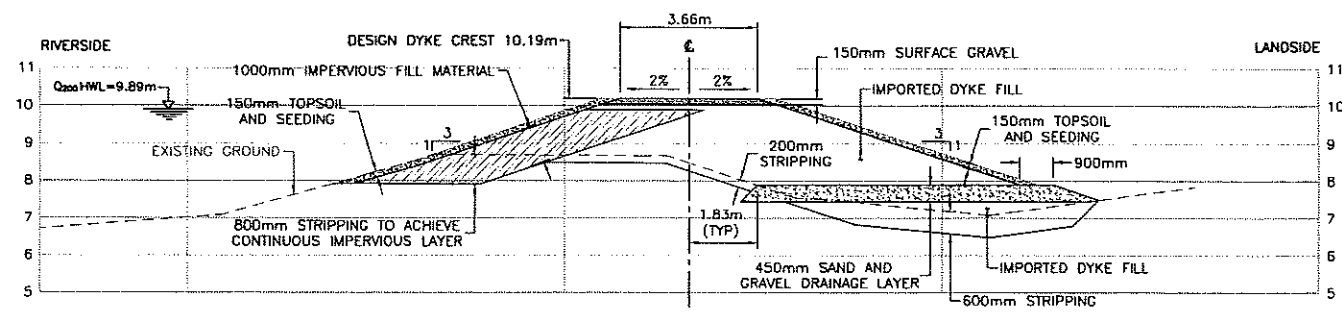
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-104	2	

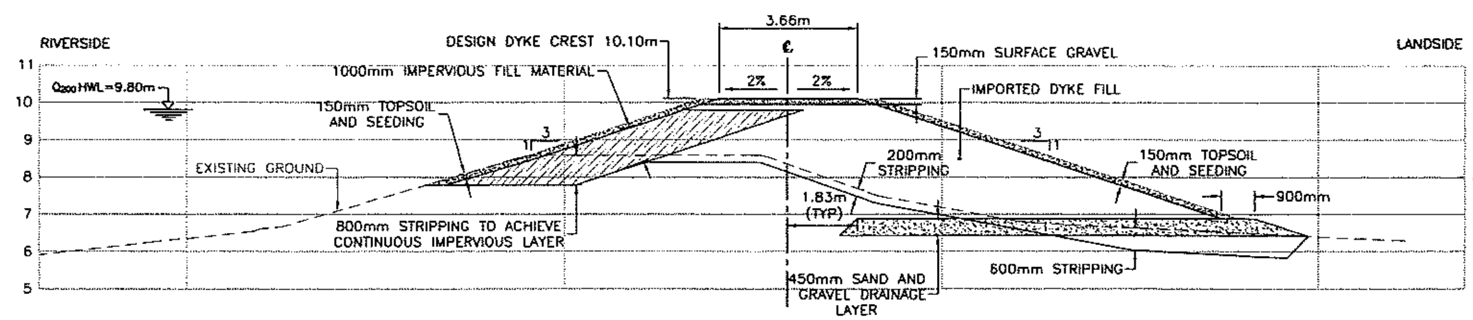
This Drawing Is For The Use Of The Client Project Indicated
No Representations Of Any Kind Are Made To Other Parties



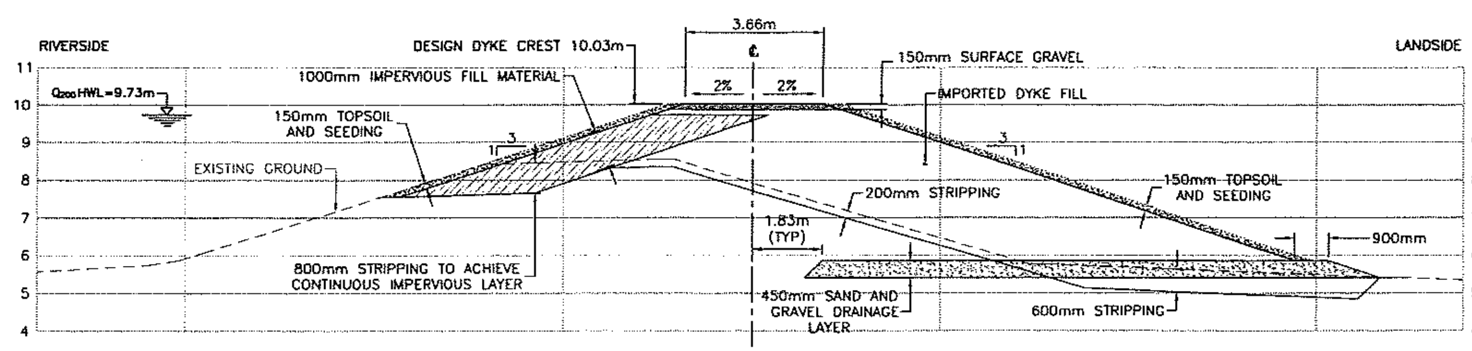
X–SECTION @ STA. 7+174R



X–SECTION @ STA. 7+108R



X–SECTION @ STA. 7+062R



X–SECTION @ STA. 7+004R

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

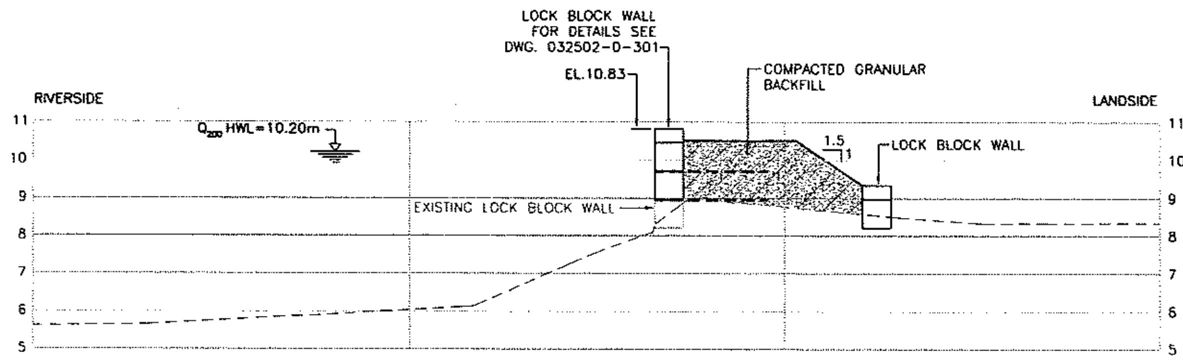
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
CROSS SECTIONS

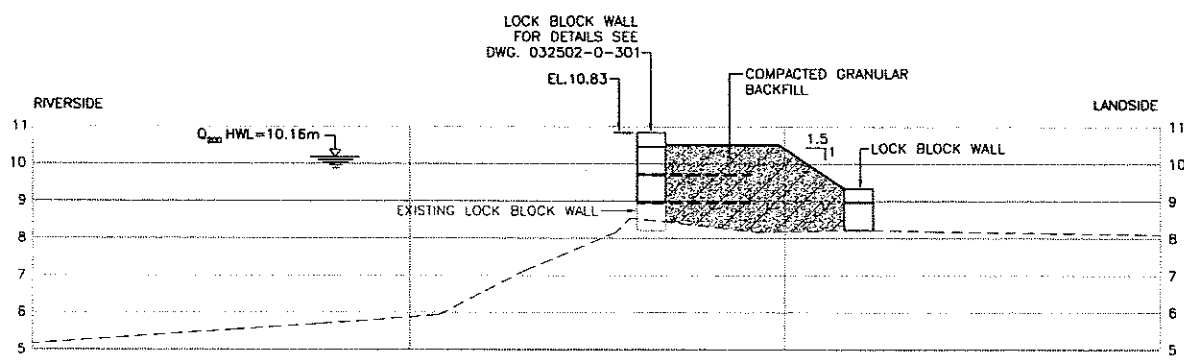
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-105	2	

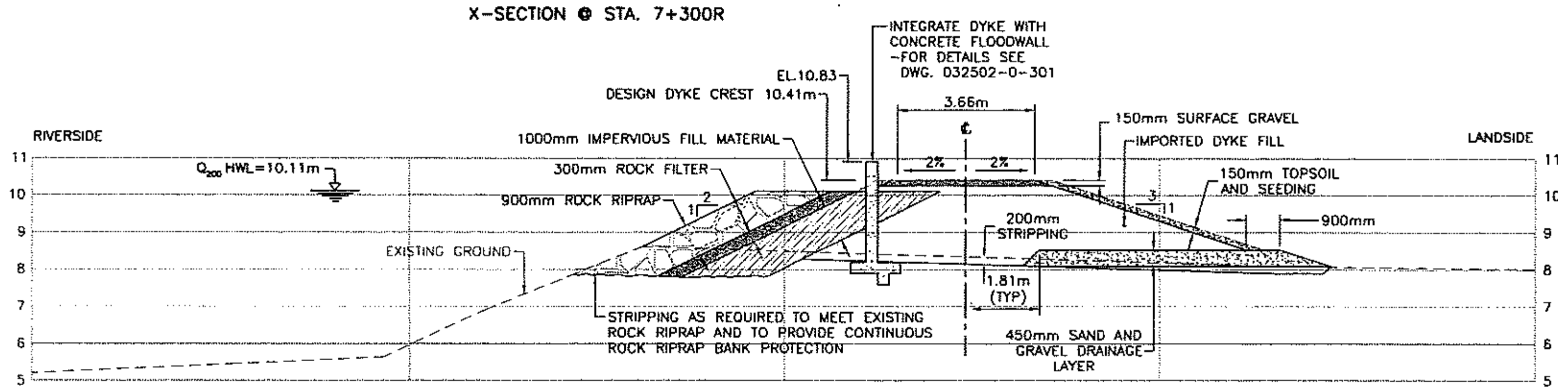
This Drawing Is For The Use Of The Client, Project Indicated
No Representations Of Any Kind Are Made To Other Parties



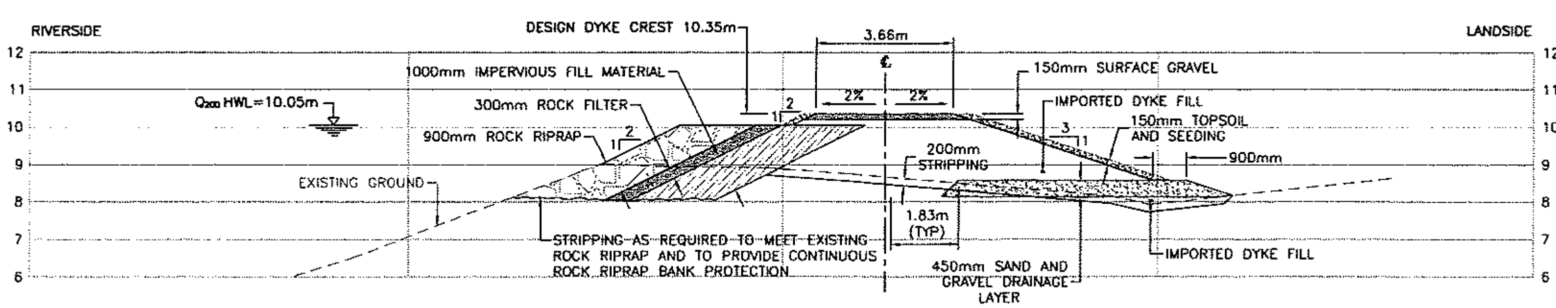
X-SECTION @ STA. 7+345R



X-SECTION @ STA. 7+300R



X-SECTION @ STA. 7+256R



X-SECTION @ STA. 7+217R

ROCK RIPRAP GRADATION PERCENTAGE LARGER THAN GIVEN ROCK MASS (kg)		
85%	50%	15%
10	100	300

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.O.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.O.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

