DISTRICT OF WEST VANCOUVER
INTEGRATED STORMWATER MANAGEMENT PLAN FOR
PIPE, WESTMOUNT,
CAVE, TURNER AND GODMAN CREEKS
DISTRICT OF WEST VANCOUVER

DECEMBER 2013

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INTEGRATED STORMWATER MANAGEMENT PLAN FOR PIPE, WESTMOUNT, CAVE, TURNER AND GODMAN CREEKS

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

APPENDICES

LIST OF TABLES

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

LIST OF FIGURES

DISTRICT OF WEST VANCOUVER
INTEGRATED STORMWATER MANAGEMENT PLAN FOR PIPE, WESTMOUNT, CAVE, TURNER AND GODMAN CREEKS

EXECUTIVE SUMMARY

This report summarizes the climate and drainage characteristics of the five watersheds (6.2 km^2) for the Pipe (1.7 km^2), Westmount (1.1 km^2), Cave (0.9 km^2), Turner (0.7 km^2) and Godman (1.8 km^2) creeks. An Integrated Stormwater Management Plan (ISMP) is developed to protect the watershed health of the five creek drainage areas and to provide a master drainage plan for securing drainage protection within this study area, including priorities and costs for major and minor improvements in a staged business plan. The ISMP is designed to protect life and properties from flood and erosion hazards, maintain public safety through creek management, and protect fisheries and wildlife habitat.

The District of West Vancouver retained Opus DK to lead the investigation with assistance from InterCAD Services Ltd., SLR Environmental, Golder Associates Ltd., and Webster Engineering. Northwest Hydraulic Consultants provided flow data for model calibration. Aqua-Tex Scientific Consulting Ltd. prepared the draft Proper Functioning Condition Assessment for Pipe, Westmount, Cave & Turner Creeks dated 2011.

The drainage networks for the five watersheds were modeled in PCSWMM and flood protection for the area was analyzed under the 200-year storm event. Environmental protection ensured that a base flow of 50% of the Mean Annual Rainfall (MAR) remained in the creek system under various conditions. Maximum Permissible Velocities (MPV's) of the creek channel sections and observed conditions were used to analyze potential erosion problems in the creeks. Best Management Practices (BMP) secure environmentally sustainable solutions for habitat and the

public benefit, and include the management of stormwater quality. This was achieved through the assessment of various Low Impact Development (LID) techniques that have been recommended in order of priority as part of the study.

The existing drainage system was analyzed under the 200-year return period event. Eighteen culverts and 24 channel sections below the Upper Levels Highway were deemed inadequate to safely convey the resulting peak flows from this event.

The construction of detention storage facilities and the construction of a diversion pipe to control the runoff from large storms were considered as potential stormwater solutions to attenuate peak flows during the designated storm. Detention storage facilities were modeled as 1 m depth (for safety) at the upper reaches of the creeks at or above the Upper Levels Highway. The size requirements of the detention storage facilities were considered too large to fit in the steep terrain of the five creeks, and the diversion pipe solution was recommended. The diversion pipe was modeled to attenuate 200-year flows under four scenarios. Diversion inlet and pipe sizes were modeled and a cost estimate was developed for each scenario below:

- Diversion for Existing Conditions Only
- Diversion for Post-Development Conditions above Highway One
- Diversion for Post-Development Conditions above Highway One with a 25% increase in impervious area to the developed lands below Highway One.
- Diversion for Post-Development conditions above Highway One, but only diverting flows greater than the 25-year flow.

The diversion was also sized to control runoff from small storms, and its attenuation of peak flows aids in environmental protection.

Environmental protection was recommended by an analysis of LID measures including absorbent soils, permeable pavers, roof runoff collection in rock pits, and wetland infiltration and/or rain gardens. Recommendations include a schedule of improvements to enhance and preserve general public safety, convenience, and natural habitat amenities in the study area.

Two management options were developed to address concerns related to life and property safety. The two options included:

- Option A - Construct the diversion pipe as defined in Scenario 2. This diversion pipe would be sized for maximum risk aversion and would minimize the number of downstream works required.
- Option B - Construct the diversion pipe as defined in Scenario 4. This diversion pipe would be smaller than in Option A and hence less expensive to build initially. However, it would result in additional downstream works as well as the need to accept a higher risk of damages to private and public property.

A summary of the cost estimates for Options A and B are estimated as follows:

Description	Major Cost	Minor Cost	O&M	Total
Option A	$9,030,200.00	$479,368.00	$95,096.00	$9,604,664.00
Option B	$7,412,850.00	$1,725,725.00	$91,386.00	$9,229,961.00

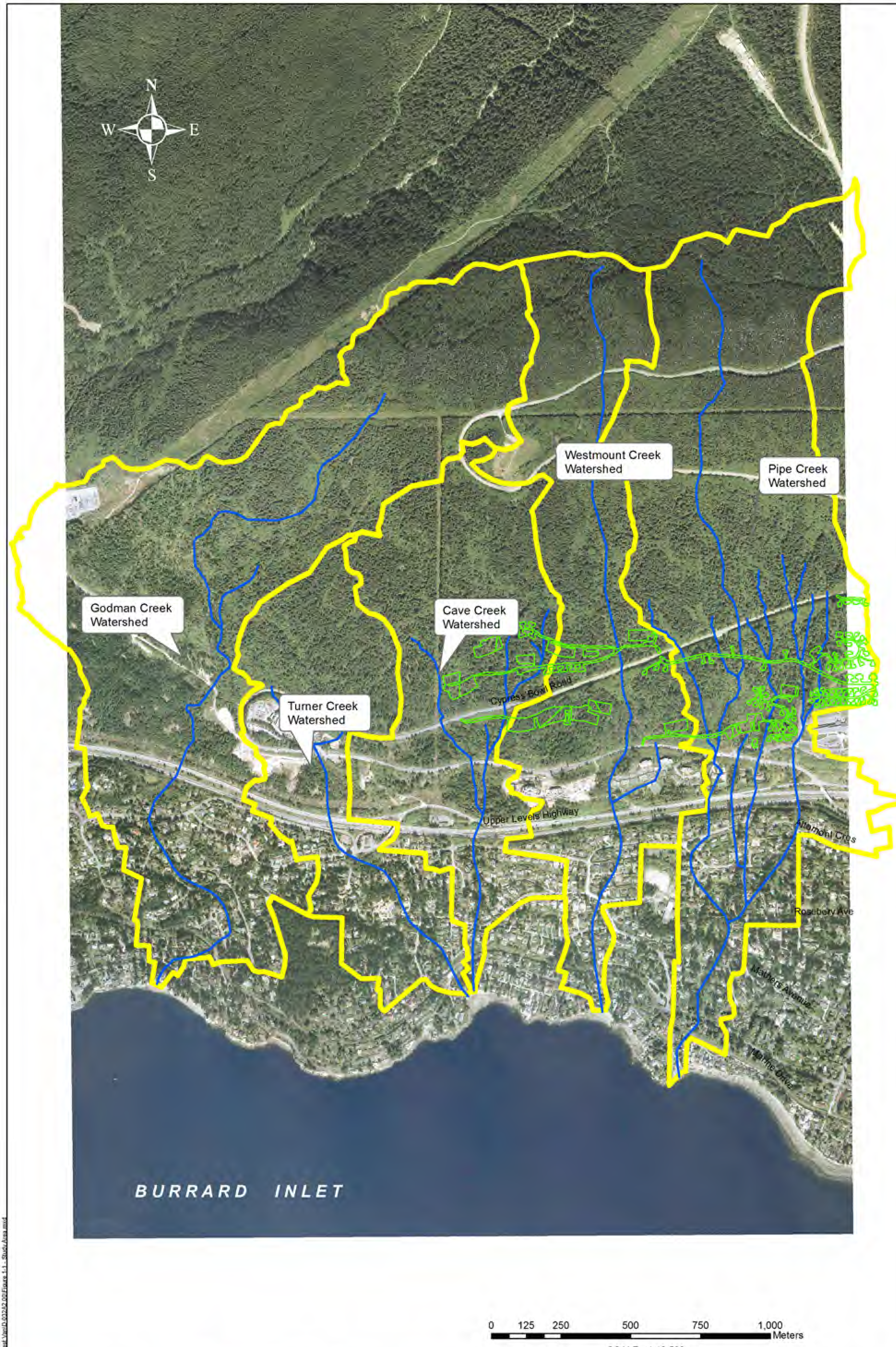
Diversion Option A is recommended for implementation as it provides a higher level of protection to downstream life and property while not resulting in a significant increase to overall costs. The implementation strategy prioritizes the recommended upgrades.

DISTRICT OF WEST VANCOUVER
INTEGRATED STORMWATER MANAGEMENT PLAN FOR PIPE, WESTMOUNT, CAVE, TURNER AND GODMAN CREEKS

1.0 INTRODUCTION

The five watersheds under study in this report total about 6.16 km^2 (616 ha) and drain as a line of fall of steep mountain drainage along the south face of the Coastal Mountain range of West Vancouver to Burrard Inlet . The study drainage area and five creek headwaters are bounded on the north at about 790 m GSC and on the west by the largest drainage area in West Vancouver, Cypress Creek (about 13.3 km^2). The east limit is bounded by the fifth largest drainage area in West Vancouver, Rodgers Creek (about 3.3 km^2). Within the study area in order of size, the largest of the five drainage areas, Godman Creek (the tenth largest drainage area in West Vancouver) is about 1.8 km^2; this is followed by Pipe Creek at 1.7 km^2 and by Westmount Creek the fifteenth largest at 1.1 km^2. The two others include Cave Creek at 0.9 km^2 and Turner Creek at 0.7 km^2. The study area is illustrated on Figure 1-1.

Above the 200 m GSC contour and within the study area (Upper Levels Highway 1 and lower Cypress Bowl Road), British Pacific Properties Ltd (BPP) is undertaking six residential housing developments as part of the Rogers Creek Neighborhood Proposed Area Development Plan. The development stretches west from Marr Creek and Rodgers Creek through the study area, which from east to west includes several Pipe Creek drainage tributaries, Westmount Creek tributaries, and Cave Creek tributaries. Future planned residential development is anticipated to extend further west to the Cypress Creek watershed, including Turner Creek and Godman Creek



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GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
STUDY AREA AND CREEK WATERSHEDS

FIGURE 1-1

tributaries. The total current (below highway) and future potential development lies across about 0.93 km^2 of the five watersheds, or about 15% of the total drainage area for the five watersheds. Figure 1-1 identifies the watershed locations and study area. The Turner Creek watershed is shown to be left untouched in current development plans.

The primary purpose of this report is to provide an integrated stormwater management plan (ISMP) to protect the health of the five creek drainage areas, and to provide a master drainage plan for securing drainage protection within this study area, including priorities and costs for major and minor improvements in a staged business plan. The report identifies best management practices (BMP) to secure environmentally sustainable solutions for habitat protection and public benefit. Recommendations include a schedule of improvements to enhance and preserve general public safety, convenience, and natural habitat amenities in the study area.

Metro Vancouver (formerly the Greater Vancouver Regional District) prepared guidelines to undertake drainage studies that integrate neighborhood planning, land use planning, environmental health, and watershed protection and restoration safeguards (GVRD, 2002) . Tools included in the guidelines for measuring the current state of a watershed and the success of the process include:

a) a watershed classification system;
b) a measurement of the effective impervious surface area in the watershed;
c) protection and or redevelopment of riparian area; and,
d) measurement of creek biota diversity

This report is developed in eight sections not including summaries and appendices. The eight sections represent the project development including study definition, background, criteria, current conditions, analyses of alternatives and recommended solutions. Appendices provide supporting detail including terms of reference, and supplementary reports. A glossary of terms and references is given before the appendices.

1.1 Objectives

The objectives of this study were as follows:

- prepare an integrated stormwater management plan (ISMP) for the Pipe, Westmount Cave, Turner and Godman Creek watersheds to help achieve the goals of the ISMP guideline document; and

- provide a plan for developing the stormwater drainage improvements for protection of life and property in the planned developed area and in the five downstream currently developed watersheds of West Vancouver.

1.2 Scope of Work

To meet the study objectives and schedule, the investigation was carried out in a phased program that included the following scope of work:

1. Identify Watershed and Regional Character:
   - 1.1 Review existing stormwater program and historic data,
   - 1.2 Collect hydrometric data, and determine catchment response to rainfall,
   - 1.3 Prepare inventory of drainage system, watercourse characteristics and develop a partial inventory of instream hydraulic structures such as culverts and bridges that form significant barriers for major flow; Delineate drainage basins for both internal and possible external drainage and define the sub-basin boundaries.
   - 1.4 Undertake hydrometric and geological assessment; identify BMP opportunities for infiltration and other,
     - 1.4.1 Assemble relevant geological hydrogeological criteria
     - 1.4.2 Undertake field reconnaissance of channels in consideration of 1 to 200 year flow capacities
     - 1.4.3 Identify natural hazards and impact on drainage concepts

1.4.4 Provide recommendations for ISMP measures that reflect the character and constraints of the watersheds

1.4.5 Prepare comments on infiltration capacity within the development.

1.5 Assemble environmental information and identify enhancement opportunities:

1.5.1 Physical stream parameters

1.5.2 Aquatic and riparian habitat

1.5.3 Terrestrial wildlife habitat

1.5.4 Environmentally sensitive areas

1.5.5 Wetland delineation

1.6 Assemble planning information for land use to identify pervious impervious ratios and riparian area protection.

2. Undertake technical analysis:

2.1 Develop design criteria for hydrology, hydraulics, minor and major flow apportioning for open and closed drainage. Estimate design flows and volumes to determine hydraulic analysis requirements,

2.2 Assemble and develop hydraulic model entry data for OCP planning, meteorology, land use, hydrology, and major and minor sewer collection systems (Q10, Q200) as well as high frequency low intensity storms.

2.3 Develop Best Management Practice (BMP) and Low Impact Development (LID) guidelines for the study area, identify mitigative solutions for erosion control and sediment transport,

2.4 Assemble habitat protection requirements and determine constraints for undertaking the drainage investigations, including agency needs and structural requirements.

3. Assess Alternatives

3.1 Undertake PCSWMM modeling for minor system for Q10 and Q200 flows and determine flood routing for major flows.

4. Prepare Integrated Stormwater Management Plan
   4.1 Develop master drainage plan to delineate minor and major improvements and cost for priorities in a business plan.
   4.2 Prepare master drainage plan to illustrate requirements of development proponents to meet the stormwater management needs of the study area.

1.3 Limitations

This study was limited to the drainage areas surrounding the five creeks, and did not include evaluation of the Cypress Creek, Rogers Creek or Marr Creek areas except to recognize constraints of the adjacent drainages to urban drainage and the planned development; (this work was done by others and is referenced in the text).

This study did not include extensive planning level modeling but did include design and analysis modeling of piped systems for minor protection and major flood routing. Municipal records were used for existing drainage works. The model was limited to single event storm conditions and was restricted to pipe or channel flows. Topographic plans were used to identify probable gradient and basin dimension ratios. The modeling work assessed impacts of flows from frequent rainfall events and the increase of these flows as a result of development. In examining existing drainage, the minor drainage sub-basin storm drain capacity was examined independent of diversion options that would consider integrating the drainage solution for Rodgers and Marr Creeks. Examination of these minor system diversions would extend beyond the terms of reference.

Culvert analysis was also limited to those identified below the Upper Levels Highway, including culverts crossing the highway. An inventory of all culverts above the Upper

Levels Highway has been completed by InterCAD. An analysis into these culverts is beyond the scope of this report.

While this study does not explicitly analyze the impact of climate change, there are recommendations for specific components, such as pipe and inlet sizing, which would accommodate increased variability from climate change compared to current District drainage policy.

The development plans reflect the District of West Vancouver and British Pacific Properties Ltd. planning and refer to the Official Community Plan Bylaw No. 4360, 2005 as amended by Bylaw No. 4567, 2008 (Rodgers Creek Area) . Changes beyond this plan will be the responsibility of future development proponents.

1.4 Conduct of Study

This investigation included the integration of land use plans, community goals, geological, environmental and hydrological recommendations. The ISMP was prepared under the guidance and direction of the District of West Vancouver who ultimately approves the program, and with public stakeholder and agency involvement. British Pacific Properties Ltd. provided the overall site development planning, and the liaison and notification to the public stakeholders.

The study was undertaken through a phased program to secure a complete integrated stormwater management plan for the study drainage area. The phases were initiated with meetings and required interim meetings and discussion for guidance.

Study direction was identified and data were collected for review. A stakeholder's meeting was held on June 14, 2009 to confirm approach and to receive comments from the stakeholders for study direction.

1.5 Acknowledgements

We are grateful to the District of West Vancouver for their assistance in the preparation of this report and in particular to Mr. Ray Fung, M.Eng, P.Eng, Director of Engineering and Transportation, Mr. John McMahon, M.Eng, P.Eng, Manager Utilities, Mr. Tony Tse, P.Eng, Land Development Engineer, Andy Kwan, P.Eng, Utilities Engineer and Ms. Jenn Veenstra, B.A.Sc., Engineering and Transportation Technologist. Mr. Geoff Croll, P.Eng and Mr. Walter Thorenloe, P.Eng provided overall study coordination for BPP Ltd. Mr.
Richard Cook of Jorden Cook Associates provided planning and coordination with the Wong property owners and consultants.

The report was prepared with the assistance of Sean Rooney, EIT, and Clive Leung, EIT, who undertook model development for Dayton & Knight under the direction of Allan Gibb,Ph.D., P.Eng. and Harlan Kelly, P.Eng.; James Neville R.P.Bio, John McCulloch, P.Biol and James Malick PhD, R.P.Bio, P.Ag P.Eng. of SLR undertook environmental integration of the plan; Mathew Munn P.Eng, Mark Gold, B.A.Sc, P.Eng, Russell Wong P.Geo., Brad Panton, EIT, and Andrew Nelson, field technician of Golder Associates undertook the geological assessment. Mr. Richard Skapski P.Eng, and Mr. Iain Lowe of InterCAD Consulting provided overall guidance on the storm drainage practicalities and planning infrastructure. Paul Webster, P.Eng., and John Tynan, EIT, of Webster Engineering Ltd. provided input on the storm drainage practicalities and planning infrastructure of the Wong property. Creek flow data for model calibration was provided by Northwest Hydraulic Consultants Ltd. Aqua-Tex Scientific Consulting Ltd. prepared the 2011 Proper Functioning Condition Assessment for Pipe, Westmount, Cave & Turner Creeks.

2.0 EXISTING AND FUTURE LAND USE

The drainage area contains mainly single family residential housing from the Upper Levels Highway to the Seawall. Mixed commercial and residential areas are near the seawall, and public parks and schools scattered throughout the area. The areas above elevation 365 m GSC (1200 feet) are protected parks areas. No agricultural or heavy industries are in the drainage area.

Development according to the District of West Vancouver (2004) planning is to focus on the construction of residential development above the Upper Levels Highway. The land below the Highway is largely built out and only redevelopment is possible. Residential development in the study area is to remain below elevation 365 m GSC. Land use above this elevation is mainly for recreational purposes that will generally maintain the natural environment.

The planning information was used to identify model runoff parameters and land use. The breakdown of developed versus natural areas is shown in Table 2-1.

Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 2-1 - Existing Study Area with Aerial.mxd

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GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP

EXISTING DEVELOPMENTS

FIGURE 2-1

TABLE 2-1
WATERSHED AND LAND USE AREAS

	Godman	Turner	Cave	Westmount	Pipe	Total
Total Area	182 ha	66 ha	88 ha	106 ha	173 ha	616 ha
Natural Forest Area	144 ha (79%)	41 ha (62%)	75 ha (86%)	90 ha (85%)	140 ha (81%)	491 ha (80%)
Developed Area	38 ha (21%)	25 ha (38%)	13 ha (14%)	16 ha (15%)	33 ha (19%)	125 ha (20%)

The characterization of the five catchments within the Godman, Turner, Cave, Westmount and Pipe ISMP was carried out by Opus DaytonKnight through a review of the cadastral and aerial photographs provided by the developer and the District of West Vancouver. Delineation of the five catchments and subsequent sub-catchments was determined through the review of topographical contours from the District's GIS system as well as information provided by InterCAD. InterCAD provided ground surface information above Highway One derived from a variety of sources including ground survey, LIDAR mapping, aerial surveys and TRIM mapping from the Province of British Columbia.

2.1 Existing Impervious

Existing cadastral and aerial photographs were used to determine the existing parcels contributing to the study area. An overlay of the aerial photography was used to determine the impervious areas within each catchment. Impervious areas were summarized for the parcels and roads to determine a percent impervious area for each sub-catchment and have been recorded in Appendix F of the draft report.

Figure 2-1 shows the existing cadastral and aerial photograph used to determine the impervious areas calculated for the model. It is noted that development in all five creeks is essentially built out below the Upper Levels Highway and remaining buildout capacity is located on lands above the Highway.

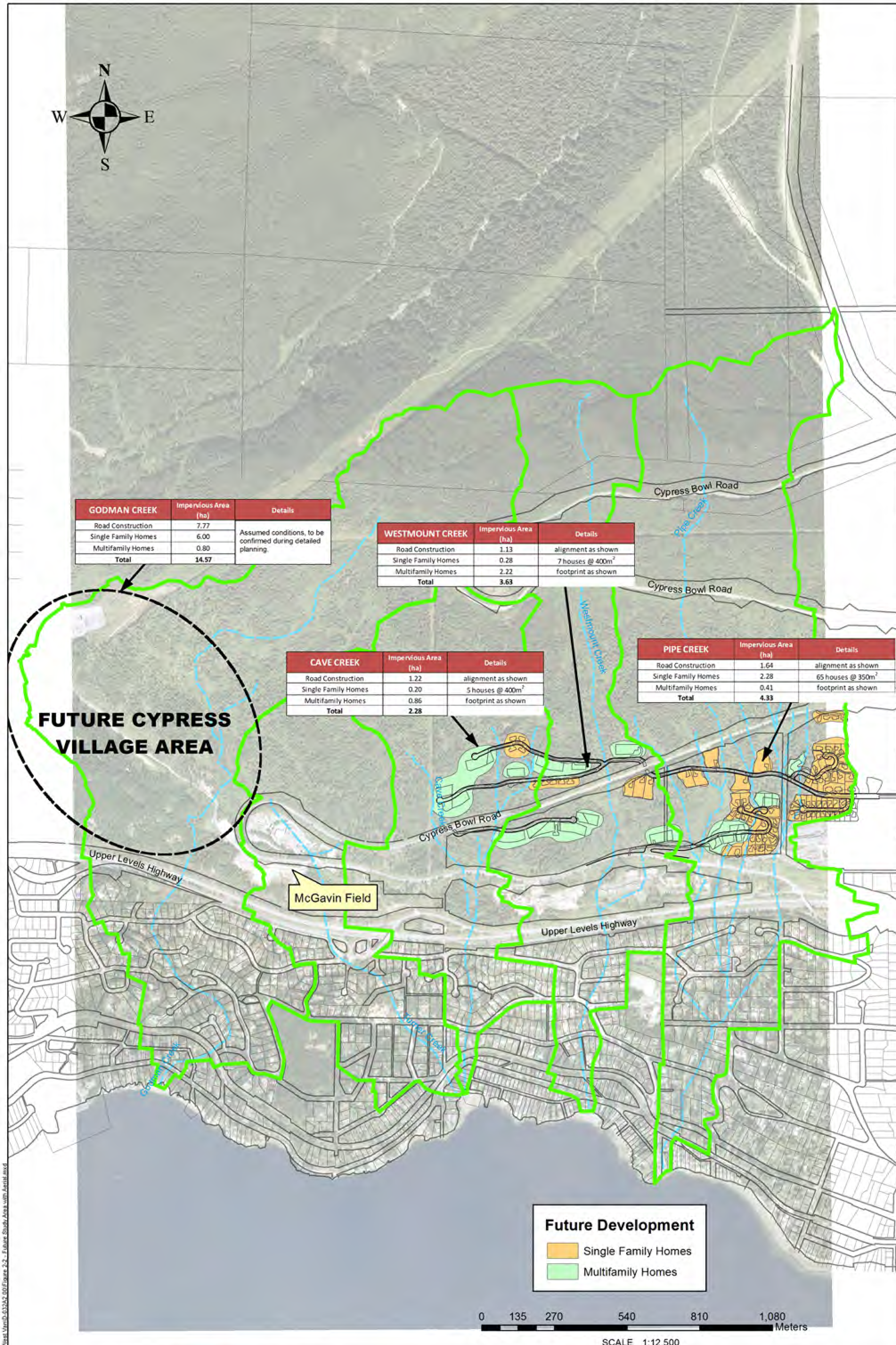
2.2 Future Impervious

Proposed development plans were provided for the Godman, Cave, Westmount and Pipe catchments above the highway. These plans were received by Opus DaytonKnight as follows:

1) Rodgers Creek Development Plans received from InterCAD on July 31, 2008. The drawings received included development plans for 'Area 3' (Wong, Roeck, et al lands).
2) Cypress Village Development Plans received from InterCAD on June 14, 2010.

The proposed cadastral for the new developments were overlain on the existing cadastral and aerial photographs. Impervious areas were approximated at each future parcel and were summarized and added to the existing impervious areas calculations to determine the future total impervious area. Figure 2-2 shows the assumptions made for future land use. These future impervious areas have also been recorded in Appendix F.

The figure identifies future land use for single and multiple family homes within the Godman, Cave, Westmount and Pipe drainages.



GODMAN CREEK	Impervious Area (ha)	Details
Road Construction	7.77	Assumed conditions, to be confirmed during detailed planning.
Single Family Homes	6.00	
Multifamily Homes	0.80	
Total	14.57	

WESTMOUNT CREEK	Impervious Area (ha)	Details
Road Construction	1.13	alignment as shown
Single Family Homes	0.28	7 houses @ 400m^2
Multifamily Homes	2.22	footprint as shown
Total	3.63	

CAVE CREEK	Impervious Area (ha)	Details
Road Construction	1.22	alignment as shown
Single Family Homes	0.20	5 houses @ 400m^2
Multifamily Homes	0.86	footprint as shown
Total	2.28	

PIPE CREEK	Impervious Area (ha)	Details
Road Construction	1.64	alignment as shown
Single Family Homes	2.28	65 houses @ 350m^2
Multifamily Homes	0.41	footprint as shown
Total	4.33	

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GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
FUTURE DEVELOPMENTS

FIGURE 2-2

Path: H:\Projects\000032 West Van\D-032A2.00\Figure 2-2 - Future Study Area with Aerial.mxd

3.0 WATERSHED CHARACTERIZATION

This section provides a template for defining the drainage solutions. Descriptions of the physiographic, climatological and bioclimatic characteristics of the region and study area are included. Geology and topography are described to define surface drainage constraints. Past and recent studies are referenced to illustrate historic and current planning. The study area climate, geology, geomorphology, receiving waters and land use are identified to establish a context for drainage solutions. Lastly, this section identifies natural resources including overall environmental objectives as well as water quality issues and habitat. This information was used to derive general and specific solutions for securing the water resource needs of the study area. Subsequent sections provide criteria and specific analyses.

3.1 Study Area

The study area physiography, including climate, geology, geomorphology, soils and land use are briefly described to support background for selection of rainfall-runoff parameters. Geology, soils, bioclimate and climatic conditions determine the amount of rainfall that becomes runoff.

Figure 2-1 in Section 2 illustrates the study area and the five primary drainage watersheds.

The drainage study area of 6.16 km^2 includes five mountain stream watersheds that drain the lower southern face of the north shore mountains to Burrard Inlet. The five watersheds are drained by single to multi-branched creeks over a slope of about 30% to 35% from the headwaters to the Trans Canada Highway (TCH), and about 25% to 30% as essentially five single defined creeks below the highway to the inlet. The creeks cross Cypress Bowl Road, the TCH, major traffic routes and the Canadian National Railway to the point of discharge at beaches along the West Vancouver waterfront. The major drainage areas from east to west include:

1) Pipe Creek;
2) Westmount Creek;
3) Cave Creek;
4) Turner Creek; and
5) Godman Creek

The major physiographic features surrounding the study area include Cypress Creek to the west and north, Hollyburn Ridge to the northeast, Rodgers Creek to the east, and Burrard Inlet to the south. The Wong development is a special study area and is located at the east corner of the Pipe Creek watershed.

3.2 Climate

The 6.16 km^2 study area lies on the north shore of Burrard Inlet at the west end of the Pacific Ranges of the coastal mountains on the south face of Hollyburn Ridge and Black Mountain. The area is dominated by Polar Maritime air and by south-westerly flows, with the Strait of Georgia moderating temperature extremes. The study area is in a region where oceanographic effects (i.e., rising ground elevations cooling humid air masses) increase rainfall intensities.

The rainfall pattern is highly seasonal, with pronounced wet and dry seasons. Frontal and low pressure systems predominate during winter in the study area, producing wet winters. During summer, the Alaskan low moves north and the Hawaiian high becomes a semi-permanent fixture, bringing drier weather.