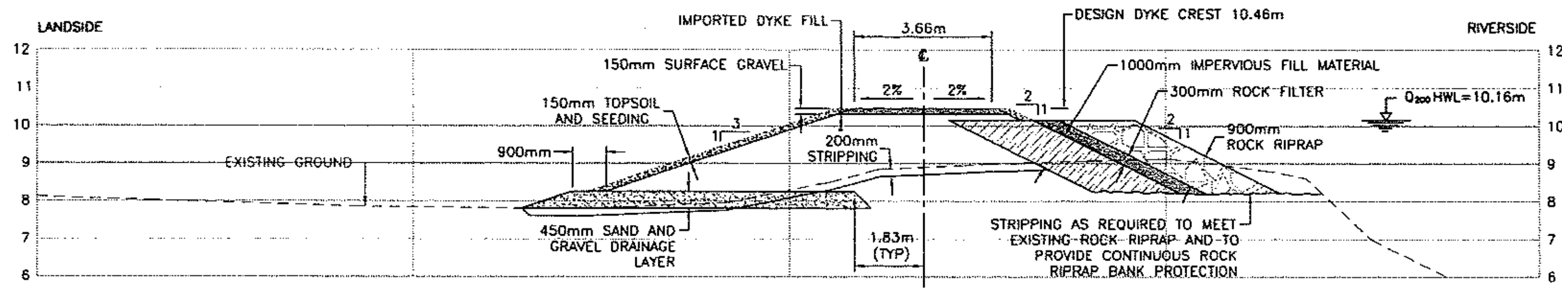
PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.O.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

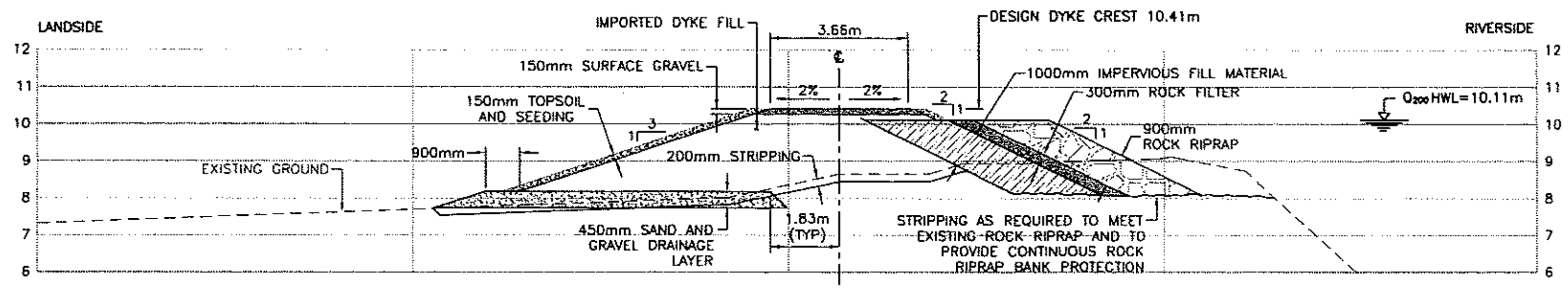
FLOOD MITIGATION WORKS
CROSS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

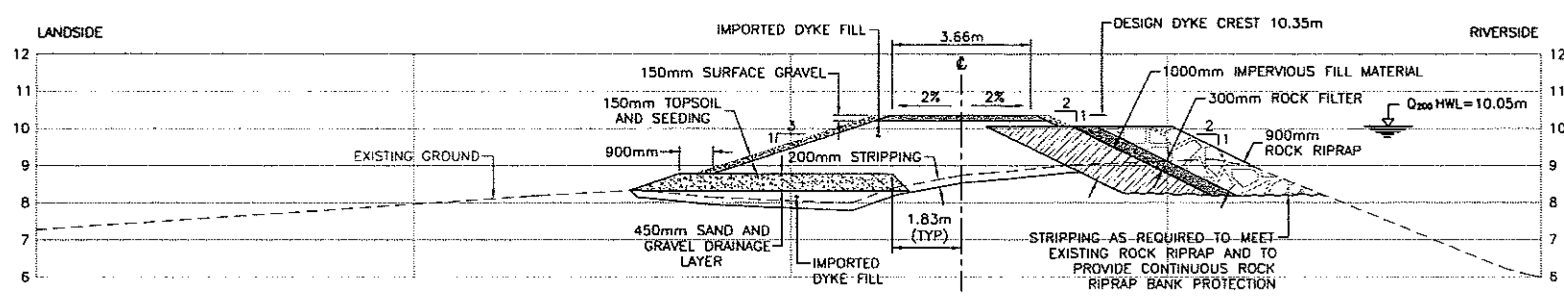
DRAWING NUMBER	REV. NO.	SHEET
032502-0-106	2	



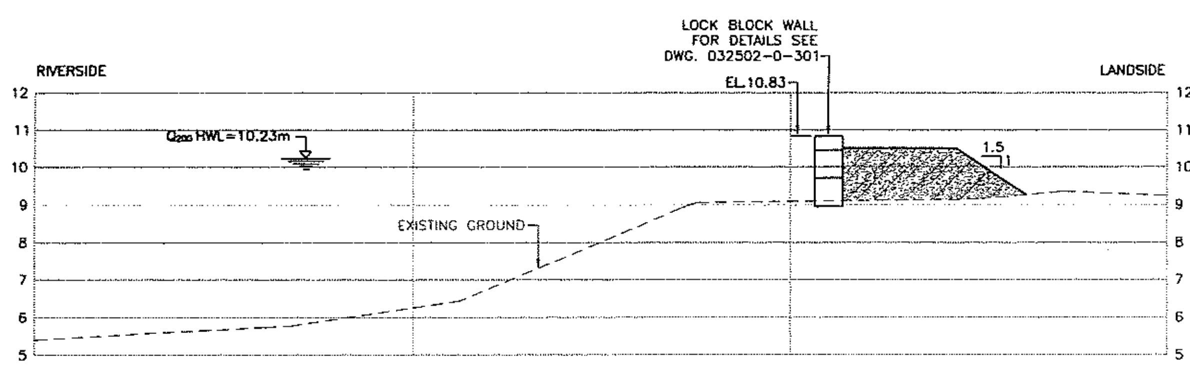
X-SECTION @ STA. 7+314L



X-SECTION @ STA. 7+274L



X-SECTION @ STA. 7+224L



X-SECTION @ STA. 7+377R

ROCK RIPRAP GRADATION PERCENTAGE LARGER THAN GIVEN ROCK MASS (kg)		
85%	50%	15%
10	100	300

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.G.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.G.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.G.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

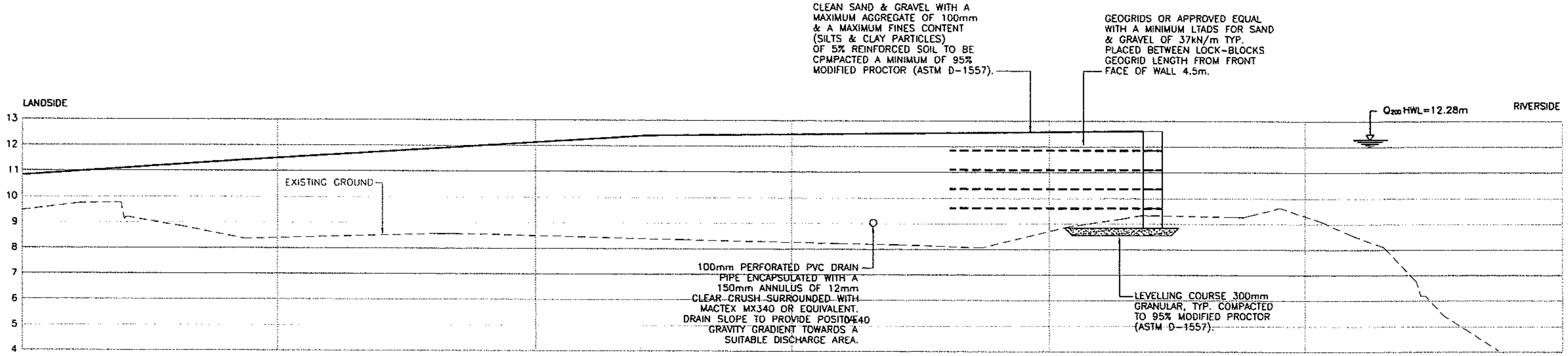
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
CROSS SECTIONS

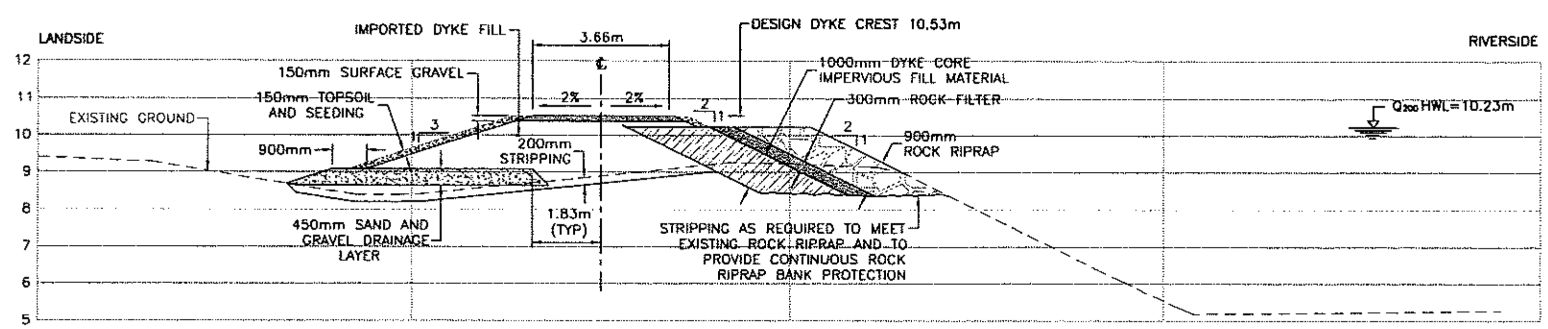
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-107	2	

This Drawing Is For The Use Of The Client, Project Indicated
No Representations Of Any Kind Are Made To Other Parties

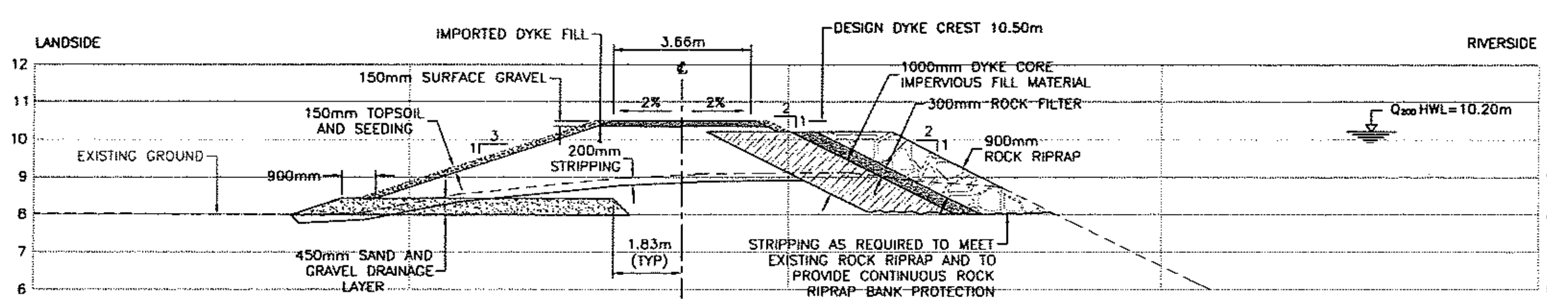


X-SECTION @ STA. 7+479L



X-SECTION @ STA. 7+389L

ROCK RIPRAP GRADATION PERCENTAGE LARGER THAN GIVEN ROCK MASS (kg)		
85%	50%	15%
10	100	300



X-SECTION @ STA. 7+355L

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

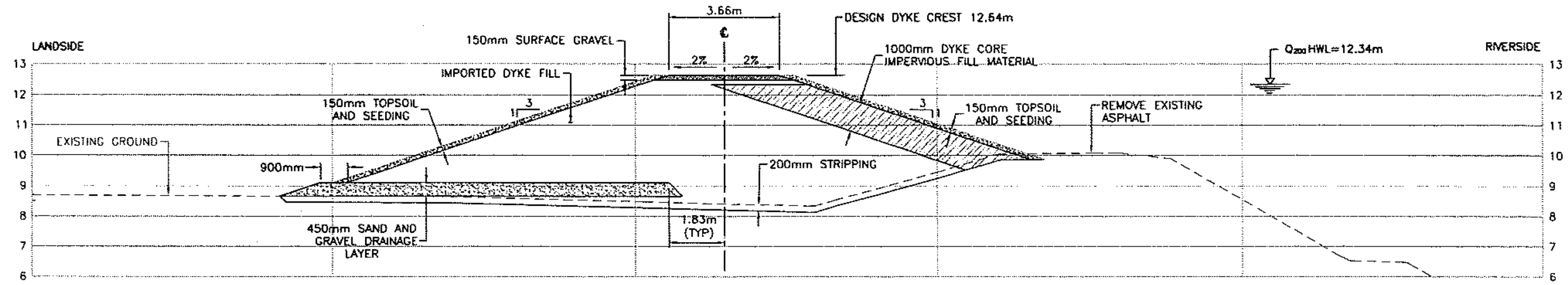
PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