Table 3-1 provides a summary of climatic normals for the West Vancouver Capilano Golf & Country Club recording rainfall gauge. The average total yearly rainfall is 2208.5 mm, with typically 97 percent of the precipitation in the form of rainfall. On average, 1593 mm, or 70 percent of the total annual precipitation, occurs during the 6 month period between October and March. The peak precipitation months are November, December, and January, when 42 percent of the average total yearly precipitation occurs.

Climate change is predicted to increase intensities and the frequency of intense storms, however volumes of precipitation are understood to remain largely unchanged.

TABLE 3-1
WEST VANCOUVER CAPILANO GOLF & COUNTRY CLUB
RAINFALL SUMMARY 1976-1998 (ELEVATION 200.9m)

Climate ID 1108825 49º 21'N 123º 07'W	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec.	Year
Daily Maximum Temperature	5.2	7.9	10.4	13.3	16.7	19.5	22.1	22.7	19.5	13.6	7.7	5.1	13.6
Daily Minimum Temperature	-0.3	0.7	2.3	3.9	6.4	9.4	11.5	12.1	9.7	5.7	1.7	-0.4	5.2
Daily Temperature	2.5	4.4	6.4	8.6	11.6	14.5	16.8	17.4	14.7	9.7	4.8	2.4	9.5
Extreme Maximum Temperature	10.4	14.0	16.3	22.1	25.3	27.6	29.4	29.7	26.9	20.5	13.2	10.4	29.7
Rainfall	270.6	215.1	190.6	169.6	130.5	112.3	73.5	75.0	112.1	226.6	352.2	280.5	2208.5
Snowfall	16.5	12.9	4.2	1.2	0	0	0	0	0	0.2	3.4	21.2	59.6
Total Precipitation	287.1	228.0	194.8	170.8	130.5	112.3	73.5	75.0	112.1	226.8	353.8	302.9	2267.5
Greatest Rainfall in 24 hrs	76.0	95.0	100.6	63.4	55.5	46.7	77.0	87.5	79.3	125.0	104.0	141.3	141.3
Years of Record	17	19	20	21	21	22	23	22	23	21	19	18	
Greatest Snowfall in 24 hrs	25.4	28.0	10.0	15.0	0	0	0	0	0	3.0	8.0	25.0	28.0
Years of Record	17	19	20	21	21	22	23	22	23	21	19	18	
Greatest Precipitation in 24 hrs	76.0	95.0	100.6	63.4	55.5	46.7	77.0	87.5	79.3	125.0	104.0	141.3	141.3
Years of Record	17	19	20	21	21	22	23	22	23	21	19	18	

* From Environment Canada – Climate Data Online

In the winter, the majority of precipitation is the result of continuous frontal storms, which cover wide areas ranging from 250 km^2 to 2500 km^2. Because of the high precipitation in winter months, maximum surface wetting and maximum runoff occurs during the winter.

In the summer, showers from weaker frontal storms bring most of the precipitation. Occasional thunder or convective type storms result from thermal stratifications causing instability in the atmosphere and intense cores of rainfall over concentrated areas. These convective type storms govern peak runoff conditions in the summer months.

The convective storm is in sharp contrast to the frontal storm. Long rainfall durations exceeding 1 to 12 hours or more are typical of frontal storms, while high intensities lasting for minutes are associated with the summer convective storms.

Snowfall is more common at higher elevations. Snowmelt would then contribute to runoff but would be after the precipitation event. Snowmelt volumes and comparison with the design rainfall events are discussed in Section 4.2.5. Model calibration with measured stream flow and recorded precipitation, includes the effects of precipitation, which can include snowmelt on creek peak flows and base flows.

The vegetation and surficial soils and underlying geological complex yield an annual cycle of groundwater levels and base flow discharge into the creeks. Dense vegetation, such as forest with underbrush, delays runoff and promotes evapotranspiration. In some areas such as Cypress Creek, the undulating surficial character of the forested mountain topography creates numerous pools and natural detention. In other areas such as Pipe Creek, the terrain is steep and flat, promoting sheet runoff. The surficial soils in some areas, when of a sufficiently loose composition, promote infiltration. Other areas contain till or bedrock.

During winter, rainfall is high, evapotranspiration is low, and infiltration recharges groundwater stored in the vegetation layer and surficial soils. During the summer low rainfall season, direct runoff is less than in winter, evapotranspiration is high, and stored groundwater releases into the base flow of the perennial creeks.

3.3 Geology and Soils

Field investigations to characterize the geology and soils in the area were conducted by Golder & Associates Ltd. The final report is attached as Appendix C and is summarized below.

The surficial character of the study area within the drainage area is characterized by dense, relatively low-permeability sediments such as minor bedrock, tills, glaciomarine deposits, glaciofluvial sediments, and shallow lake deposits including silts and peats and alluvial deposits. These colluvial sediments are underlain by dense till and/or granitic bedrock of the 130 million year old Mesozoic and Tertiary-aged Coast Plutonic complex, which have comparatively low permeability. Due to the restriction of the vertical movement of groundwater caused by the dense till and bedrock, most of the study area is found to be moderate to poorly drained.   Little opportunity for natural precipitation infiltration is available.

The geology of the study area provides insight into the runoff-infiltration-storage process, the stability of the surface deposits, and the underlying support structure, which allows an assessment of erosion and landslide potential under past and present drainage conditions. The topography of the geology in each individual creek is described in Section 3.6.

3.4 Geomorphology

Study of the geomorphology of valley and creek formations provides an understanding of the conditions which shaped the valley floor and stream forms. This information is used to establish existing capacities and channel stability for movement by down cutting or meandering.

The reaches of the creeks consist largely of sediment source zones and transportation zones. Little deposition is observed until the beach at the inlet water level. At the higher elevations the creeks experience relatively steep high energy stream flow; these are sediment source zones from which water and sediment are derived. The creek beds in these regions are characterised by bedrock, boulders and cobbles. Finer sediments are eroded and transported downstream. However, undulating topography will invariably show signs of deposition or sedimentation in protected reaches during low flow seasonal conditions. The drainage system variables in this zone determine its hydrologic products, which in turn establish the nature of channel morphology and sedimentary deposits in the transportation and deposition zones. The important variables to the morphology and mechanics of the source areas include time, slope, geology, climate and vegetation.

The zone of sediment transport occurs mainly below 200 m GSC below TCH, where the streambed slopes are less than 25 percent and the streams have largely been incorporated into the residential landscape. The mountain stream is a series of step-pools that are combinations of rock steps, boulder steps and riffle steps (low slopes) as opposed to meanderforms on low-gradient drainage areas, which dissipate and manage the natural energy flow of the water. Step-pool structures are formed by an armouring process occurring at relatively high flows and the channels are extremely stable under usual flow conditions. In forested catchments, large organic debris also form steps in steep mountain streams (Thorne, Bathurst and Hay, 1987). For the West Vancouver mountain streams, the steps at the high gradients are on average less than 2 m long at the 25% to 35% grades. Less steep areas at lower elevations are often channelled through man-made flumes and may include riffle-steps. High flows can flush out the steps. However, to conserve stream energy, the steps will naturally reform as flow decreases. The streams are largely in the Zone 1 (sediment source) and Zone 2 (sediment transport). The Zone 2 erosion source is supply limited, which suggests energy is available for continued erosion and transport.

This is evident in visible erosion sites seen as cutbanks. Soil creep provides sediment source as well as is evident in areas showing “jack strawed” trees.
Generally, the creeks are vertically stable and will not rapidly incise below the present profiles. As noted, active lateral erosion has been observed. Smaller tributaries are more susceptible to erosion during high intensity low frequency (100 to 200-year) storm flows. Increased inflows to the main channels will create increased erosion-soil transport-deposition and habitat losses in lower reaches. This needs consideration when high frequency storm runoff events are investigated.

Upper elevation collection areas, where thin soil mantle is present shows evidence of numerous parallel shallow streams. These are often blocked by natural or manmade constructions, causing realignment and changes in flow paths. The Pipe, Westmount and Cave systems show evidence of these natural flood path changes. In many instances, the effects of abandoned logging roads and related operations impact the drainage efficiency in these sites.

The drainage areas regardless of the steepness all contain undulating topography and variably sized surficial barriers and depressions where water can be rerouted or retained. These and other heterogeneous soil/slope conditions throughout the watershed are not easily simulated in generic modelling tools, which assume relatively homogeneous conditions throughout defined areas.

3.5 Receiving Waters

The receiving water is Burrard Inlet; creeks discharge into fans of fluvial material, often disappearing below the beach before entering the ocean at low tide.

Field observations by Opus DK noted that tidal flood levels did not appear to reach the creek discharge outlets, and therefore would not affect flooding in the areas upstream of the drainage structures.

3.6 Topography

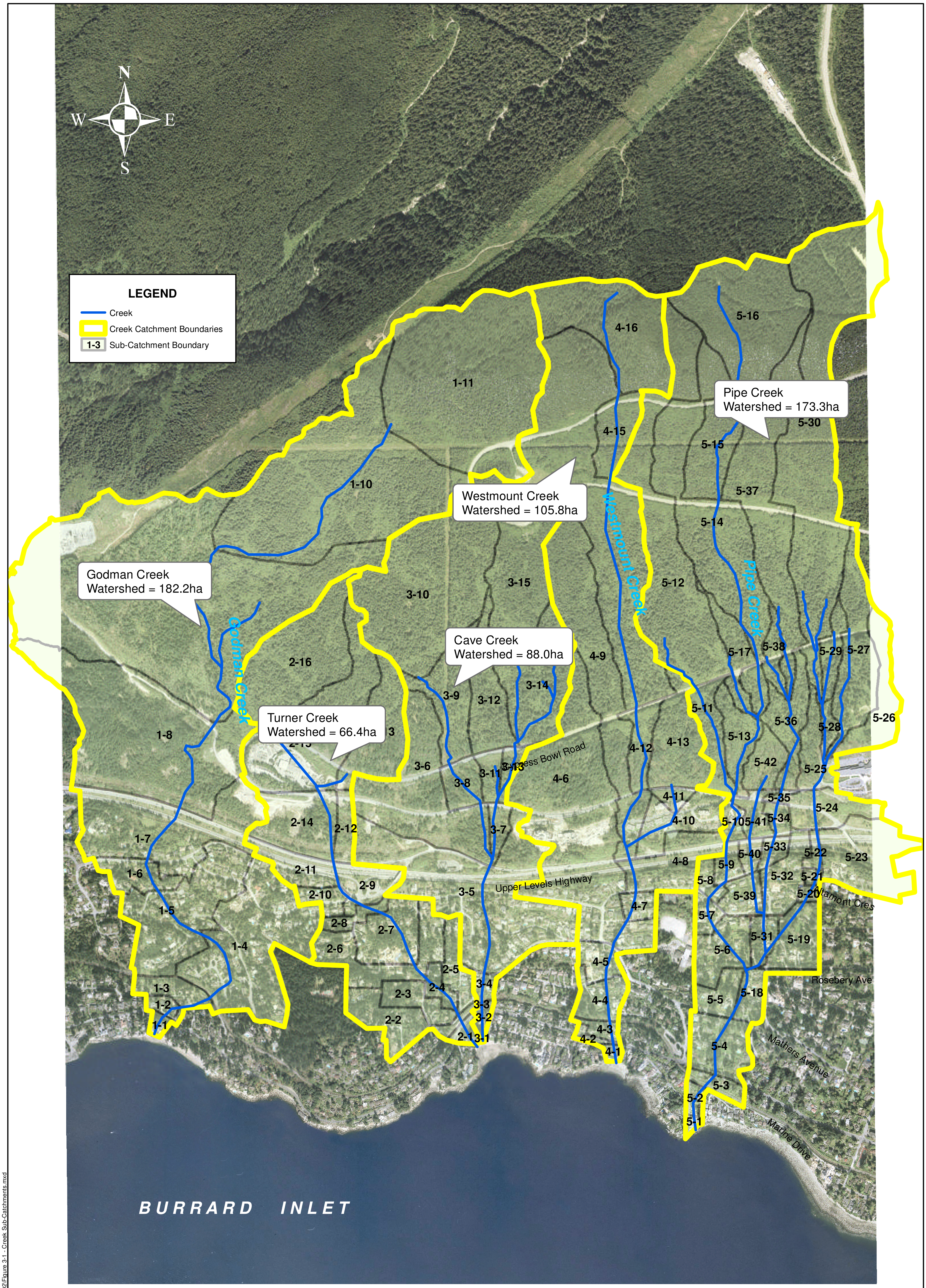
The drainage area and topography are fully described in drainage reports that were published by the City and others since 1973. Included are the District of West Vancouver Drainage survey (D&K, 1973), and the Hydro-Geotechnical Stream Assessment for the five creeks (Golder, 2009).

The study areas are broken up into sub-catchments that reflect distinct drainage characteristics for each of the five creeks. Each sub-catchment drains to one location within the sub-catchment. The existing sub-catchment boundaries were delineated from topographical contours and from input from InterCAD. The percent impervious area was interpolated from aerial photography.

Development in all five creeks is essentially built out below the Upper Levels Highway, with capacity for development available on lands above the Highway.

3.6.1 Godman Creek

The 182.2 ha of the Godman Creek watershed area is illustrated on Figure 3-1. The drainage area includes 11 sub-catchments, and drops in elevation from 796 m to sea level. The sub-catchments in the Godman Creek watershed are summarized in Table 3-2.



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GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
CREEK SUB-CATCHMENTS

FIGURE 3-1

Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 3-1 - Creek Sub-Catchments.mxd

TABLE 3-2
GODMAN CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
1-1	0.53	0-23 m	24.7	59
1-2	0.63	19-28 m	11.1	42
1-3	2.15	19-43 m	13.6	17
1-4	15.17	39-140 m	19.3	37
1-5	10.71	41-141 m	17.4	34
1-6	2.53	103-120 m	5.6	35
1-7	13.28	110-202 m	18.2	22
1-8	24.44	145-300 m	16.8	3
1-9	53.43	205-477 m	27.4	4
1-10	30.68	387-602 m	23.1	0
1-11	28.68	558-796 m	34.5	2

3.6.2 Turner Creek

The 66.4 ha of the Turner Creek watershed area is illustrated on Figure 3-1. The drainage area includes 16 sub-catchments, and drops in elevation from 469 m to sea level. The sub-catchments in the Turner Creek watershed are summarized in Table 3-3.