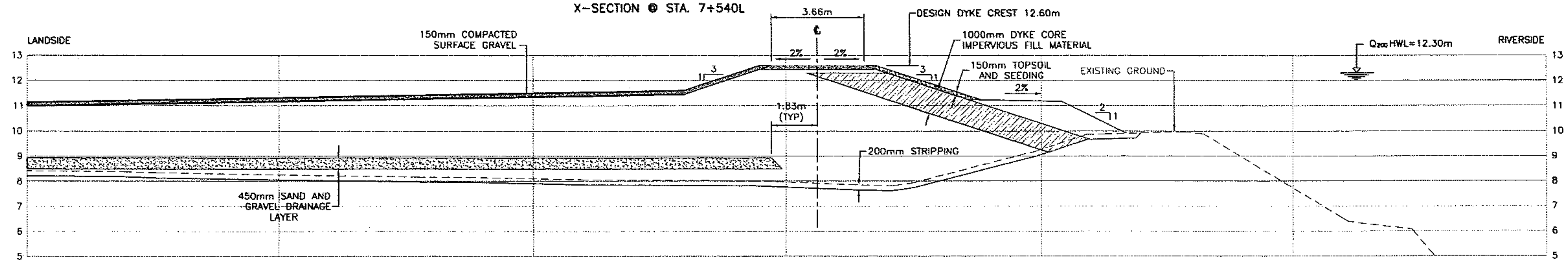
FLOOD MITIGATION WORKS
CROSS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

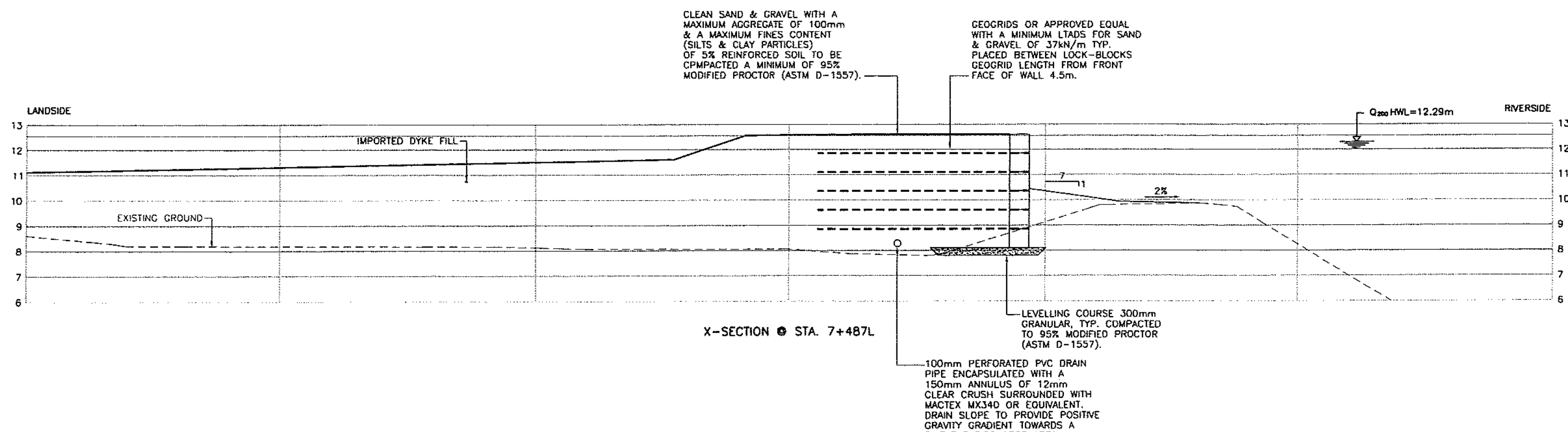
DRAWING NUMBER	REV. NO.	SHEET
032502-0-108	2	



X-SECTION @ STA. 7+540L



X-SECTION @ STA. 7+503L



X-SECTION @ STA. 7+487L

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.O.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.O.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.O.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

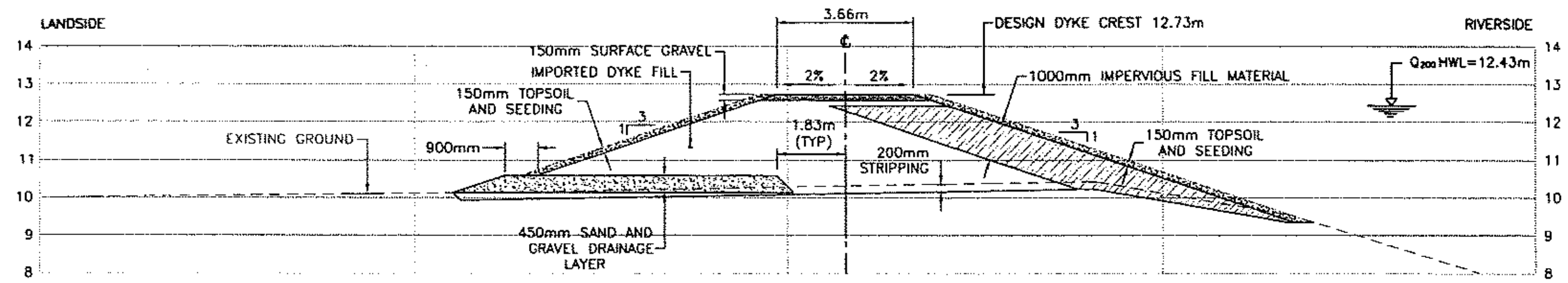
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
CROSS SECTIONS

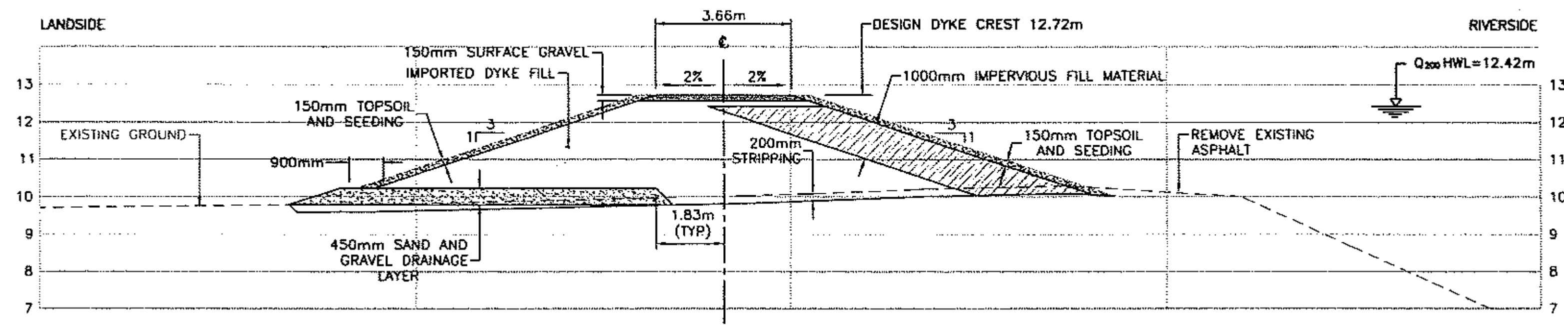
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-109	2	

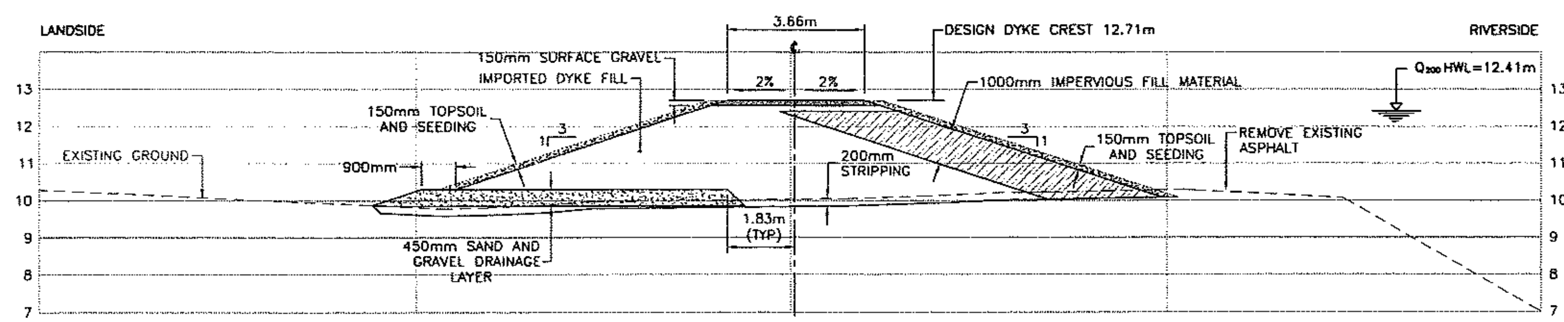
Date: 2004/2/24



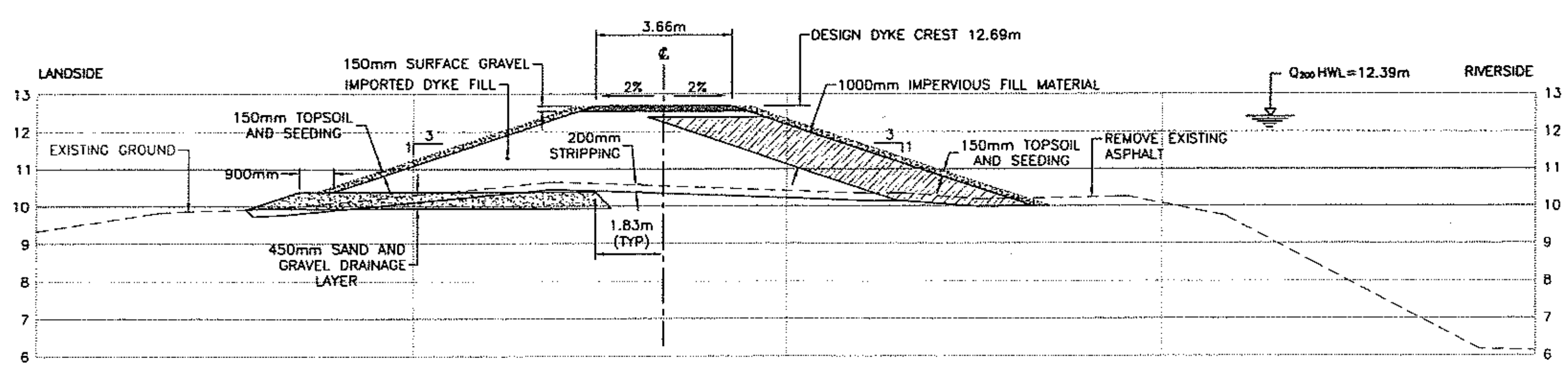
X-SECTION @ STA. 7+709L



X-SECTION @ STA. 7+667L



X-SECTION @ STA. 7+630L



X-SECTION @ STA. 7+589L

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

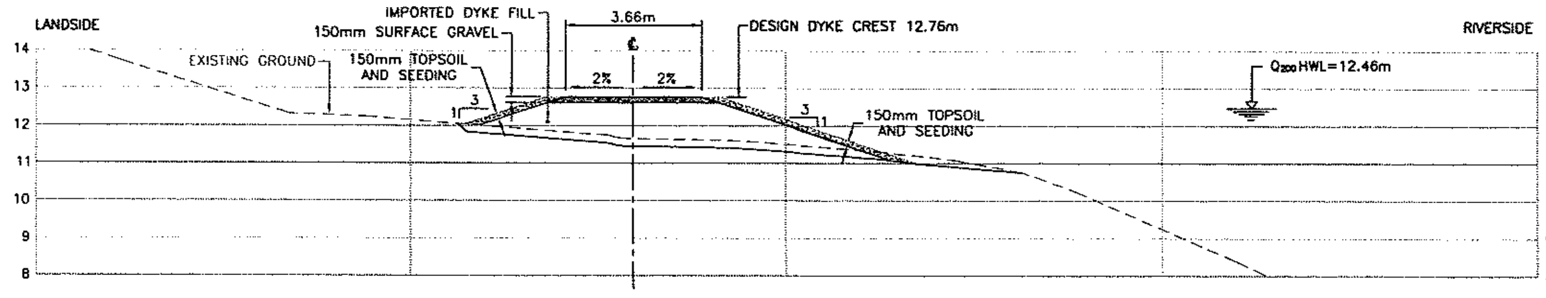
PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

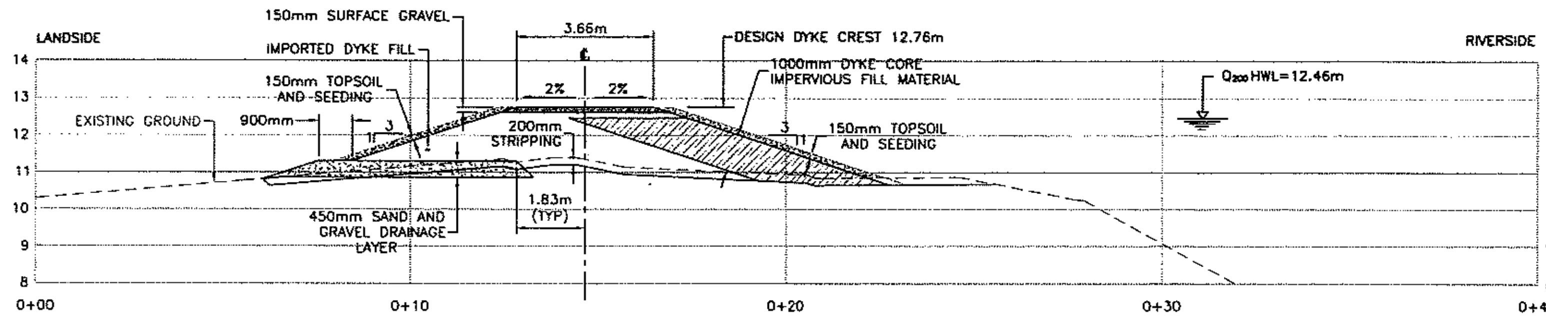
FLOOD MITIGATION WORKS
CROSS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

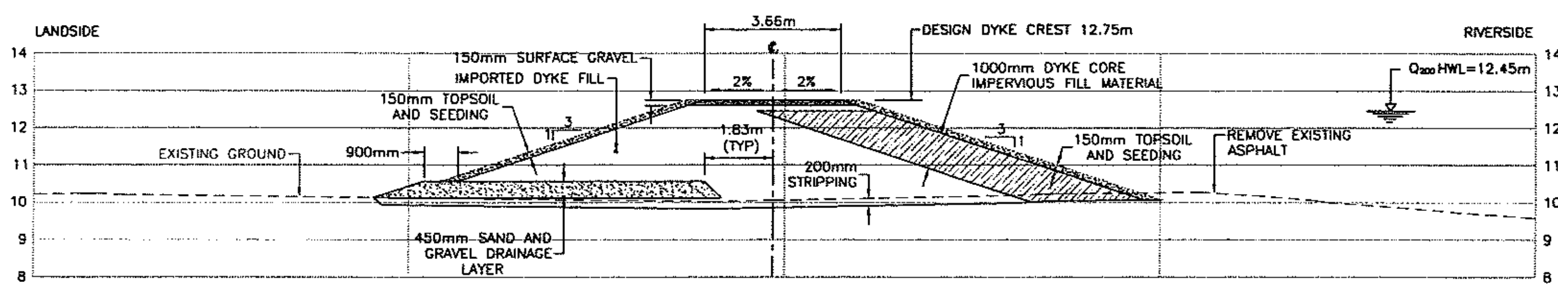
DRAWING NUMBER	REV. NO.	SHEET
032502-0-110	2	



X-SECTION @ STA. 7+847L



X-SECTION @ STA. 7+807L



X-SECTION @ STA. 7+762L

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

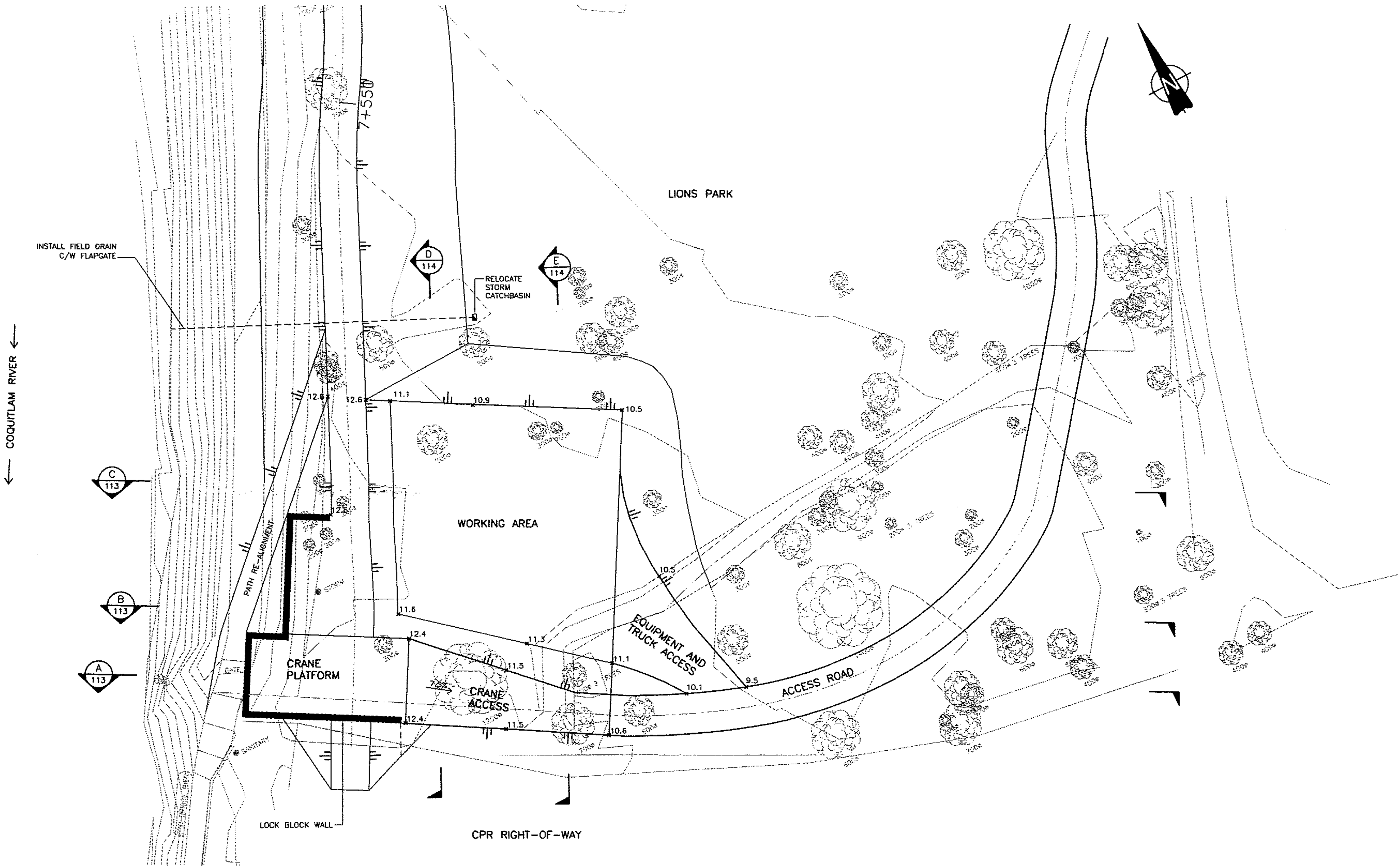
PROJECT No.	032502		
SCALE	1:200		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS
CROSS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-111	2	



NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

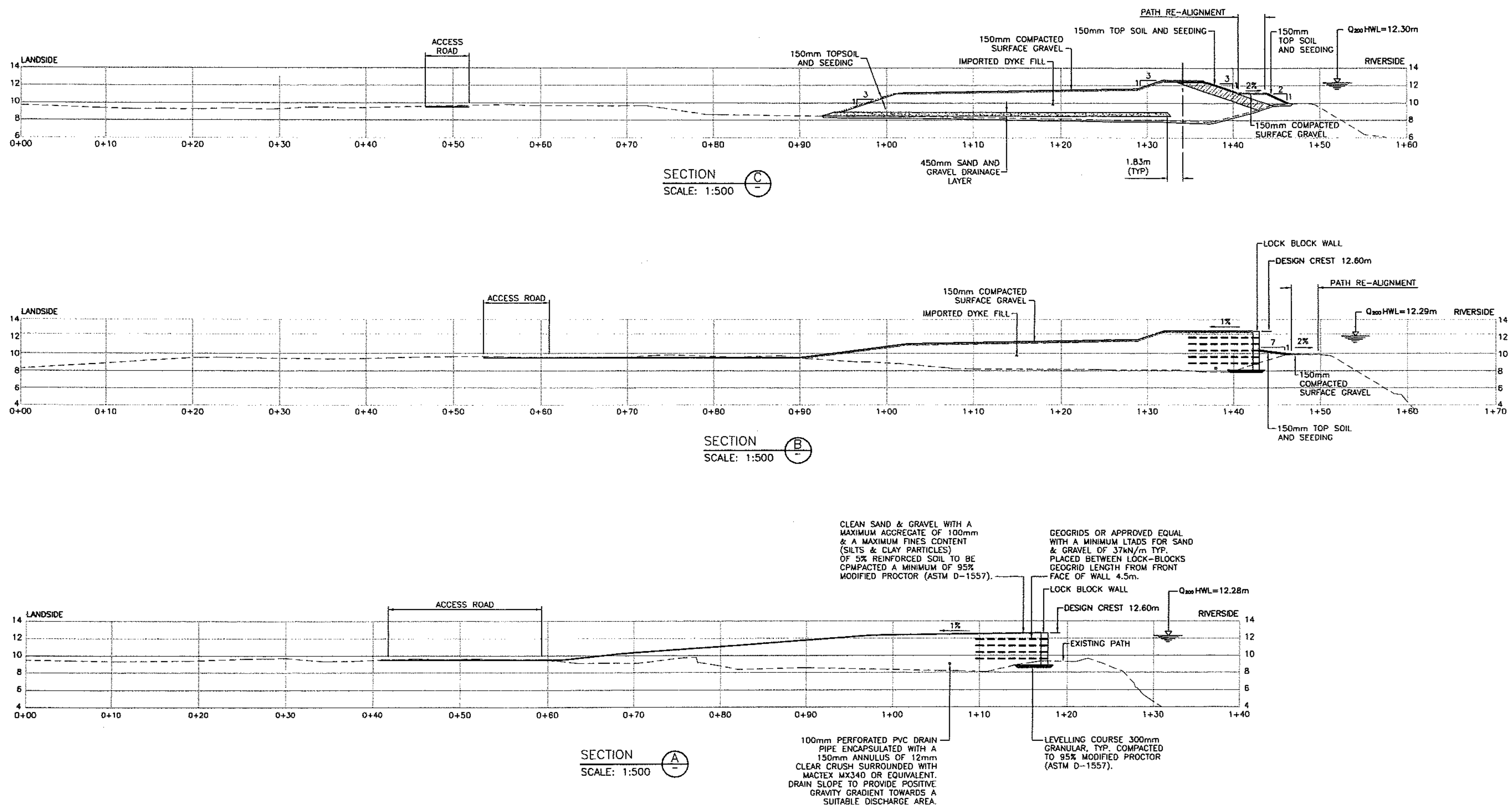
PROJECT No.	032502		
SCALE	1:500		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