TABLE 3-3
TURNER CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
2-1	0.70	0-31	30.3	38
2-2	4.25	28-104	19.5	37
2-3	2.19	29-81	18.9	23
2-4	1.07	30-62	16.2	38
2-5	2.17	29-104	22.5	30
2-6	5.41	55-130	18.8	31
2-7	4.29	66-123	21.6	36
2-8	1.1	117-130	5.8	59
2-9	2.67	120-130	4.8	59
2-10	1.35	123-132	4.6	42
2-11	2.81	125-144	5.2	54
2-12	1.58	129-202	28.9	9
2-13	6.28	200-406	41.7	2
2-14	5.62	132-200	28.1	11
2-15	13.44	195-450	36.8	21
2-16	11.47	209-469	42.9	0

3.6.3 Cave Creek

The 88.0 ha of the Cave Creek watershed area is illustrated on Figure 3-1. The drainage area includes 15 sub-catchments, and drops in elevation from 625 m to sea level. The sub-catchments in the Cave Creek watershed are summarized in Table 3-4.

TABLE 3-4
CAVE CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
3-1	0.04	0-6	26.1	30
3-2	0.55	6-37	23.8	0
3-3	0.05	30-36	37.5	35
3-4	0.83	33-60	25.0	36
3-5	20.12	56-188	22.5	34
3-6	12.13	124-355	32.2	8
3-7	1.19	124-188	28.6	9
3-8	2.10	188-259	38.8	7
3-9	5.51	248-430	34.4	0
3-10	20.79	363-625	32.5	1
3-11	1.29	188-266	41.9	8
3-12	4.90	258-478	37.5	0
3-13	1.10	188-278	41.9	13
3-14	8.71	270-479	38.8	1
3-15	8.66	390-602	43.9	3

3.6.4 Westmount Creek

The 105.8 ha of the Westmount Creek watershed area is illustrated on Figure 3-1. The drainage area includes 16 sub-catchments, and drops in elevation from 793 m to sea level. The sub-catchments in the Westmount Creek watershed are summarized in Table 3-5.

TABLE 3-5
WESTMOUNT CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
4-1	0.37	0-17	27.0	35
4-2	0.41	17-28	7.2	62
4-3	0.78	18-31	15.3	30
4-4	2.41	29-65	18.5	14
4-5	1.72	54-82	18.4	48
4-6	23.45	76-392	29.6	24
4-7	1.13	93-110	17.8	24
4-8	4.89	107-167	25.8	20
4-9	13.46	132-593	38.0	3
4-10	3.96	132-188	22.2	36
4-11	0.84	162-188	28.1	29
4-12	2.45	188-305	37.7	7
4-13	5.91	188-305	27.1	7
4-14	28.91	295-773	34.6	4
4-15	3.42	584-654	21.8	6
4-16	11.73	646-793	27.4	0

3.6.5 Pipe Creek

The 173.3 ha of the Pipe Creek watershed area is illustrated on Figure 3-1. The drainage area includes 42 sub-catchments, and its area drops in elevation from 794 m to sea level. The sub-catchments in the Pipe Creek watershed are summarized in Table 3-6.

TABLE 3-6
PIPE CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
5-1	0.36	0-13	12.6	53
5-2	0.64	7-21	12.7	43
5-3	1.23	20-29	14.8	15
5-4	4.16	28-57	14.1	10
5-5	1.93	50-74	13.4	29
5-6	3.55	62-101	19.4	28
5-7	1.05	95-118	14.6	36
5-8	0.99	109-141	19.8	30
5-9	1.10	120-150	22.2	37
5-10	0.92	150-187	23.6	27
5-11	3.13	186-326	34.8	4
5-12	18	316-794	34.2	2
5-13	3.04	186-323	33.3	1
5-14	17.34	250-760	36.1	4
5-15	5.55	553-688	34.9	4
5-16	13.89	576-794	39.3	0
5-17	2.74	250-468	42.5	5
5-18	1.99	50-89	10.4	24
5-19	3.45	70-114	13.8	33
5-20	0.43	110-123	19.7	38
5-21	0.34	120-134	22.2	47
5-22	0.56	130-166	26.4	30
5-23	7.22	126-190	23.3	10
5-24	3.39	168-213	29.2	14

TABLE 3-6 (cont'd.)
PIPE CREEK SUB-CATCHMENTS

Model ID	Area (ha)	Elevation Range	Average Slope %	Percent Impervious %
5-25	2.74	204-312	28.7	11
5-26	4.2	226-392	26.4	2
5-27	7.08	228-521	35.8	3
5-28	0.83	225-285	29.6	8
5-29	1.88	278-410	35.8	5
5-30	27.01	282-783	28.3	3
5-31	1.45	70-102	17.1	26
5-32	3.13	102-169	21.5	40
5-33	0.90	129-157	23.3	42
5-34	0.88	157-186	25.4	24
5-35	0.27	184-202	32.7	38
5-36	3.86	196-345	30.2	1
5-37	14.01	295-755	32.4	3
5-38	1.51	295-414	34.0	6
5-39	1.84	100-129	19.3	40
5-40	0.84	129-157	22.8	45
5-41	1.55	155-194	23.1	25
5-42	2.27	190-253	26.6	0

3.7 Hydrology and Drainage

Background studies for drainage within the District study area date back to 1973 (D&K 1973). Golder & Associates Ltd. were retained to develop individual stream reconnaissance observations, which are summarized below (Golder, 2009).

3.7.1 Godman Creek

The upper watershed of Godman Creek is relatively open, and the ground is fairly impervious with bedrock either exposed or near the surface. The channel morphology is mainly bedrock controlled with multiple channels either underlain by smooth bedrock surfaces or descending via a series of bedrock cascades. Channel gradients range from 20% to 45%, while adjacent native slopes have similar gradients and no apparent stability concerns. Godman Creek also has a western tributary, which has a relatively low channel gradient (5% to 15%) before extending into a small wetland area. The tributary then joins Godman Creek further downstream in the meandering reach of the main channel.

On the south side of the Upper Levels Highway, Godman Creek crosses into a park area, which remains natural, with bedrock exposed in many places. From there it proceeds downstream through a number of residential properties before discharging to Burrard Inlet. At the low-gradient stream reaches, the channel is either heavily aggraded with an associated decrease in streambank height, or displays significant bank erosion and undermining of locally higher sidewall slopes composed of till. These areas are detailed in the Golder report.

Godman Creek has a relatively low physiological runoff potential compared to the other watersheds; the calculated drainage density of Godman Creek is 2.1 km/km^2. It is typical of a creek with a relatively shallow profile, compared to Westmount, Cave and Pipe Creeks; however, model results imply higher flows than anticipated suggesting the steep slope and aspect ratio of the basin concentrates runoff quickly.

3.7.2 Turner Creek

Above the Upper Levels Highway, Turner Creek is relatively open, and the ground is fairly impervious with bedrock either exposed or near the surface. The channel and sidewall slopes are predominantly bedrock and large woody debris has locally created a number of cascades within the channel. Channel gradients range from 10% to 30%, while adjacent native slopes range from 15% to 50% with no apparent stability concerns.

South of the Upper Levels Highway, Turner Creek flows in either a natural or a concrete-lined channel through a number of private properties before its discharge into the Burrard Inlet. Below Westmount Road both the creek channel and facilities are generally deficient in terms of a major flood. Small culverts at private driveways are restricted, however overflow is usually back into the channel, and damage is mainly limited to adjacent private properties. These areas are detailed in the Golder Report. A detention pond serves to attenuate peak flows and act as a sedimentation basin.

Turner Creek has a relatively low runoff potential; the calculated drainage density is 1.7 km/km^2. It is more typical of a creek with a relatively shallow profile compared to Westmount, Cave and Pipe Creeks.

3.7.3 Cave Creek

On the north side of the Upper Levels Highway, Cave Creek is slightly steeper than Godman and Turner Creeks, while the ground is fairly impervious with bedrock either exposed or near the surface. The channels are poorly defined, with old-logging roads and trails that can divert flows during large floods. Channel gradients range from 20% to 45%, while adjacent native slopes have similar gradients with no apparent stability concerns. On the main creek, approximately midway between the headwaters and the first crossing of Cypress Bowl Road are two small zones of streambank instability. The first is a 7 m wide zone of sliding and/or slumping on the left bank. The landslide track is about 8 m long with a slope of 50%. The second zone of instability occurs about 16 m further downstream and is associated with wind throw of a large tree located on bedrock at the crest of the right bank. Between the first crossing of Cypress Bowl Road and the Upper Levels Highway, the streambed and sidewall slopes are predominantly composed of bedrock overlain by a colluvial veneer. Channel slopes range from 10% to 60%, while adjacent native slopes have no apparent stability concerns.

On the south side of the Upper Levels Highway, Cave Creek crosses into residential properties, and its creek bed is largely gravelly with many sections under the influence of erosion. Bedrock is exposed in many places, and some bank protection and energy dissipation improvements have been added. However, the stream flows within a natural channel in some parts with local erosion/undermining of the 1 m high till banks. These areas are detailed in the Golder report. Aside from these channels, the stream flows within either a naturally armoured channel, a bedrock or retaining wall-bound channel, or a buried pipe and exhibit no visible bank erosion.

Cave Creek has a relatively moderate runoff potential; the calculated drainage density is 2.8 km/km^2. This watershed is more typical of a creek with a moderately steep profile.

3.7.4 Westmount Creek

The upper watershed of Westmount Creek is steeper when compared to Godman and Turner Creeks, while the ground is fairly impervious with bedrock either exposed or near the surface. The channel is poorly defined at the higher elevations and consists mainly of sand and small woody debris. Channel gradients range from 30% to 35%, while adjacent native slopes are about 20% to 30% with no apparent stability concerns. At lower elevations, the channel is largely bedrock-controlled with a bedrock bed for most of its reach. Till is exposed along the immediate streambanks. Channel gradients range from 20% to 70%, while adjacent native slopes are 25% to 60% with no apparent stability concerns.

Near the Upper Levels Highway, the stream flows between areas of existing residential development. Bedrock is exposed in the streambed, and channel gradients range from about 15% to 30%, while adjacent slopes are about 45% and display no evidence of instability.

At elevations below the Upper Levels Highway, Westmount creek crosses through a number of residential properties before discharging into the Burrard Inlet. The stream flows within channels bounded by bedrock or by concrete walls and show no apparent erosion concerns. There is potential for overtopping where channel banks are low, and there is evidence of scouring at the base of a hand constructed rock streambank retaining wall. These areas are detailed in the Golder report.

Westmount Creek illustrates a relatively high rate of runoff potential; the calculated drainage density is 4.7 km/km^2. Modelling suggests that relatively high flows predicted by a high drainage density are, however not realized. This watershed is typical of a creek on a relatively steep terrain.

3.7.5 Pipe Creek

The upper watershed of Pipe Creek is similar in steepness to Westmount Creek, and is characterized by three distinct zones. Pipe Creek has multiple tributaries from multiple branch extensions, which are not covered in detail. However, these tributaries all experience similar topographical features at different elevations. At higher elevations, the stream flows in channels with colluvial substrate and a channel gradient of about 20% to 30%. Dense tills are exposed along the streambank, and sidewall slopes are not well-developed. Adjacent native slopes range from about 15% to 30% with no visible evidence of instability. At the lower elevations, the channel becomes largely bedrock-controlled with a bedrock bed. Surficial materials consist of till and locally, sandy to bouldery colluvium. Channel gradients range from 25% to 50%, while adjacent native slopes are about 7% to 50% with no apparent stability concerns. The lower to mid elevation topography gradually takes on a more ravine-like morphology. Thicker till is visible in the ravine reach, however, ravine sidewall slopes show no evidence of instability. Near the Upper Levels Highway, residential development exists above both sides of the stream. The average gradient for the stream in this area is 25%. There is a section of the left bank sidewall, which has been previously covered with shotcrete in an apparent attempt to mitigate erosion.

On the south side of the Upper Levels Highway, Pipe Creek crosses through a number of residential properties before discharging into the Burrard Inlet. The stream flows within channels bounded by bedrock or by concrete wall; however, there are a few areas where erosion and undermining of the streambanks have occurred. These areas are detailed in the Golder report.

Pipe Creek is anticipated to have a relatively high runoff potential; the calculated drainage density is 4.6 km/km^2. This watershed is typical of a creek on a relatively steep terrain.

3.8 Natural Resources

The District of West Vancouver under the Local Government Act has responsibility for drainage, and is ultimately responsible for undertaking flood protection measures within the study area. District, Provincial and Federal regulating agencies all work to protect the environment for preservation of the natural habitat. One of the objectives of the District's community planning process is to identify and protect areas of high environmental sensitivity.

The primary and current drainage concerns within the study area are:

a) overtopping of banks and flooding of local streets and existing properties due to inadequate drainage;

b) limited channel capacity and erosion especially in existing developments below the Upper Levels Highway;
c) water quality issues related to fisheries and construction;
d) depletion of riparian habitat due to existing and new development.

New development is expected to increase peak flows and diminish base flows, unless proper mitigating management measures are carried out.

Section 3.10 addresses the watershed health of the study area and discusses the ecological overview conducted by SLR Consulting (Canada) Ltd. SLR's investigation includes water quality sampling and analysis and assessment of riparian habitat and other natural resources.

3.9 Flow Monitoring and Rainfall Events During Monitoring Period

This section provides our review of the rainfall and flow monitoring data used in the ISMP. The purpose of this section is to identify key hydrological events that occurred during the flow monitoring period and to establish an acceptable methodology for review of those events as they relate to the subsequent calibration of the hydraulic model.

3.9.1 Background

As part of the ISMP process, stream flow monitoring stations were set up by Northwest Hydraulic Consultants (NHC) on Pipe, Cave and Godman Creeks. Two stations on each of these creeks were installed: 1) at the lower end of the watershed near tidewater, and 2) in the upper reaches of the creeks above the Upper Levels Highway. Flow data was collected every 10 minutes from March 2008 to April 2010. A copy of NHC's report, including hydrographs and rating curves for each monitoring station is included in Appendix Q. A map showing the flow monitoring stations is included in Appendix R as InterCad's Figure C.1 (reproduced with permission).

Rainfall data is available for a number of gauges in the vicinity of the ISMP study area. The various gauges are shown in InterCAD's Figure C.1. The gauges are summarized in Table 3-7.

TABLE 3-7
RAINFALL MONITORING STATIONS

Station	Location	Elevation	Operated by
Madrona Reservoir	Horseshoe Bay	90 m	Metro Vancouver
DWV Works Yard	Cypress Bowl Road, near Upper Levels Highway	200 m	District of West Vancouver
AES WA2	Cypress Bowl Road at Upper Levels Highway	178 m	Atmospheric Environmental Services
Cypress Ranger Station	Cypress Bowl Road, near Cypress Mountain Ski Area	930 m	District of West Vancouver
VW14	District of West Vancouver Municipal Hall	41 m	Metro Vancouver
VW51	Capilano Golf and Country Club	201 m	Metro Vancouver

The DWV Works Yard gauge is located directly within the ISMP study area. It lies in the upper reaches of the Turner Creek watershed. Rainfall data is available from this gauge in 5 minute increments for the entire flow monitoring period (March 2008 to April 2010). The AES WA2 gauge is also located within the study area, but only has hourly rainfall data available.