EQUIPMENT LANDING AREA
FOR DEBRIS CONTROL

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-112	2	



NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

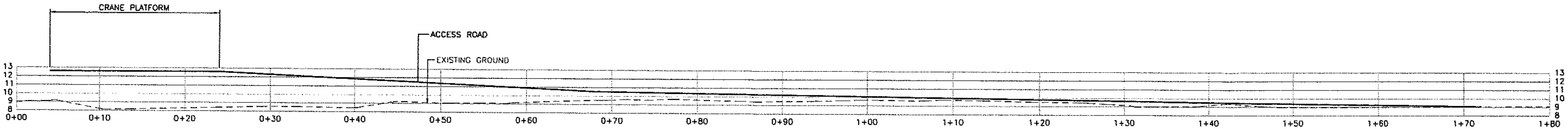
PROJECT No.	032502		
SCALE	1:500		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

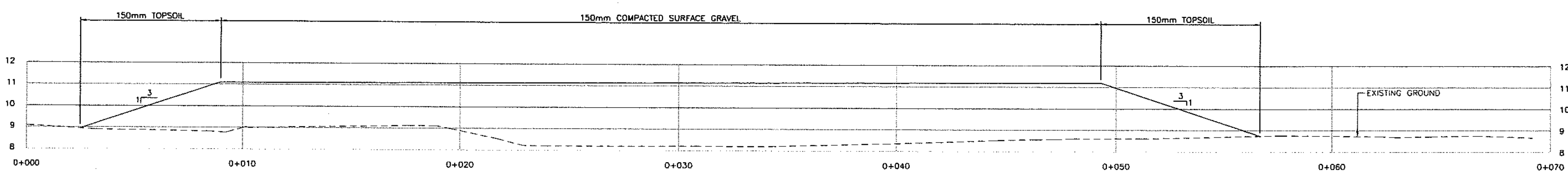
EQUIPMENT LANDING AREA
FOR DEBRIS CONTROL
CROSS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

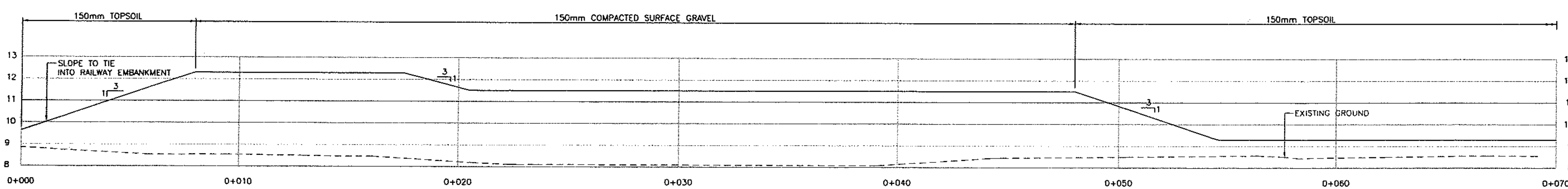
DRAWING NUMBER	REV. NO.	SHEET
032502-0-113	2	



ACCESS ROAD ℄ PROFILE
SCALE: 1:500



SECTION E
SCALE: 1:200



SECTION D
SCALE: 1:200

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	PRELIMINARY DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

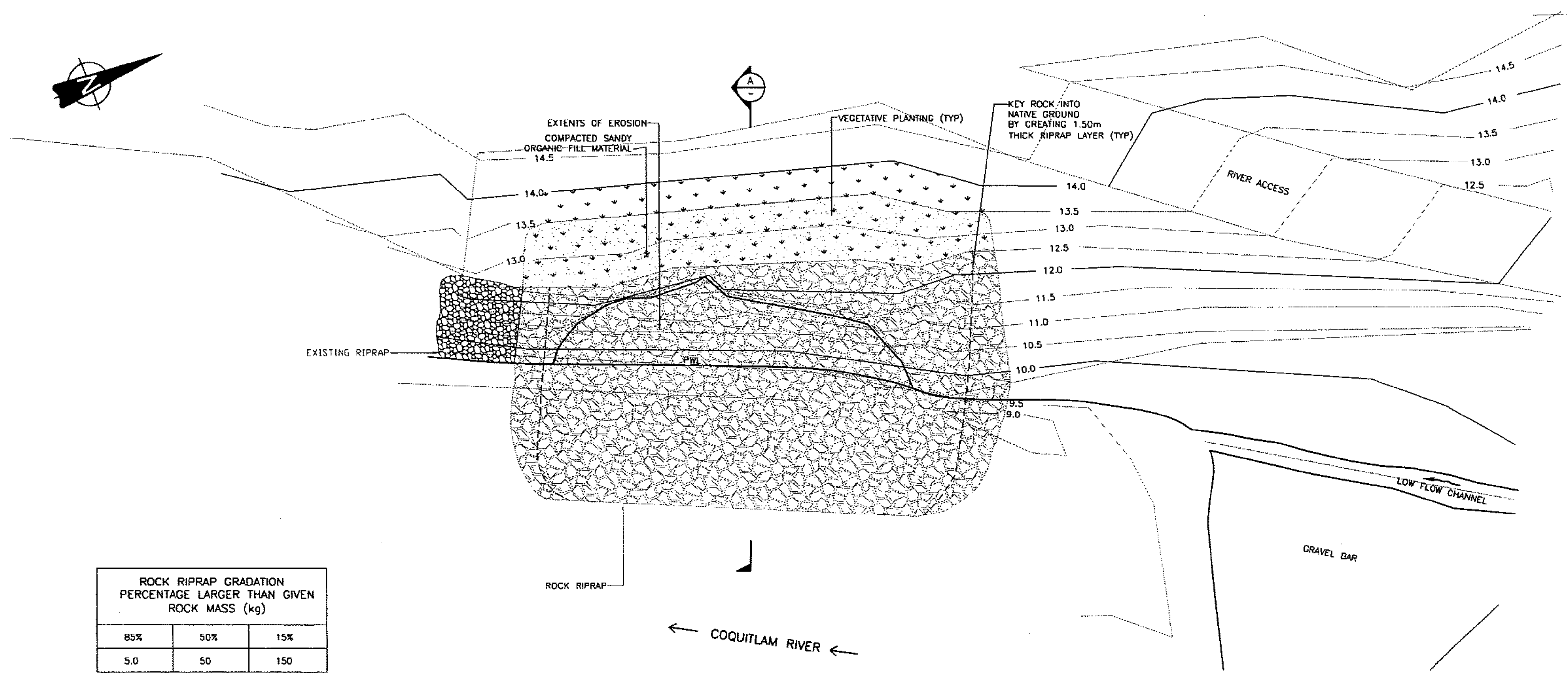
PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	G.Q.		JAN. 2004
DESIGNED	J.F.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

EQUIPMENT LANDING AREA
FOR DEBRIS CONTROL
PROFILE AND CROSS SECTIONS

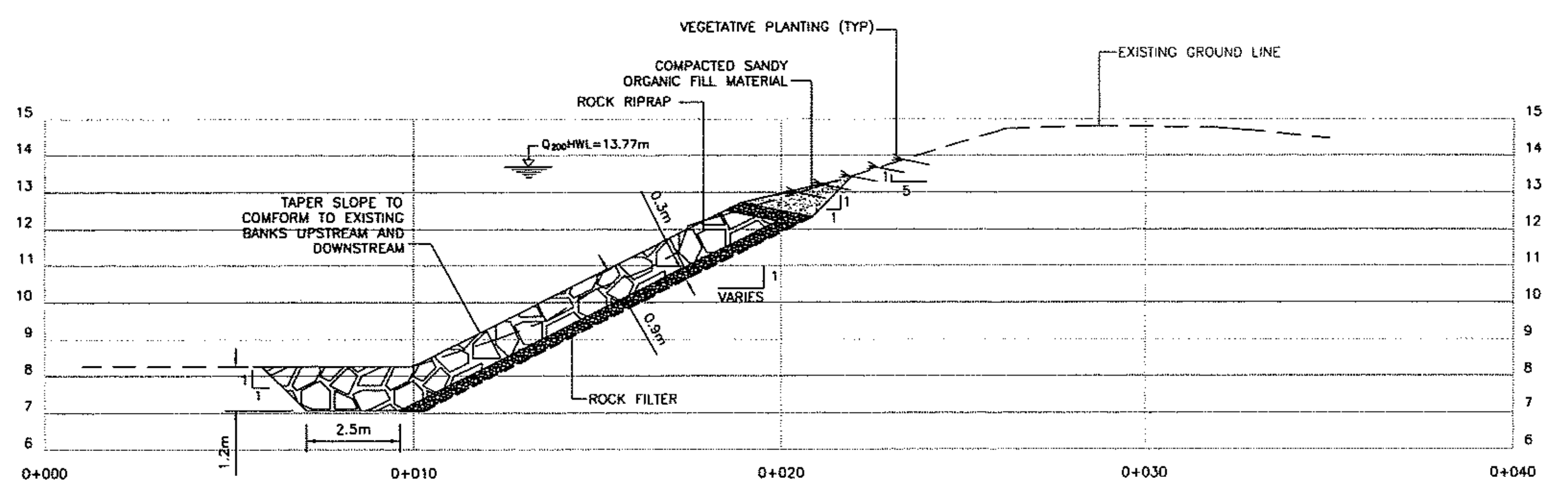
FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-114	2	



ROCK RIPRAP GRADATION PERCENTAGE LARGER THAN GIVEN ROCK MASS (kg)		
85%	50%	15%
5.0	50	150

Equiv. Spherical dia. 250mm 320 500



SECTION A
1:200

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	J.F.	G.Q.	DETAILED DESIGN
1	JAN. 2004	J.F.	G.Q.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

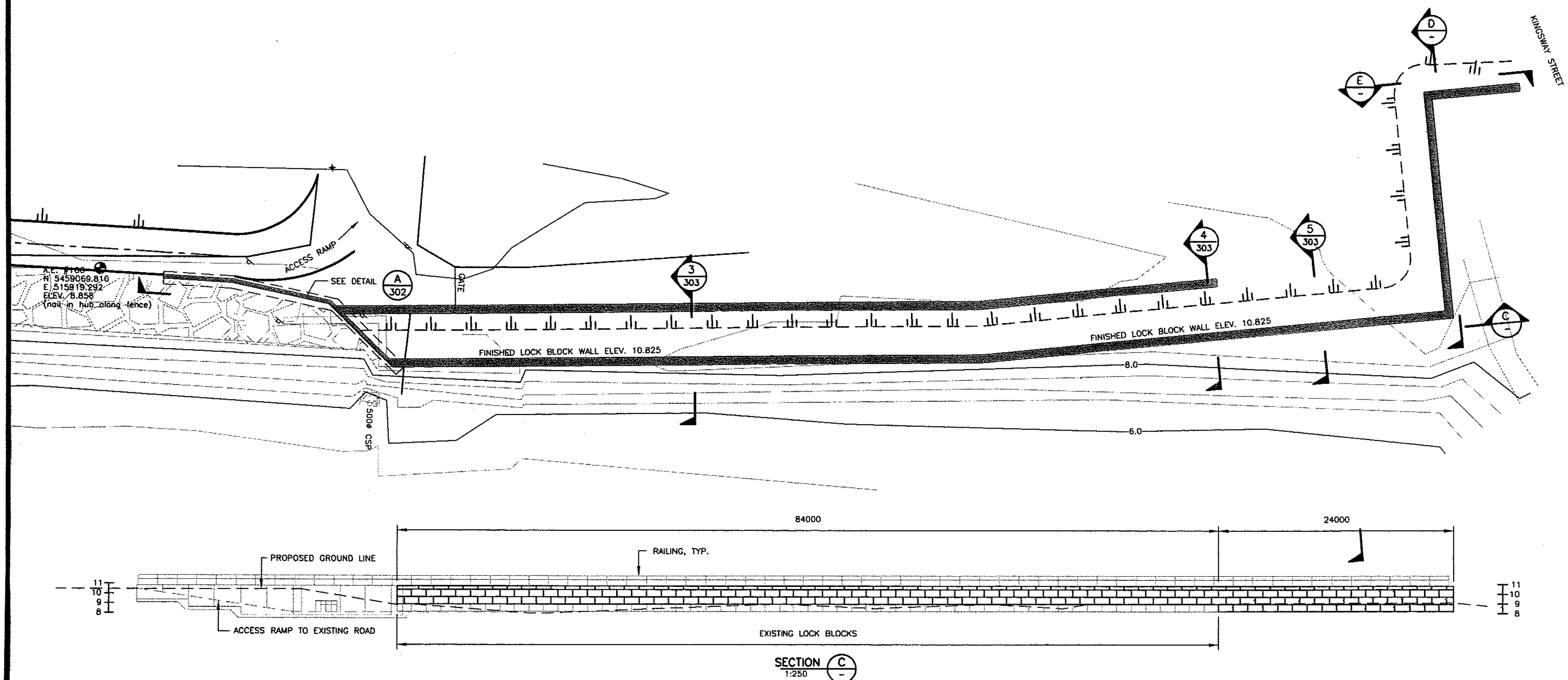
PROJECT No.	032502		
SCALE	1:250		
DRAWN	G.Q.		DEC. 2003
DESIGNED	J.F.		DEC. 2003
CHECKED			
APPROVED			
DATE		INITIAL	

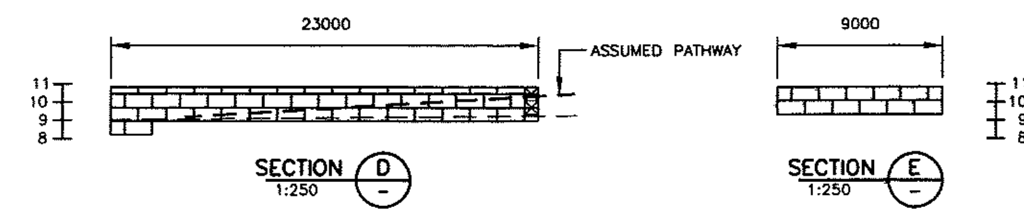
CITY OF PORT COQUITLAM

RIGHT BANK EROSION AREA
500m UPSTREAM OF LOUGHEED HWY.
PLAN VIEW

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-115	2	





GEOTECHNICAL ENGINEERING IS NOT INCLUDED WITHIN THE SCOPE OF SERVICES BEING PROVIDED BY ASSOCIATED ENGINEERING (B.C.) LTD. THEREFORE THIS DESIGN HAS BEEN PREPARED WITHOUT THE BENEFIT OF GEOTECHNICAL FIELD INVESTIGATION OR ADVICE. GROUND CONDITIONS MAY VARY. THE FOUNDATION REQUIREMENTS MAY NEED TO BE MODIFIED TO ACCOMMODATE ACTUAL SITE CONDITIONS ENCOUNTERED DURING CONSTRUCTION. ASSOCIATED ENGINEERING (B.C.) LTD. ACCEPTS NO LIABILITY OR RESPONSIBILITY FOR DELAY OR ADDITIONAL COSTS WHICH MAY RESULT IF GROUND CONDITIONS VARY FROM THOSE ASSUMED. PLEASE CONTACT ASSOCIATED ENGINEERING (B.C.) LTD. IF GEOTECHNICAL DATA AND RECOMMENDATIONS ARE AVAILABLE FOR THIS SITE OR IF THE OWNER WISHES TO EXPAND THE ENGINEERING SERVICES TO INCORPORATE THIS EXTRA WORK.

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	R.K.	S.P.	PRELIMINARY DESIGN
1	JAN. 2004	R.K.	S.P.	DRAFT

REVISIONS

VERIFY SCALES

BAR IS 20mm ON ORIGINAL DRAWING

0 — 20mm

IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

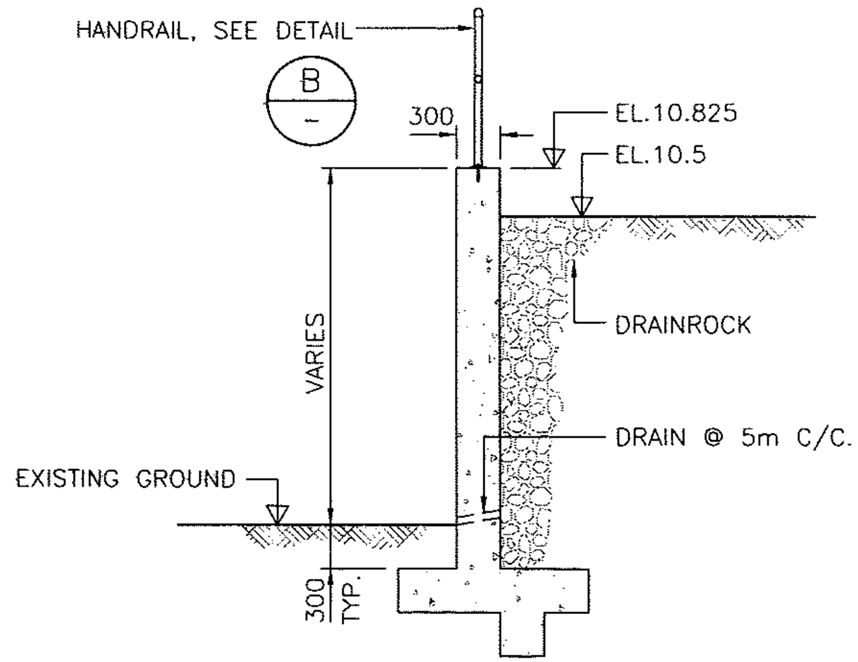
PROJECT No.	032502		
SCALE	1:250		
DRAWN	S.P.		JAN. 2004
DESIGNED	R.K.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

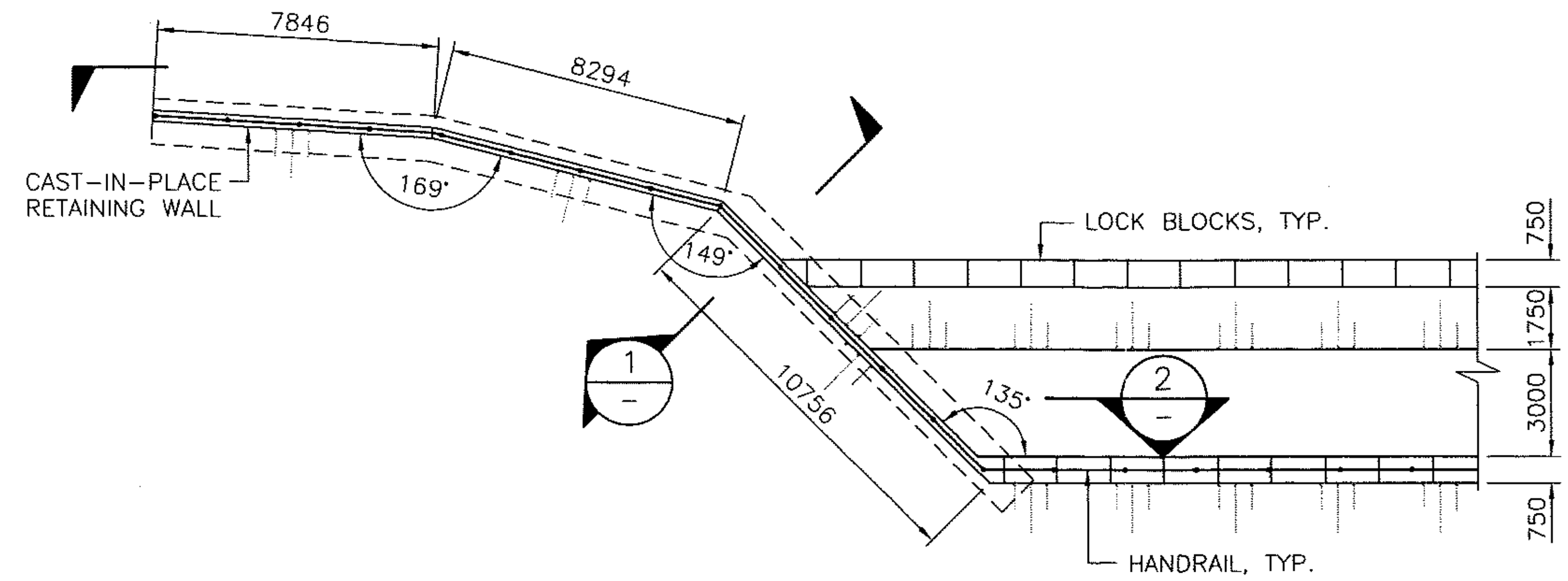
FLOOD MITIGATION WORKS
FLOODWALL – PLAN VIEW

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

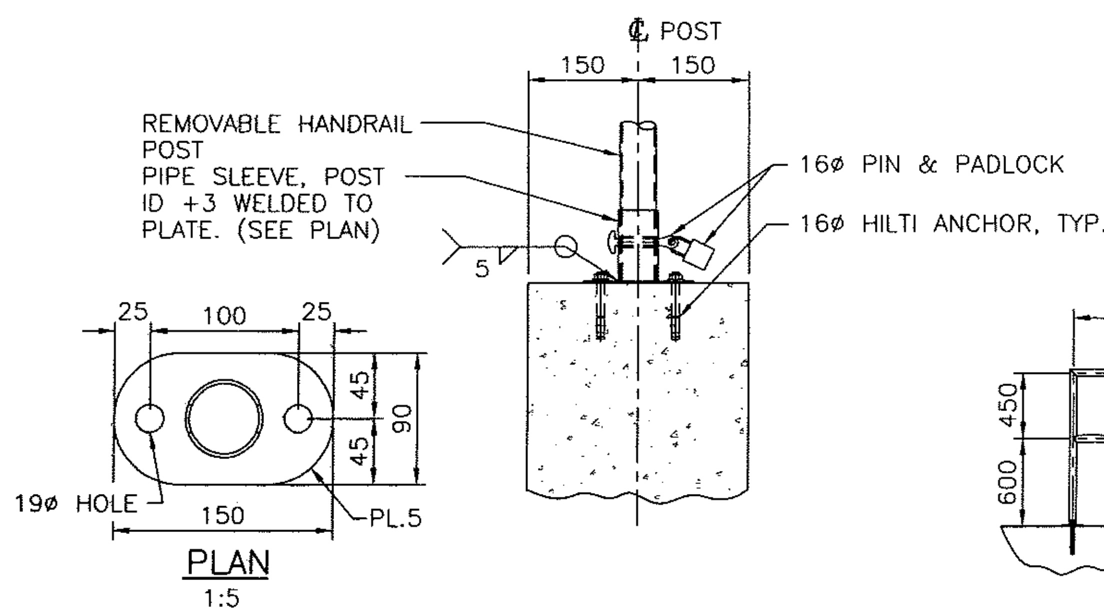
DRAWING NUMBER	REV. NO.	SHEET
032502-0-301	2	