3.9.2 Review Of Key Events

Review of NHC's flow monitoring data yields three major stream flow events: December 21, 2009; January 15, 2009; and January 7, 2009. These events are the three largest stream flows recorded in the two year period. They are the largest stream flow events by total volume of flow and peak instantaneous flow.

Review of the DWV Works Yard rainfall data yields two additional significant rainfall events: September 6, 2009; and November 18, 2009. Along with the three major flow events noted above these events experience the most significant rainfall during the flow monitoring period. However, these two rainfall events do not result in significant stream flows. Table 3-8 summarizes the recorded stream flows for the five major events.

TABLE 3-8
KEY EVENTS FROM MARCH 2008 TO APRIL 2010

Event	Peak Recorded Stream Flow		
	Lower Pipe Creek (L/s)	Lower Cave Creek (L/s)	Lower Godman Creek (L/s)
Dec. 21, 2009	2,849	1,448	4,017
Jan. 15, 2010	2,166	1,165	2,719
Jan. 07, 2009	2,138	1,280	1,805
Sept. 6, 2009	416	451	277
Nov. 18, 2009	1,191	781	1,369

This section provides a review of the above five events and their potential use in the calibration of the hydraulic model.

Rainfall events with greater temporal and spatial consistency provide a more direct correlation between recorded rainfall and recorded stream flow. Because the calibration process takes a recorded rainfall event at a specific location and time and applies it over the entire catchment area to produce a simulated stream flow, events with more stationary weather patterns will produce a simulated stream flow that better matches the recorded flow. It is difficult to simulate a rainfall event which fluctuates widely across the catchment area. During any given event, it is only possible to record the rainfall at one location (DWV Works Yard) within the study area.

To illustrate the spatial and temporal variation of the rainfall events, Figures R-1 to R-5 plot in Appendix R, the recorded rainfall at the various rainfall stations for each of the five major events. Also included in the figures are tables showing the peak intensity, duration, time of peak and total volume measured at each gauge.

For the three key flow events, December 21, 2009 appears to be a much more transient weather pattern than the other two storms. This event experienced fluctuating rainfall intensities recorded at the various rain gauges in West Vancouver. The January 15, 2010 and January 7, 2009 events appear to have much more consistent rainfall intensities across the surrounding area. It is clear from Figure R-1 that the December 21, 2009 event had widely varying rainfall intensities both over time and at the different locations. This spatial and temporal variance in rainfall may be the reason why the initial calibration in the latest version of the ISMP resulted in poorly matching total runoff volumes. The January 15, 2010 and January 7, 2009 are likely better candidates for the initial calibration of the model due to their more consistent nature. Of these two events, January 15, 2010 appears more consistent as the January 7, 2009 event experiences no recorded rainfall at the District Municipal Hall, which is located to the east of the study area.

The two rainfall events which do not register significant stream flows are shown in Figures R-4 and R-5. From Figure R-4 it is clear that the September 6, 2009 event recorded at the Works Yard is isolated to that location. The November 18, 2009 event shown in Figure R-5 is more consistent, but still varies between the locations and actually measures no precipitation at the Cypress Ranger Station. Because these two events appear to be more isolated to a specific location and do not result in significant stream flows we do not believe they are suitable for use in calibration of the hydraulic model.

3.9.3 Elevation/Intensity Scaling Factors

For the calibration of the model, no elevation/intensity scaling factors were applied to the recorded rainfall, prior to inputting into the model. After reviewing the rainfall data at the various locations, this appears to be a logical approach for the calibration process, depending on the event selected for calibration.

During the January 7, 2009 event both total volume of rainfall and peak rainfall intensity appear to increase with elevation. From the table on Figure R-3, the peak rainfall intensity varies from 0 mm/hr at the District hall (el. 41m) to 6 mm/hr at the Works Yard (el. 200 m) to 12 mm/hr at the Cypress Ranger Station (el. 930m). The total rainfall volumes vary similarly. It should be noted that this variance could also just be a spatial phenomenon and not be a result of increase in elevation.

From the table on Figure R-2 it is evident that during the January 15, 2010 event there is generally no direct or significant increase in rainfall intensities or volumes at the gauges of higher elevation. Therefore, no elevation factors should be applied to the rainfall data when using this event for calibration.

3.9.4 Snowmelt and Snowfall

To accurately simulate a recorded runoff event it is necessary to consider the effects of snowfall and snowmelt. Hourly temperatures and recorded daily snow depths are available from Atmospheric Environmental Services at the West Vancouver WA2 gauge, located within the study area on Cypress Bowl Road at the interchange with the Upper Levels Highway. The actual depth of snow on the ground is recorded each day in the early morning.

Figures R-6 and R-7 show the recorded hourly temperatures and snow depths during and prior to the January 7, 2009 and January 15, 2010 events. The figures show a period of two weeks leading up to the events. The data shown suggests that for both events the temperatures during the event are high enough such that precipitation is occurring as rain, not snow. However, the data also shows that at the onset of the January 7, 2009 event there is up to 15 cm of snow already on the ground. The recorded snow depths decrease as the storm progresses suggesting the occurrence of snowmelt. At the onset of the January 15, 2010 event, there is no recorded snow on the ground. The temperatures for the period leading up to this event also indicate that it is unlikely any snow on the ground was present even at the highest elevations in the study area. Due to the likely absence of complex snowfall and snowmelt effects on runoff in the January 15, 2010 event, it is likely a better candidate for the calibration of the hydraulic model.

3.9.5 Flow Data Verification

In order to assess the accuracy of the flow monitoring data, we have selected the January 15, 2010 event to compare recorded rainfall volumes with resulting stream flows. Because this event experiences the most consistent rainfall across the area, it is best suited to compare the recorded rainfall with the recorded stream flow. Figure R-8 shows the recorded rainfall at the Work's Yard compared directly to the resulting stream flows at the six monitoring stations. Table 3-9 below compares the rainfall and flow peaks, durations, time of peaks and total volumes. The total volume of flow is expressed in millimetres over the given catchment area to allow for a direct comparison to total volume of rainfall.

TABLE 3-9
JANUARY 15, 2010 RAINFALL VS RUNOFF

Location	Peak	Duration	Time of Peak	Total Volume
Rainfall	**mm/hr**	**hrs**	**Date-time**	**mm**
Works Yard	12	26.6	Jan 14 22:50	101
Runoff	**L/s**	**hrs**	**Date-time**	**mm**
Lower Pipe Creek (171ha)	2166	54	Jan 15 9:00	65
Lower Cave Creek (88ha)	1165	50	Jan 15 10:20	54
Lower Godman Creek (175 ha)	2719	54	Jan 15 11:00	57

The total volume of runoff to total volume of rainfall varies from 53% in the Cave Creek catchment, to 55% in the Godman Creek Catchment and up to 65% in the Pipe Creek catchment. The total volumes reported above account for the base flows recorded in the creeks at the onset of the storm (see hydrographs on Figure R-8) – i.e. the flow volumes given do not include those already in the creek.

This result indicates a loss of volume to interception and/or infiltration for this particular rainfall event. The Works Yard rainfall data indicates there were no large precipitation events in the three days prior to this event, suggesting capacity for initial capture of the rainfall. However, small amounts of rainfall are recorded leading up to the event and, evidently, enough to provide baseflow to the creeks as they experience small stream flows at the onset. Calibration to this event would need to take this observation into account.

The peak flows recorded at the three lower monitoring stations during the January 15, 2010 event all fall at the upper end of their respective rating curves - see rating curves for the lower monitoring stations in the enclosed NHC report (Appendix Q). The rating curves developed by NHC for each of the flow monitoring stations include between 6 to 8 verification points.

3.9.6 Conclusion on Rainfall Events During Monitoring Period

Based upon the preceding review of the five significant rainfall events during the ISMP flow monitoring period, we have concluded that the January 15, 2010 event will likely produce the most accurate calibration of the hydraulic model. Due to the absence of complex snow fall and snowmelt effects and a more spatially and temporally consistent rainfall distribution, the hydraulic model is better suited to simulate the watershed during this event. The review of the rainfall and flow monitoring data also highlights the complex nature of the rainfall events and how they relate to the resulting stream flows.

The January 15, 2010 event is selected for the calibration of the hydraulic model. The Works Yard rainfall data is used in the calibration process as it is the only gauge situated within the study area. The calibration to this event does not apply elevation factors to the recorded rainfall because of the preceding observations. For each of the gauged creeks the calibration follows a two-step process: first calibrate the upper catchments using the upper gauges; then, the remaining lower catchments to the lower gauges.

The December 21, 2009 event is not considered for the initial calibration of the model due to its spatial variance across the surrounding area and temporal variance throughout the storm duration. This event is a likely candidate for use as verification of the model, once the initial calibration is complete.

The January 7, 2009 event is not considered appropriate for calibration due to the apparent presence of snowmelt and its complex effects on runoff.

The September 6, 2009 event is not considered appropriate for calibration because it appears to be an isolated rainfall at the Works yard gauge with minimal rainfall recorded at all the other gauges.

The November 18, 2009 event is not considered for the initial calibration of the hydraulic model because rainfall is not observed in the upper regions of the study area (suggested by the readings at Cypress Ranger Station). This is a potential candidate for use as verification of the model, after initial calibration; however, the modelled flow volumes will likely not correlate well with the recorded volumes as the rainfall event does not appear consistent across the area.

Further flow monitoring of the creeks should be conducted in the future in order to capture additional events suitable for additional calibration of the hydraulic model. The events discussed herein represent the only suitable data available at this time.

3.10 Watershed Health Assessment

An ecological overview report on the five watersheds was completed by SLR Consulting Ltd. The final draft dated March 2009 is included in Appendix B. SLR's report forms the ecological investigation portion of the ISMP and follows the ecological component of Metro Vancouver's ISMP template.

The objectives of SLR's report are as follows:

- provide ecological information;
- ensure valued ecosystem components are accounted for during development within the watersheds; and
- ensure ecologically relevant information is available for continued monitoring of watershed health.

Key aspects of the SLR report are summarized in this section.

3.10.1 Streams and Riparian Habitat

Much of the information on streams and riparian habitat was derived from recent SLR reports on Rodgers Creek and Cypress Creek Neighbourhoods. New assessments of Turner Creek were also undertaken. Sections of all five of the ISMP creeks are known to support populations of salmonid fish. Resident cutthroat trout have been reported in sections of Godman Creek above Highway 1.

SLR's report includes a Riparian Areas Assessment which establishes minimum setbacks along the undeveloped creek sections within the study area. The assessment was based on the methodology of the B.C. Riparian Areas Regulation (RAR), a methodology which is recognized as being ecologically relevant and scientifically sound in the protection of fish habitat. The RAR methodology intends to help developers satisfy the requirements of the federal *Fisheries Act* requirement of "No-Net-loss" of fish habitat.

According to SLR's assessment, the minimum width of the riparian areas ranges from 10 to 17 meters. The report also includes a riparian corridor assessment which establishes a measure of the Riparian Forest Integrity (RFI). The RFI is an indicator of the degree to which a stream is enclosed in culverts and has forested riparian setbacks of at least 30 meters. The RFI values of the five creeks above Highway 1 range from 71% (Turner Creek) to 92% (Cave Creek). These values compare to an RFI of 0% for the portions of the five creeks below Highway 1. (A stream with intact 30-metre treed riparian zones along both sides of its entire length would have an RFI of 100%.)

3.10.2 Water Quality Monitoring

SLR collected in situ water quality measurements at two locations along each of the five creeks. Results were considered typical of fast-flowing mountain streams of BC coastal areas. See SLR full report in Appendix B for detailed water quality sampling results.

The British Columbia Ministry of Environment and the Canadian Council of Ministers of the Environment water quality guidelines were referred to for this section.

Un-expectantly high faecal coliform levels were discovered in Godman Creek at the sampling site upstream of Westridge Avenue. District Operations staff investigated the coliform levels and determined the likely source to be dog faeces from the adjacent Westridge Park. Off-leash dogs were observed in the immediate vicinity of the creek, which is crossed numerous times by dog trails in the park. The nearest District sanitary sewer is downstream from the sampling site on Westridge Avenue.

3.10.3 Benthic Invertebrate Community Investigations, Godman Creek

During the ISMP development it was decided to choose one representative site within the five watersheds for a Benthic Invertebrate Community Investigation. The site chosen for sampling was a 52 meter reach below Highway 1 between Westridge Avenue and Viewmount Place. Population densities and the composition of benthic communities is considered a useful indicator of watershed health. The sampling can be used as a base case scenario to monitor land development effects on stream health.

The Benthic Invertebrate sampling and analyses were conducted in accordance with both the Module 4 Stream Invertebrate Survey developed by the DFO for Streamkeeper organizations and the Benthic Index of Biological Integrity (B-IBI) as per Metro Vancouver’s ISMP template (GVRD 2005). The Module 4 Stream Invertebrate Survey resulted in an Acceptable Site Assessment Rating. The B-IBI rating was a Good Stream Condition of 38. Detailed discussion and results of the investigation are included in Appendix B.

3.10.4 Terrestrial Ecosystem and Vegetation Characteristics

Vegetation surveys and ecosystem mapping were completed to characterize forests of the proposed development area. Detailed results are included in Appendix B. The development area consists mostly of second growth trees, a result of re-growth after clear-cutting in the early 20th century. There are no observed old growth forests within the ISMP study area. SLR’s report concludes that, within the study area, there are no known rare element occurrences of vascular plants or ecological communities and sensitive ecosystems are mainly limited to riparian areas, wetlands and rock outcrops.

3.10.5 Wildlife of the ISMP Study Area

Through ground reconnaissance and previous studies SLR investigated wildlife occurrence in the study area. Appendix B provides a list of vertebrate wildlife species that could potentially occur in the study area. Also provided is the potential occurrence in the study area of Red-Listed and Blue-Listed terrestrial and amphibious vertebrate species. Potential for species occurrence is generally derived from observed habitat availability. There are five listed bird species and five listed mammal species that have the potential for occurrence within the study area but whose presence has not been confirmed. Two listed species of frog, the coastal tailed frog and the red-legged frog have been confirmed present. Occurrence probability of dragonflies and butterflies was also assessed. The area provides only low-quality habitat for listed dragonfly and butterfly species.

3.10.6 Watershed Health

Metro Vancouver’s ISMP template recommends the following three quantifiable biophysical characteristics for assessing watershed health:

- Percent Riparian Forest Integrity (RFI);
- Effective Impervious Area (EIA); and
- Benthic Index of Biotic Integrity (B-IBI)

All three of the above characteristics for the Godman Creek watershed were analyzed and discussed in SLR's report giving a baseline measurement of watershed health for the study area. The actual B-IBI score of 38 at Godman Creek exceeds the predicted B-IBI score of 34 indicating that there are no concerns related to the baseline health of Godman Creek Watershed (SLR, 2009). The watershed health assessments made in the SLR report can be used as a base measurement for assessing future development effects on watershed health and the effectiveness of low-impact development (LID) practices in limiting those effects. Effectiveness of LID measures would result in a rightward movement of the EIA-RFI point on the watershed health assessment graph provided in SLR's report.

4.0 MODEL DEVELOPMENT AND CALIBRATION

This section establishes the basis for the Pipe, Westmount, Cave, Turner and Godman watersheds ISMP through a brief description of the runoff processes, selection of the preferred improvements, rainfall-runoff criteria, environmental criteria and presentation of costing criteria. Also included in this chapter is a review of drainage regulations. The purpose of this chapter is to introduce the parameters that are appropriate for the analyses that are undertaken in the following sections.

The first and last parts of this section are general and deal with the fundamentals of the hydrologic process, and provide general discussion on Stormwater Management and Regulation. For the technically trained reader, these parts can be omitted. Central parts identify technical criteria pertinent to the study.

4.1 Runoff Process

Understanding of the runoff process is essential in meeting rainfall management objectives in the drainage areas. In this study, the management objectives primarily include flood risk and erosion processes although water quality and protection of the biological health of streams is implicit in the measures to be undertaken.

Frequent storms or annual rainfall events are considered steady or semi-continuous events that, because of the greater frequency of storms can cause the most significant erosive problems in an unprotected drainage area. The solution to the erosive properties of frequently occurring low intensity storms are undertaken through the limitation of allowable flows in the creeks. The approach is to limit flows in the creeks up to half of the 2-year rainfall event (50 % of the Mean Annual Rainfall) flow rates. On site infiltration can be used in some cases to reduce the runoff from high-frequency events. A detention or diversion solution will normally be required to protect life and property from runoff that results from large, low-frequency events.

While runoff criteria are typically determined for frequent, low intensity rainfalls, they are also extended to the infrequent 10 year and 200 year high intensity 24-hour events for channel and overbank protection. Where storage is required, the detention facility will be designed to protect the geological conditions of the area and should be designed to store the 200 year storm event.

For the 10 and 200 year events, the runoff process is a combination of both hydrologic and hydraulic processes.

4.1.1 Runoff – Hydrologic Process

The primary aims of urban hydrology are twofold:

- First it is to predict the hydrologic loads, both stormwater volume and peak runoff rates, under existing and future land use conditions.
- Second it is to predict the base flow volumes and to ascertain means for changes of volumes due to land use change.

The context to undertaking these predictions varies with the constraints imposed on the predictive method; constraints are both physical and social.

1. Physical Constraints For the physical constraints, the infiltration and runoff process starts with rainfall. Rainfall type, aerial distribution, intensity and pattern affect the runoff process. Only after the rainwater has sufficiently wetted the surface, filled depressions and soaked into pervious ground materials will additional rainfall become runoff. These processes are not well defined nor are they well understood. They are termed initial abstractions and in

the cases that were just mentioned are interception, depression storage and infiltration. Antecedent rainfall and ground moisture conditions, soil and cover type, and percentage of pervious or impervious areas, which contribute to drainage collectors, affect the amount and rate of runoff and the infiltration that becomes base flow. The watershed surficial character further impacts the base flow distribution.

The above becomes further complicated where interflow conditions exist that allow groundwater to discharge and become part of the surface water flow and return to groundwater flow. Interflow is often seasonal and unpredictable.

The change of surface from pervious to impervious speeds the runoff rate and increases the runoff volume because of a reduction in rainfall losses from surface wetting, depression storage and soil infiltration. The change may increase sinkhole formation and also increase pollutant loads. Urban development often produces changes by the construction of impervious surfaces such as roofs, streets, sidewalks and parking lots. Drainage collection areas not covered in this manner are usually landscaped. Because the landscaped areas are often covered with turf and lower density vegetation and are often treated with chemicals, this may also increase runoff and pollutant loads that impact water quality, habitat, flood risk and erosion processes.

2. Social Constraints In assessing social constraints, hydrologists consider environmental goals and risk acceptance to set limits for their analyses. Stream protection measures, erosion (stream power minimization, deposition), continuous evaluation of low flow impacts, flood loss, property loss of high flow impacts and habitat losses are among the issues that need to be integrated into the analysis.

These conditions form the hydrologic components to the runoff and infiltration process.

4.1.2 Runoff – Hydraulic Process

Once the overland runoff collects into channels or drainage pipes, it increases to a peak or to several peaks during and after the storm. The water is stored and released from numerous sections of natural or manmade channels and structures, which affect the time-distribution of the runoff hydrograph. Improved or increased hydraulic capacity in the urban drainage system can significantly alter the runoff process. When natural channels are deepened, lined and straightened and when storm sewers are installed, the result reduces watershed storage time (hold up) and increases the peak rate of runoff. Manmade structures can be provided to offset or mimic natural detention effects.

Also flow in natural bedded channels must be examined to minimize stream power and control erosion and deposition processes. Natural river meander design is often required to achieve this objective.

Velocity in watercourses, under normal circumstances, should not exceed critical velocity (i.e., should remain sub-critical) unless control structures are provided. Frequently occurring velocities, within a watercourse, can be compared to Maximum Permissible Velocities (MPV's) in order to assess their susceptibility to erosion. Allowable velocities for incipient scour in different creek bed forms are as follows (Chow, 1959):

Lining Materials	Maximum Permissible Velocity (MPV) m/s
Fine sand	0.45
Sandy loam	0.5
Silt loam	0.6
Ordinary firm loam	0.75
Stiff clay	1.1
Shales and hardpans	1.8
Fine gravel	0.75
Graded silts and cobbles	1.2
Coarse gravel	1.2
Cobbles	1.5
Rip-rapped natural channels	1.5
Till	1.8

Lining Materials	Maximum Permissible Velocity (MPV) m/s
Bedrock	3.0
CMP channels, sewers & culverts (concrete and asphalt)	7.3

Based on field observations and the Golder (2009) and Aqua-Tex (2012) reports, the MPV's for each channel reach are estimated in Appendix J, Table J-3.

4.1.3 Runoff – Management and Design Method

According to Rantz (1971) as early as the 1960's it was recognized that the construction of storm sewers, without storage detention, increases drainage peaks from 1 to 4 times for 2-year recurrence rainfalls, to 3 times for 10-year recurrence intervals, to 2.75 times for 25-year, and 2.50 times for 100-year recurrence intervals. Later, Cook (1986) showed similar effects for a small controlled drainage watershed in Ontario. More recently, researchers such as Scheckenberger and Guther (1997) have shown how increased drainage peaks contribute to unstable, eroding streambeds. Roesner et.al. (2001) looked at BMP solutions over the last 10 years and found that mitigation has been focusing on major events. However, the minor events were shown to be at least as important due to erosion problems that result from the higher frequency of smaller storms. Because of the increased runoff frequency and peak flows brought about by urban development, an attempt must be made to adopt criteria for handling or reducing these potentially dangerous and increased flows.

Standard drainage design incorporates a minor and major system for urban development. The minor system is normally designed to handle storm flows from 2-year to 10-year (and as high as 25-year in commercial high value zones) rainfall recurrence intervals, and the major system is designed to handle excess flows for the 25-year to 200-year recurrence intervals. The minor system normally handles local drainage from developed areas and remains separate from the major system. The major system provides the higher flood protection level, along streets, in major natural channels, in special floodways and through large storm sewers. Sometimes an overland route is not feasible for the major system and it must be combined with the minor system in a pipeline, particularly in areas of existing development, which were not laid out with the two system concept in mind.

Erosion protection, bank stability, and provisions for sediment transport or reduction and stream pollution also become important when a design method is selected.

The minor-major system, erosion-sediment control and pollution are management responsibilities as well as design responsibilities because management objectives and criteria must be set out for protecting major flood routes for erosion-sediment reduction and for minimizing the pollution of watercourses.

4.2 Rainfall

Rainfall drives the runoff and infiltration process. Rainfall considerations in calculating runoff peak flows include the affects of impervious area, rainfall intensities, and the distribution of the rainfall over a given duration (the rainfall pattern).

4.2.1 Rainfall Gauges

Three climate stations were used to obtain historical climate records as well as precipitation data for the period from July 2005 to April 2008 for the study area. The climate stations are summarized in Table 4-1.

The following three active stations were referenced for continuous precipitation data:

TABLE 4-1
CLIMATE STATIONS

Station Name	Station Operator[1]	Station Elevation
West Vancouver Municipal Hall (located approximately 3 kilometres to the east of the Pipe Creek Watershed); (VW14)	GVRD	41 m
Capilano Golf and Country Club (located approximately 5 kilometres to the east of the Pipe Creek Watershed) (VW51)	GVRD	200 m
Cypress Mountain Ranger Station (located north of Hollyburn Ridge, off Cypress bowl road, at an elevation of 930 m)	DWV	930 m
Note: 1 GVRD: Greater Vancouver Regional District, DWV: District of West Vancouver		

Long term climate records relevant to the study area were available from the West Vancouver Municipal Hall Station located 3 kilometres east of the Pipe Creek watershed and south of the Upper Levels Highway. Monthly and annual averages were available for the data collected. In an average year, the area receives 1,822 mm of precipitation, with the highest monthly average occurring in January (372 mm), and the lowest in August (13 mm). A typical year includes a dry season from April to September and a wet season from October to March. More precipitation occurs at higher elevations in the watersheds, as shown by an annual average of 2,635 mm of precipitation at the Cypress Mountain Ranger Station.

All these gauges will remain in operation so that further data will be available in the future, allowing the model to be used for investigation of future events after the construction of new developments above the highway.

4.2.2 Precipitation – Elevation Relationship

In recognition of the orographic effects of the North Shore mountains (increase in precipitation with elevation), a relationship was developed to develop design storms for the upper watershed areas. A relationship among all three stations was developed by comparing concurrent monthly precipitation totals.

The higher elevation Capilano Golf & Country Club station received approximately 1.4 times the precipitation compared to the West Vancouver Municipal Hall station based on the comparison of their monthly precipitation totals. For the upper watershed Cypress Mountain Ranger station, a factor of 2.0 was developed for factoring the West Vancouver Municipal Hall station data in developing design storms for the station. These factors were then plotted on an Elevation-Intensity curve shown in Figure 4-1, which illustrates a relationship between the elevations of the stations against their respective intensity factors. This curve was used to construct a unique design storm for each sub-catchment at differing elevations during the model construction process. The same rainfall distribution was used for each sub-catchment, but the intensity varied based on its elevation.

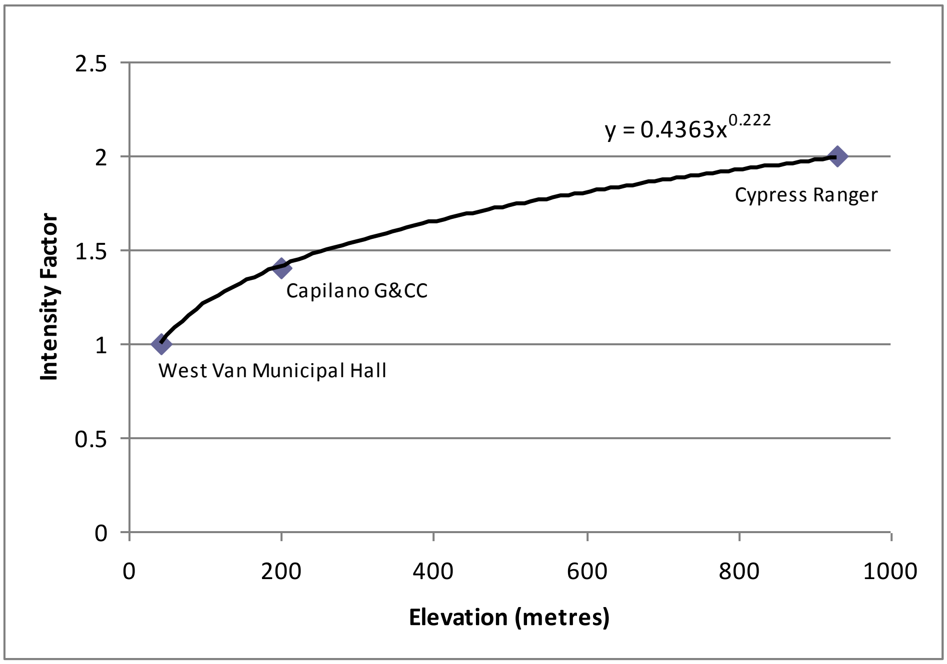


Figure 4-1 Elevation-Intensity Curve for Precipitation Stations

4.2.3 Area Effects

For drainage watersheds of 2590 ha (10 mi^2) or greater, the percentage reduction in average rainfall intensities, which have recurrence intervals of 2 years to 100 years and durations of ½ hour to 24 hours are proportional to the size of the watershed. Because the drainage area in this study is less than 2590 ha (10 mi^2), a reduction factor was not included.

4.2.4 Design Storms

Design storms were developed based on Atmospheric Environmental Services (AES) statistical distributions for the British Columbia Coast using historical summaries from the West Vancouver Municipal Hall Climate Station. The Municipal Hall station has the longest period of record of all climate stations near the watershed. Design storms were developed for the 1-hour, 2-hour, 6-hour, 12-hour, and 24-hour durations.

The AES design criteria for a 1-hour Storm Rain Distribution along the British Columbia Coast was used as the rainfall pattern for short duration storms. The 30th percentile distribution was used for hyetograph development of all storm return periods for the 1-hour and 2-hour duration storms

The AES design criteria for a 12-hour Storm Rain Distribution along the British Columbia Coast was used for the long duration storm rainfall pattern. The 50th percentile distribution was used for developing the hyetographs of all storm return periods for the 6-hour, 12-hour and 24-hour storms.

The AES Storm Distribution graphs are provided in Appendix D.

The existing Intensity-Duration-Frequency (IDF) curves for the West Vancouver Municipal Hall Station were used to develop design storms used for generating peak flows in the hydrologic model. An updated IDF curve was obtained from Metro Vancouver and is included in Appendix D.