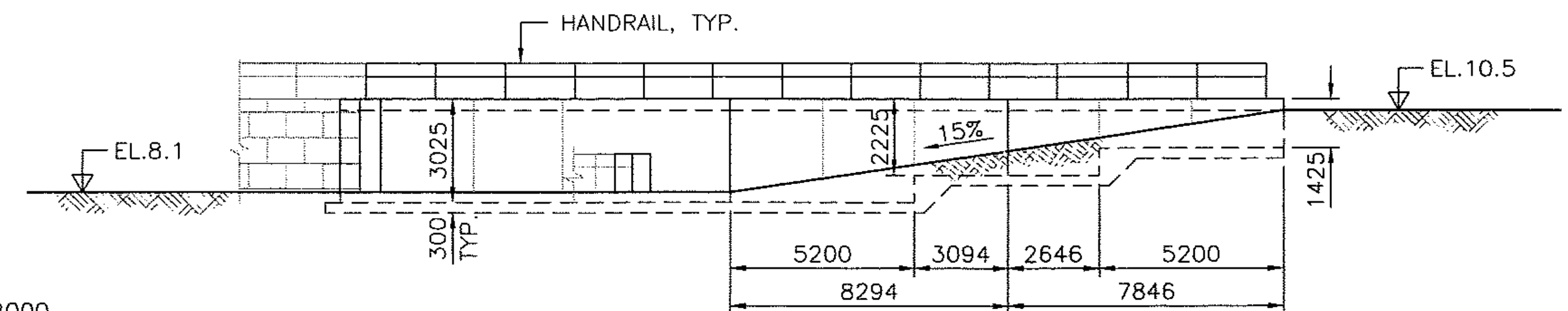
SECTION 1
1:50



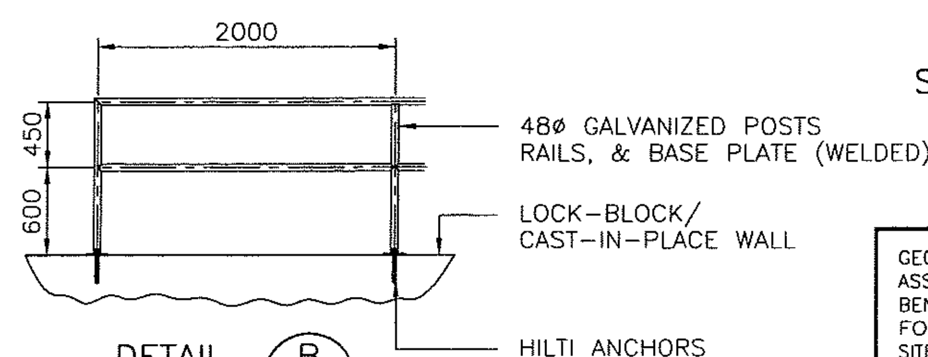
DETAIL A / 301
1:200



OPTIONAL REMOVABLE HANDRAIL DETAIL
1:10



SECTION 2
1:200



DETAIL B
1:50

GEOTECHNICAL ENGINEERING IS NOT INCLUDED WITHIN THE SCOPE OF SERVICES BEING PROVIDED BY ASSOCIATED ENGINEERING (B.C.) LTD. THEREFORE THIS DESIGN HAS BEEN PREPARED WITHOUT THE BENEFIT OF GEOTECHNICAL FIELD INVESTIGATION OR ADVICE. GROUND CONDITIONS MAY VARY. THE FOUNDATION REQUIREMENTS MAY NEED TO BE MODIFIED TO ACCOMMODATE ACTUAL SITE CONDITIONS ENCOUNTERED DURING CONSTRUCTION. ASSOCIATED ENGINEERING (B.C.) LTD. ACCEPTS NO LIABILITY OR RESPONSIBILITY FOR DELAY OR ADDITIONAL COSTS WHICH MAY RESULT IF GROUND CONDITIONS VARY FROM THOSE ASSUMED. PLEASE CONTACT ASSOCIATED ENGINEERING (B.C.) LTD. IF GEOTECHNICAL DATA AND RECOMMENDATIONS ARE AVAILABLE FOR THIS SITE OR IF THE OWNER WISHES TO EXPAND THE ENGINEERING SERVICES TO INCORPORATE THIS EXTRA WORK.

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	R.K.	S.P.	PRELIMINARY DESIGN
1	JAN. 2004	R.K.	S.P.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

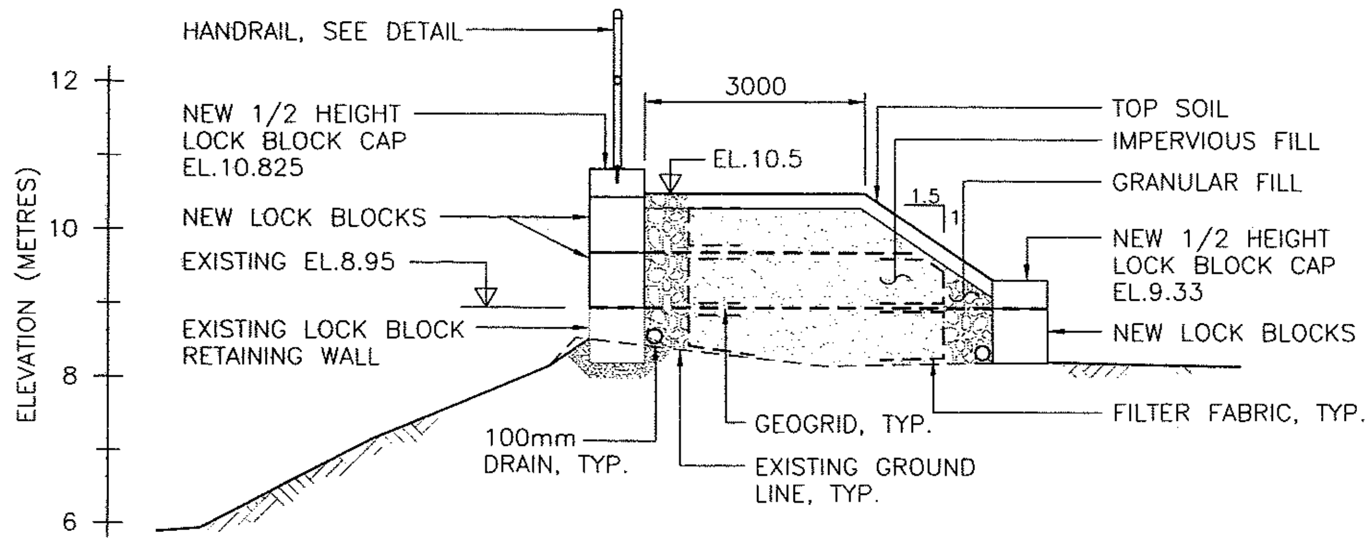
PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	S.P.		JAN. 2004
DESIGNED	R.K.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

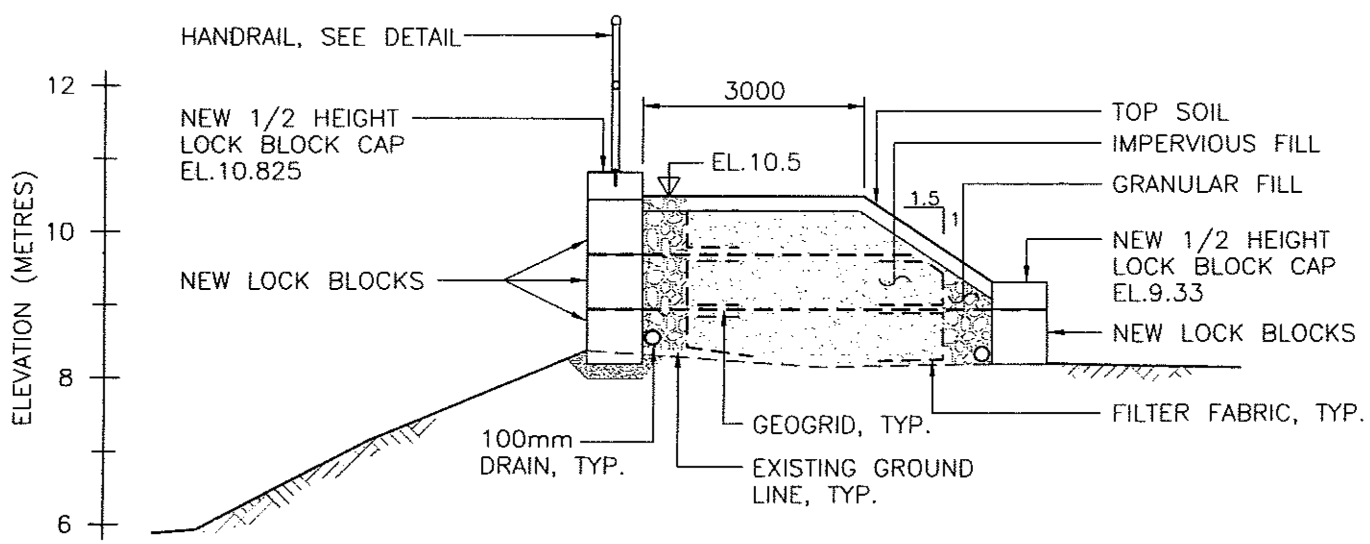
FLOOD MITIGATION WORKS
FLOODWALL – STRUCTURAL DETAILS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

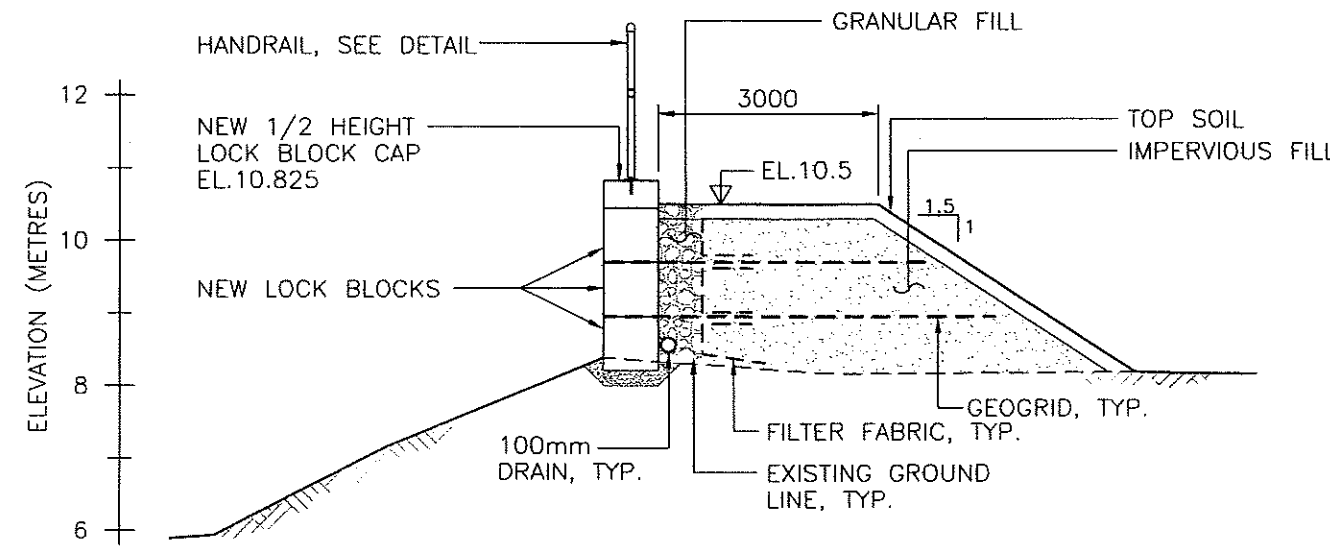
DRAWING NUMBER	REV. NO.	SHEET
032502-0-302	2	



SECTION 3
1:100



SECTION 4
1:100



SECTION 5
1:100

GEOTECHNICAL ENGINEERING IS NOT INCLUDED WITHIN THE SCOPE OF SERVICES BEING PROVIDED BY ASSOCIATED ENGINEERING (B.C.) LTD. THEREFORE THIS DESIGN HAS BEEN PREPARED WITHOUT THE BENEFIT OF GEOTECHNICAL FIELD INVESTIGATION OR ADVICE. GROUND CONDITIONS MAY VARY. THE FOUNDATION REQUIREMENTS MAY NEED TO BE MODIFIED TO ACCOMMODATE ACTUAL SITE CONDITIONS ENCOUNTERED DURING CONSTRUCTION. ASSOCIATED ENGINEERING (B.C.) LTD. ACCEPTS NO LIABILITY OR RESPONSIBILITY FOR DELAY OR ADDITIONAL COSTS WHICH MAY RESULT IF GROUND CONDITIONS VARY FROM THOSE ASSUMED. PLEASE CONTACT ASSOCIATED ENGINEERING (B.C.) LTD. IF GEOTECHNICAL DATA AND RECOMMENDATIONS ARE AVAILABLE FOR THIS SITE OR IF THE OWNER WISHES TO EXPAND THE ENGINEERING SERVICES TO INCORPORATE THIS EXTRA WORK.

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	R.K.	S.P.	PRELIMINARY DESIGN
1	JAN. 2004	R.K.	S.P.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

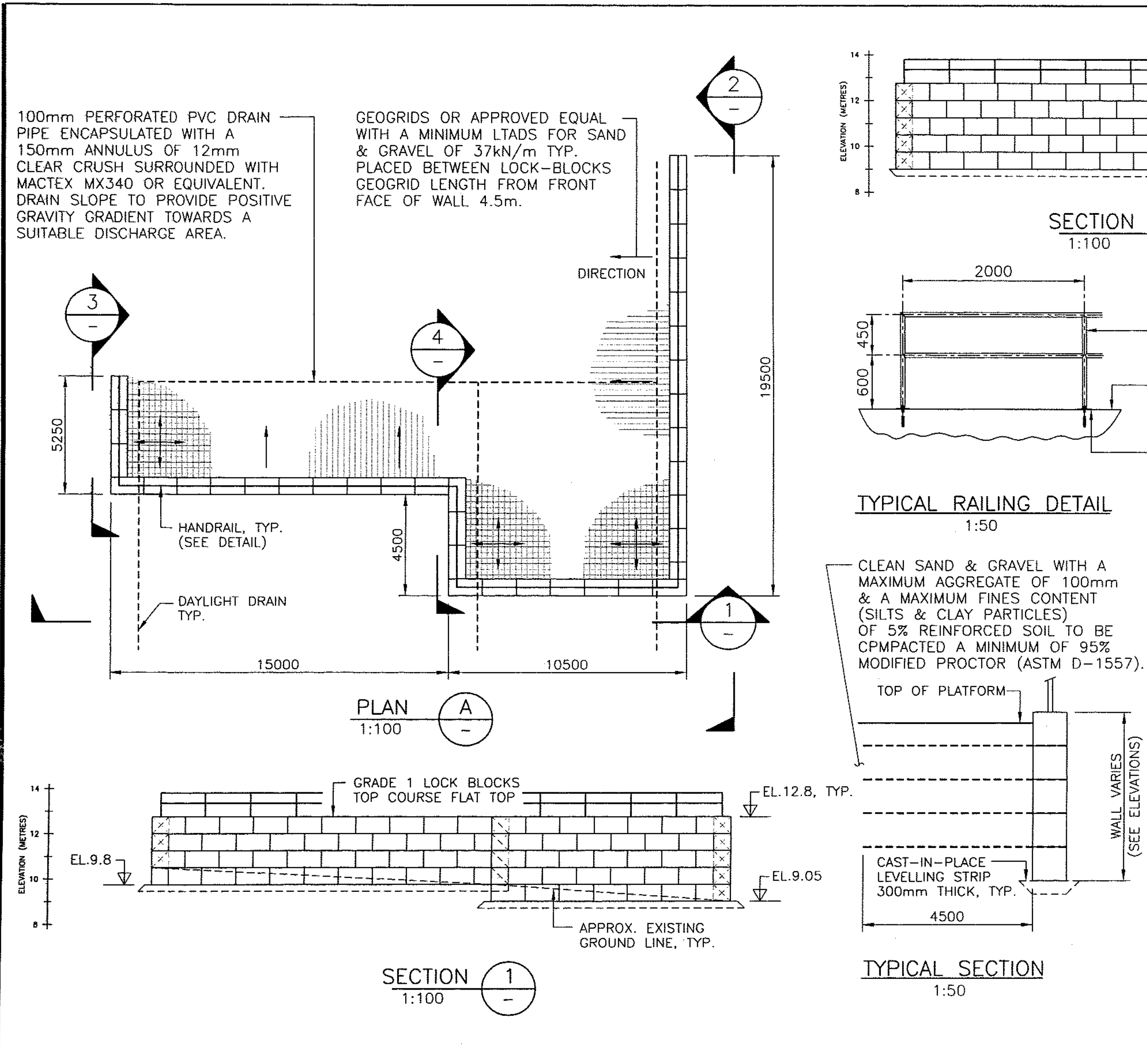
PROJECT No.	032502		
SCALE	AS SHOWN		
DRAWN	S.P.		JAN. 2004
DESIGNED	R.K.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

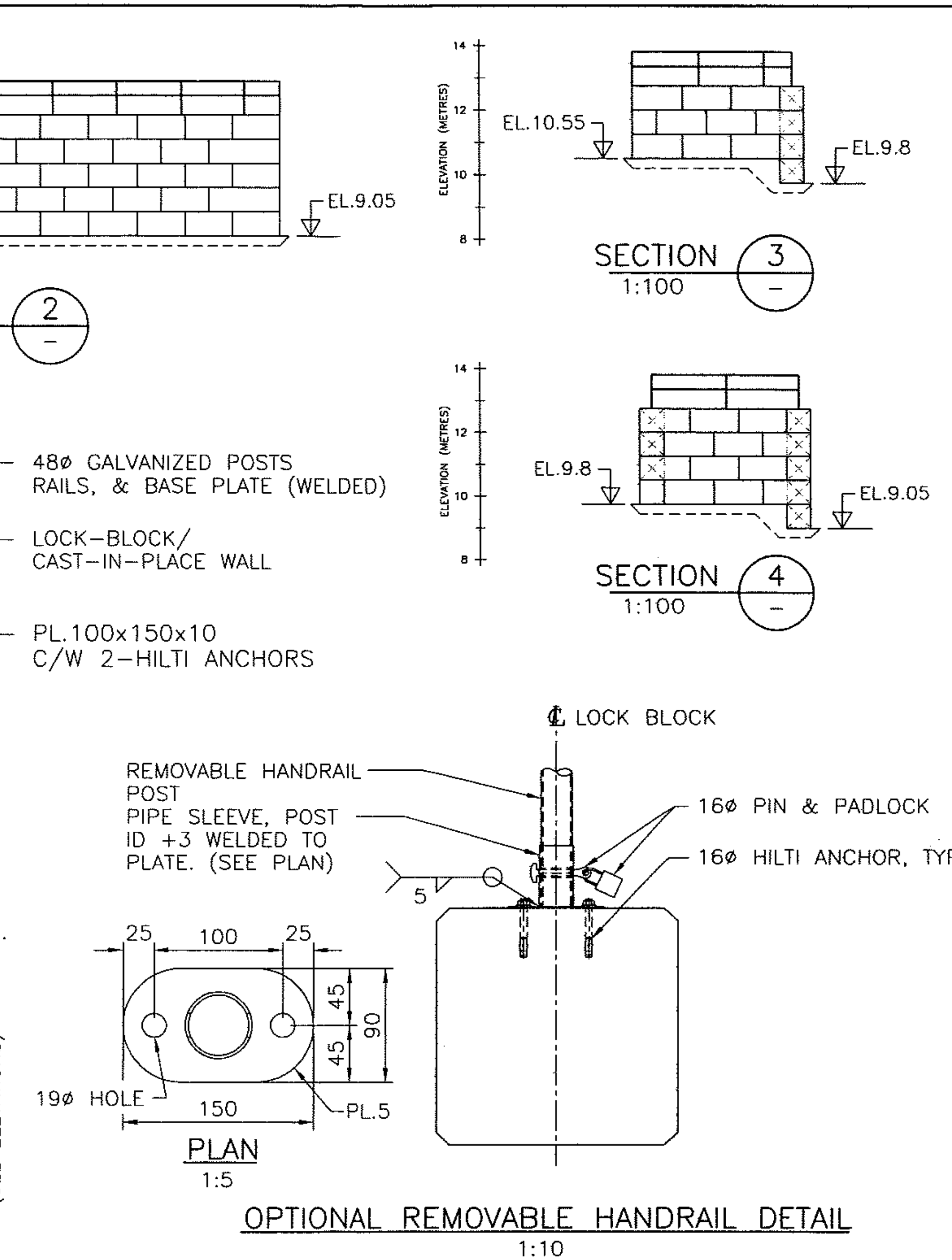
CITY OF PORT COQUITLAM

FLOOD MITIGATION WORKS SECTIONS

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-303	2	





GEOTECHNICAL ENGINEERING IS NOT INCLUDED WITHIN THE SCOPE OF SERVICES BEING PROVIDED BY ASSOCIATED ENGINEERING (B.C.) LTD. THEREFORE THIS DESIGN HAS BEEN PREPARED WITHOUT THE BENEFIT OF GEOTECHNICAL FIELD INVESTIGATION OR ADVICE. GROUND CONDITIONS MAY VARY. THE FOUNDATION REQUIREMENTS MAY NEED TO BE MODIFIED TO ACCOMMODATE ACTUAL SITE CONDITIONS ENCOUNTERED DURING CONSTRUCTION. ASSOCIATED ENGINEERING (B.C.) LTD. ACCEPTS NO LIABILITY OR RESPONSIBILITY FOR DELAY OR ADDITIONAL COSTS WHICH MAY RESULT IF GROUND CONDITIONS VARY FROM THOSE ASSUMED. PLEASE CONTACT ASSOCIATED ENGINEERING (B.C.) LTD. IF GEOTECHNICAL DATA AND RECOMMENDATIONS ARE AVAILABLE FOR THIS SITE OR IF THE OWNER WISHES TO EXPAND THE ENGINEERING SERVICES TO INCORPORATE THIS EXTRA WORK.

NO.	DATE	ENG.	BY	SUBJECT
2	FEB. 2004	R.K.	S.P.	PRELIMINARY DESIGN
1	JAN. 2004	R.K.	S.P.	DRAFT

REVISIONS

VERIFY SCALES
BAR IS 20mm ON ORIGINAL DRAWING
0 — 20mm
IF NOT 20mm ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PRELIMINARY
NOT FOR CONSTRUCTION

ASSOCIATED ENGINEERING

PROJECT No.	032502		
SCALE	1:500		
DRAWN	S.P.		JAN. 2004
DESIGNED	R.K.		JAN. 2004
CHECKED			
APPROVED			
DATE		INITIAL	

CITY OF PORT COQUITLAM

EQUIPMENT LANDING AREA
RETAINING WALL

FLOOD HAZARD MITIGATION OPTIONS
COQUITLAM RIVER

DRAWING NUMBER	REV. NO.	SHEET
032502-0-304	2	