Design storms were used to generate hydrographs and peak flows for the 2-year, 10-year, 100-year and 200-year return period. A summary of the calculated design storms with their respective hyetograph distributions are provided in Appendix E. Tables E-1 through E-5 give the design storms for various durations. Figures E-1 through E-20 illustrates the hyetographs for the storms. Design storms for the middle and upper watershed areas were then developed by multiplying the precipitation factors developed in Section 4.2.2 above.

4.2.5 Snowmelt and Rainfall Analysis

Daily rainfall and snowmelt estimates were retrieved from Environment Canada for the West Vancouver AUT station in the District of West Vancouver. The snowmelt model from Environment Canada was developed using five degree-day type equations after which a Gumbel distribution was applied to provide annual extreme snowmelt values for durations from 1 to 30 days for return periods up to 100 years. The 24-Hour (1-day) 2-year return period storm at the West Vancouver AUT gauge was found to have a total volume of 85.68 mm +/- 2.93 mm. The 50% confidence limits have been included by Environment Canada. The 24-Hour 100-year return period storm was found to have a total volume of 139.89 mm +/-12.54 mm.

Adjusted rainfall volumes used in the stormwater model were calculated using Elevation-Intensity factors applied to the Municipal Hall rainfall estimates. A comparison is made to ensure that the adjusted volumes account for the increase in rainfall intensity at higher elevations and the influence of snowmelt.

Using the Elevation-Intensity graph from Section 4.2.2, the adjusted rainfall volume for the 24-Hour, 2-year return period storm at 168 m (same elevation as the West Vancouver AUT gauge) totals 101.2 mm. This compares to the 85.68 mm calculated for rainfall plus snowmelt by Environment Canada, which is 15% less.

Similarly, the adjusted rainfall volume for the 24-Hour, 100-year return period storm at an 168m (same elevation as the West Vancouver AUT gauge) totals 225.2 mm. This compares to the 139.89 mm calculated for rainfall plus snowmelt by Environment Canada, which is 38% less.

Calculations show that the West Vancouver Municipal Hall gauge, in conjunction with the Elevation-Intensity factors, give larger values than the Environment Canada rain and snowmelt volumes for the 24-Hour, 2-Year storm and the 24-Hour, 100-Year storm.

TABLE 4-2
MODELLED RAINFALL VOLUMES VS. RAINFALL PLUS SNOWMELT VOLUMES

Design Storms	Volume @ Municipal Hall	Model at 168m	Env. Canada at 168m
		Adjusted Volume Used	Rainfall plus Snowmelt Volume
24Hr, 2Yr	74.4 mm	101.2 mm	85.7 mm
24Hr, 100Yr	165.6 mm	225.2 mm	139.9 mm

Historical data from the Municipal Hall, Capilano Golf Course and Cypress Ranger Station were used to develop the Elevation-Intensity graph. An overestimate of the rainfall plus snowmelt volumes at the West Vancouver AUT gauge is acceptable as the Capilano Golf Course and the Cypress Ranger Station are located in areas with a higher potential of snowpack accumulation. Based on this evidence, it is reasonable to conclude that the adjusted rainfall volumes using the Elevation-Intensity factors are sufficient to model rain and snowmelt in West Vancouver.

4.3 Stream Flow

The significant streams covered in the study area included the Pipe Creek, Westmount Creek, Cave Creek, Turner Creek and Godman Creek systems. Northwest Hydraulic Consultants Ltd. recorded stream flow events from six locations, two gauges each at Pipe, Cave, and Godman Creeks. These gauges monitored stream flows from March 2008 through February 2010 for purposes of model calibration and correlation of rainfall impacts on stream flows.

4.3.1 Planning Models

For modeling purposes, the study watersheds described in Section 3 were further delineated into sub-catchments. The sub-catchments were defined according to the complexity of the storm sewer system and also derived based on topographic data provided by the District of West Vancouver.

The sub-catchment areas above the highway were delineated by topgraphic data and generalized for modeling purposes.

The PCSWMM model provides for a watershed management analysis to determine sequencing of flows, and the affects of diversion and detention solutions. Rainfall storm inputs directed to sub-catchments are used to generate runoff from undeveloped and developed areas in the system. The sub-catchments are then linked to junctions, which through conduits, other junctions, and outfalls modelled in the model space, are used to hydraulically route the flows and identify where surcharge or special flow locations require evaluation.

The following sub-sections include sub-catchment delineation, design criteria for precipitation, soil infiltration rates, and surface roughness coefficients. The drainage criteria by sub-catchment are listed with the model results.

4.3.2 Sub-catchment Delineation

Section 3.6 describes the topography of the five watersheds in the study area and also summarizes the corresponding sub-catchments in each watershed. Delineations were first made based on existing stormwater plans and topographical contour data. Sub-catchment delineations for areas above the Upper Levels Highway have also been assessed and were generalized for modelling purposes. The sub-catchment delineation above the Highway is not in sufficient detail to assess specific hydraulic performance of the existing drainage facilities in these upper areas.

4.3.3 Soil Infiltration Rates

Rarely does all the precipitation contribute to surface runoff. Depression storage, interception, evapotranspiration, and infiltration reduce the amount of stormwater available for runoff. These in turn depend on soil type, vegetation and land use. Antecedent soil conditions also affect the amount of precipitation that will become runoff.

The study area consists of moderate to moderately steep (35% to 60%) slopes with erosion-resistant granitic bedrock at relatively shallow depths (i.e. generally less than 1 m to 2 m). The slope morphology is strongly bedrock-controlled. Streambeds and banks formed in bedrock are common.

Horton's infiltration model was used in the PCSWMM model. Based on analyses done recently by Golder Associates, Opus DK's past work experience in the Lower Mainland, and in consultation with InterCAD, assuming partially saturated ground to reflect winter conditions, the following Horton's infiltration parameters were used in the PCSWMM model as follows:

- Initial infiltration rate = 5 mm/hour
- Final (min) infiltration rate = 0.2 mm/hour
- Decay rate (β) = 1.8 hour^{-1}

The slopes and flow lengths of the sub-catchments were estimated from the topographical conditions Impervious depression storage and pervious depression storage were estimated as 5 mm and 15 mm, respectively.

An aerial map of the watershed as well as a development plan of the new development from InterCAD were used to calculate the impervious surfaces used in the PCSWMM model. The percentage areas for impervious surfaces pre- and post-development are shown in Appendix F.

4.3.4 Roughness Coefficients

The runoff hydrograph of the overland sub-catchments is affected by the roughness and slope of the terrain. The timing of peak runoff flows as they are conveyed downstream over land, in streams, or in storm drains are affected by the roughness coefficient (Manning's 'n') of each entity.

The PCSWMM model simulated flows over land and through streams and culverts to points of discharge for each of the creeks. No ditches were included in the model. Table 4-3 summarizes the Manning's 'n' values used.

TABLE 4-3
MANNING'S 'N' VALUES

Input Parameters	Manning's n
Channels	0.013-0.052
CMP culverts	0.024
Concrete culverts	0.013
All new storm drains	0.013
Impervious areas	0.013
Pervious area (developed)	0.2
Pervious area (undeveloped forest)	0.4

Roughness coefficients for each length of stream between each culvert in the five watersheds were approximated using Cowan's Method. Using this method, measurements of channel roughness were made based on the following stream characteristics:

- material involved;
- degree of irregularity;
- variations of channel cross section;
- relative effect of obstructions;
- vegetation; and
- degree of meandering.

The roughness of each length of stream was calculated based on these six parameters; calculated values ranged from 0.013 to 0.052. An inventory of the channel roughness for available creeks is included in Appendix G.

4.4 Stormwater Management

Solutions to protect habitat and the public can be conflicting. Development displaces natural processes, which normally result in increased storm flows that stress the drainage system and shared uses. Stress can occur in natural habitat through the increased frequency of runoff from low intensity storms that causes increased erosion and sediment transport, as well as increased pollutant loads. Stress can also occur from inadequate downstream capacity to allow the infrequent runoff from high intensity storms. This impacts both natural and developed uses.

To adequately address the drainage issues, a stormwater management solution is needed that examines both low intensity long duration periods for habitat impact, and high intensity design storms for public and property impact.

For the West Vancouver area below the Trans Canada Highway at elevation 150 m GSC, land use is now largely developed. The opportunity for improvements to habitat and public use lies in a review of existing practices and the adoption of improved practices where drainage improvements are needed. Long term solutions that restore riparian forest along stream banks and recreate natural hold-up of runoff would require an increased scope of study above what is provided here. For areas above 150 m GSC to 365 m GSC (1200 feet) low impact development (LID) solutions, should be considered.

4.4.1 Public Protection

Protection against flooding for agricultural areas is normally established at the 10-year return stream flow. For urban areas, stormwater solutions will provide attenuation of the peak flows from the 200-year return period storm and minimize flood risks.

4.4.2 Water Quality Protection

Water quality improvement facilities for stormwater should be designed to reduce the impact of runoff events that comprise the majority (e.g. 90%) of the total annual runoff volumes. The District of West Vancouver currently does not have water quality protection criteria for stormwater management facilities. These, however, should be developed for the 24-hour 1-year or 2-year return events as storms of this size and smaller are responsible for greater than 90% of the annual runoff volume.

As part of the ISMP, stormwater management solutions include the analysis of suitable types of Low Impact Development (LID) solutions to be initiated in the five watersheds. In addition to reducing peak flows in the system, the LIDs also improve water quality through different processes. The LIDs considered include:

- wetland infiltration and/or rain gardens;
- absorbent soils'
- permeable pavers;
- roof runoff collection in rock pits; and
- rain barrels.

Discussion and recommendations of LID solutions are included in Section 5.5.3 Individual Lot Development Guidelines.

4.5 Costing

The unit supply costs for drainage pipes of various diameters are shown in Appendix H. Langley Concrete Group and Woseley Inc. were contacted by Opus DK on February 3, 2010 to provide updated supply costs for concrete and HDPE drainage pipe. These costs were multiplied by a factor of two to develop installation costs and were used in the cost estimates summarized in Section 5.5.4.

4.6 Flood Design Management Guidelines

In 2004, the Province delegated to municipalities its responsibility respecting flood-proofing standards. In doing this, the Province provided a guideline for use by municipalities (MOE, 2004). In this guideline, the Province recommended that flood protection be based on the designated flood, which is described as a flood with a magnitude as to equal a flood having a 200-year recurrence interval.

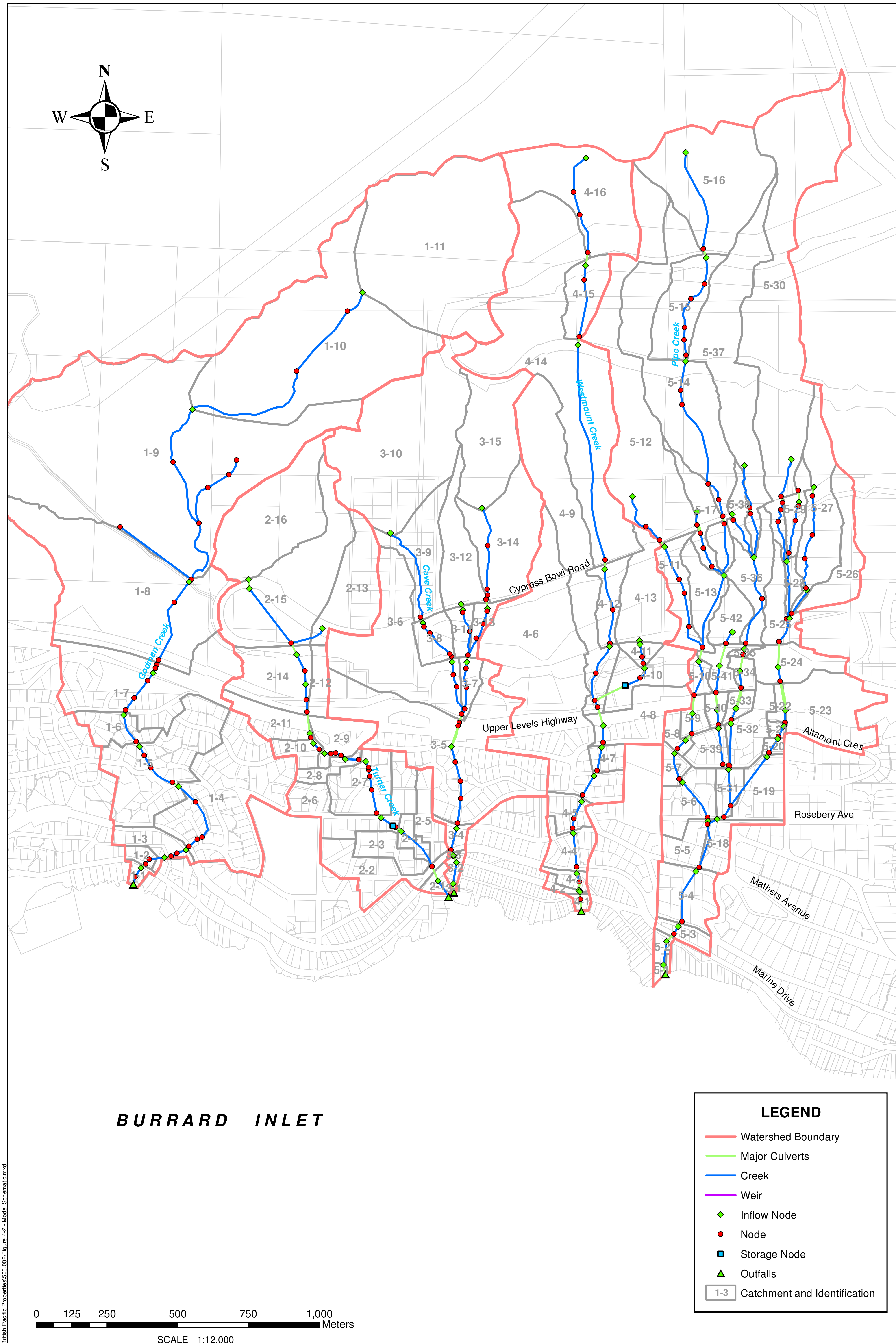
For the purposes of the ISMP, the provincial guidelines for flood control design to the 200-year storm event were used.

4.7 Model Development

To analyze the hydraulics of the five watersheds, a computer model was developed using PCSWMM (version 2.18.475). The creeks were modelled as a system of culverts, channels, and other hydraulic structure components being fed by runoff from a series of sub-catchment drainage areas. The sub-catchments were delineated by topographical contour data and drainage maps provided by the District of West Vancouver and InterCAD Consulting Ltd. Culvert sizes and locations were taken from District infrastructure inventory and field reconnaissance. The creek channel properties were taken from contour data and field observation.

The model was used for analysis of infrequent, high intensity storms to determine design maximum runoff and volume conditions for the selection of stormwater management solutions that will protect property and life. The model was also used to determine maximum allowable base flows in the stream to protect streams from channel erosion. Further analysis for frequent, low intensity storms to protect the environment is provided in Section 5.5.3 through Low Impact Development.

Catchment areas were delineated, as described in Section 4.3.2, based on the connectivity of the existing drainage system, topographic mapping, and information obtained during field work. The delineations for sub-catchment areas above the Upper Levels Highway were based on



Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 4-2 - Model Schematic.mxd

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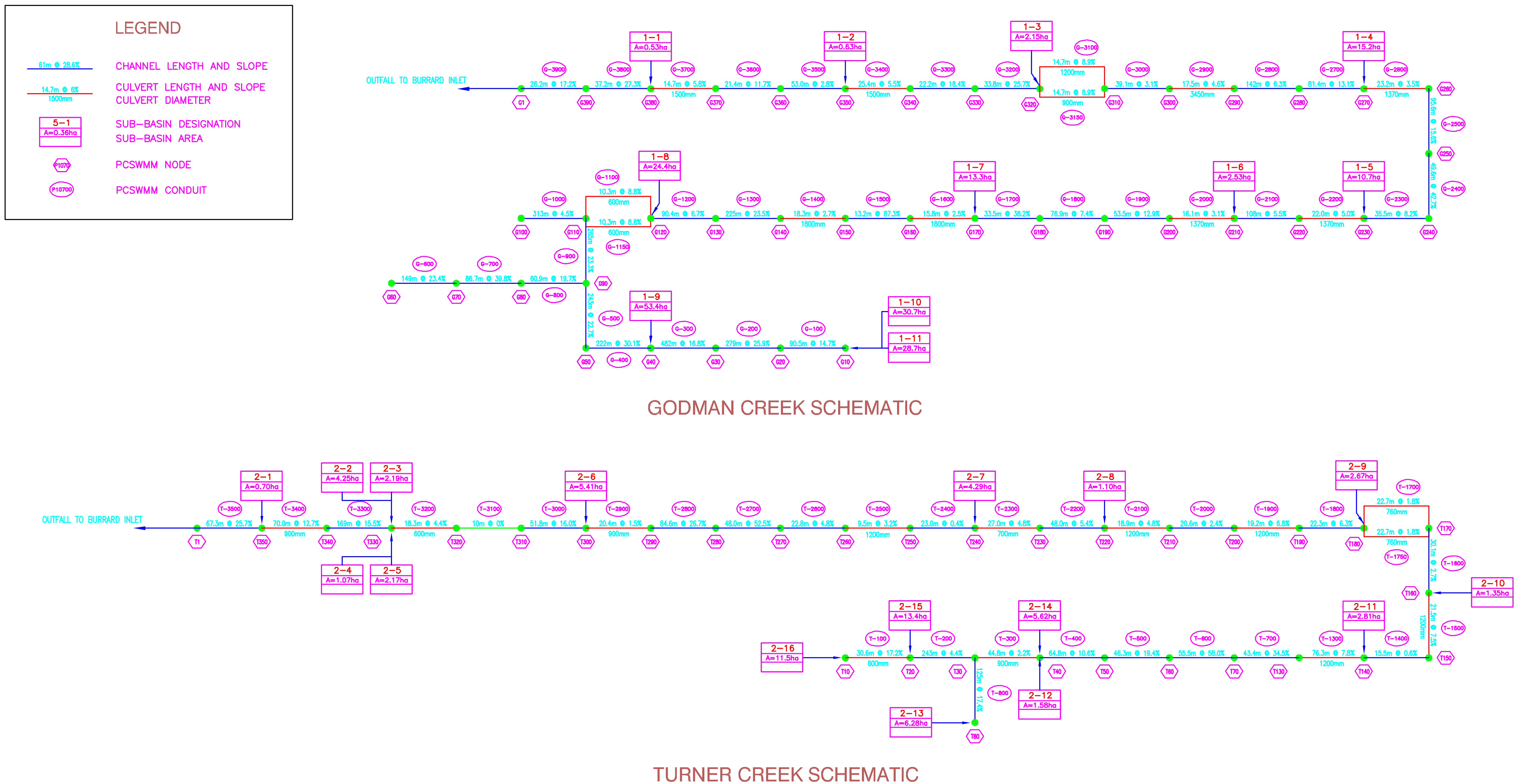
GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
MODEL SCHEMATIC AND SUB-CATCHMENTS

FIGURE 4-2



DISTRICT OF WEST VANCOUVER

GODMAN AND TURNER CREEK DRAINAGE SCHEMATIC

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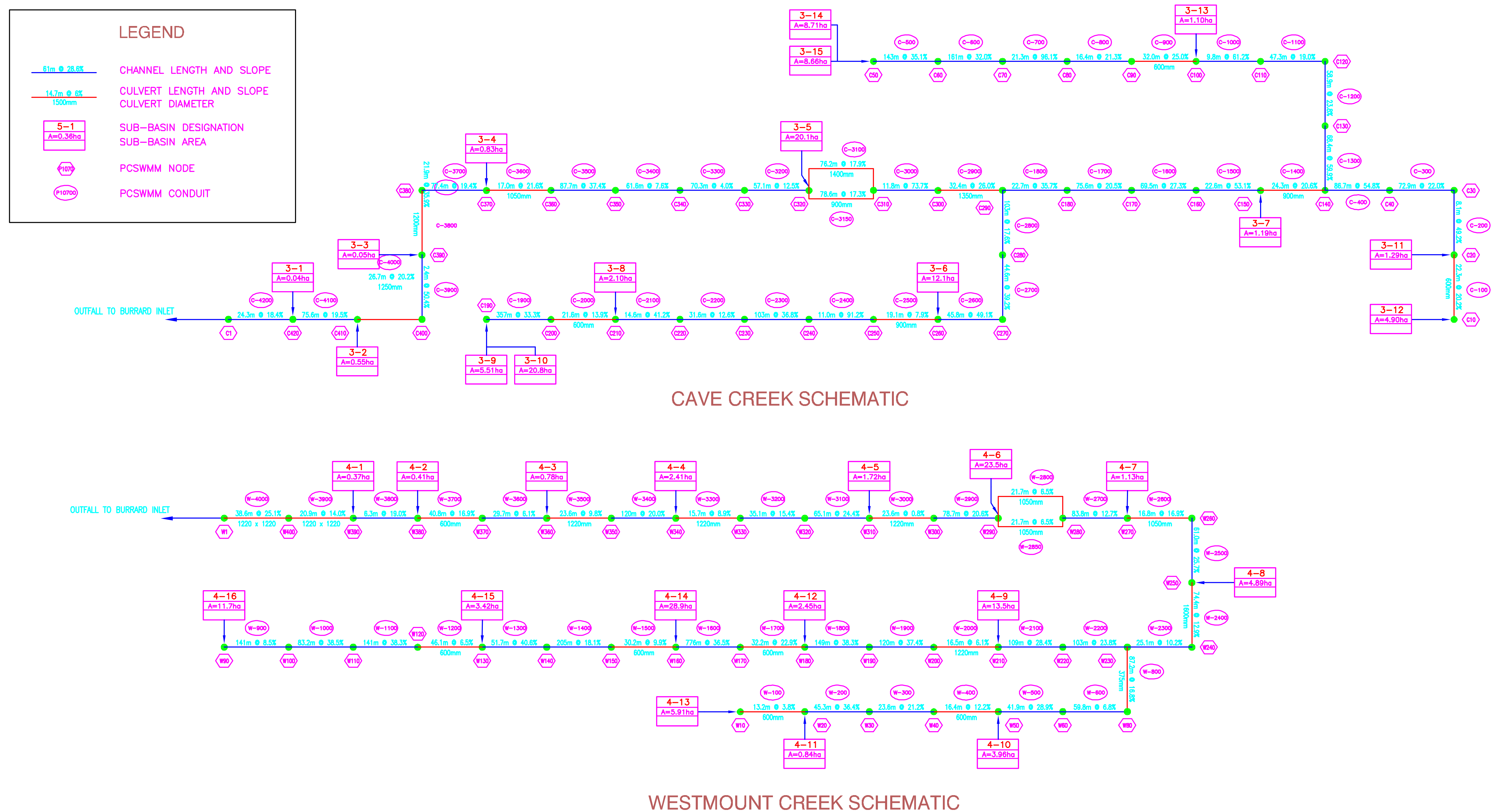
FIGURE 4-3



DISTRICT OF WEST VANCOUVER

CAVE AND WESTMOUNT CREEK DRAINAGE SCHEMATIC

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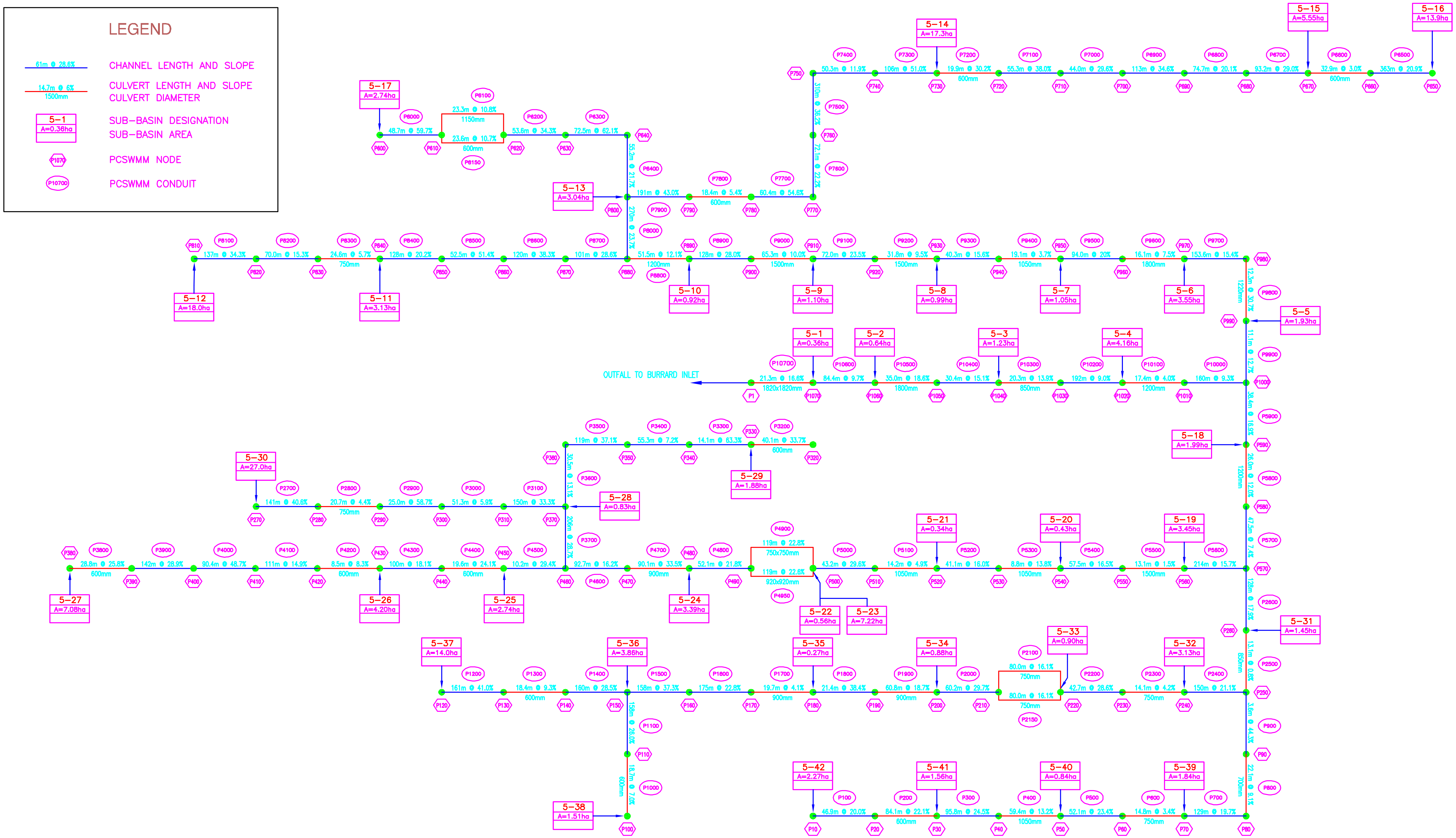
FIGURE 4-4



DISTRICT OF WEST VANCOUVER
PIPE CREEK DRAINAGE SCHEMATIC

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FIGURE 4-5

topographic mapping and from input from InterCAD. Catchments were assigned the following attributes:

- slopes, using contour information;
- impervious areas, using the most recent aerial photo; and
- overland flow lengths, using estimated lengths from the aerial photo.

The model included nodes and inflow points established at all road crossings, culverts, and other points of interest. Storage nodes were input for all of the detention ponds in the watersheds. The model schematic is shown on Figure 4-2. Figures 4-3, 4-4 and 4-5 illustrate the drainage schematics for each creek including catchment, conduit and node ID numbers used in the PCSWMM model.

Information collected during the watershed inventory was used to build the model. Channel cross-sections were simplified into representative trapezoidal channel sections for the purposes of routing in the hydraulic model. Channel slopes were estimated using the available contour elevations. Culverts were modelled based on measured sizes and information from the District. Channel and conduit roughness values were assigned based on Cowan's Equation for Estimating Channel Roughness (D&K, 1973) and published roughness values for the various conduit materials respectively.

The hydrologic runoff calculations in PCSWMM were used to estimate groundwater and interflow portions of the runoff hydrograph. Infiltration rates, soil depths, and soil hydraulic conductivity were all input based on field observations and the findings of the hydro-geological assessment. The analysis assumes that storms occur on partially saturated soil conditions typical of fall and winter seasons in the Lower Mainland.

Horton's Equation, an empirical infiltration model widely used in stormwater modelling, was chosen for this analysis. The usage and values of infiltration parameters were not noted in either the MacDonald or Rodgers Creek ISMPs. Therefore, parameters were not comparable to any data used previously in West Vancouver.

The following input parameters were used in the PCSWMM model; these inputs were based on Opus DK's past work and experience in the Lower Mainland.

- Manning's typical roughness coefficient "n"
  - open ditch with rocks on the bottom and bank — 0.045
  - excavated channel with cobble bottom — 0.04
  - excavated channel with stony bottom and weedy banks — 0.035
  - corrugated metal pipe (CMP/CSP) — 0.024 – 0.033
  - pervious area (undeveloped forest) — 0.4
  - pervious area (developed) — 0.2
  - impervious area — 0.013
- Impervious area depression storage — 0.5mm
- Pervious area depression storage — 3.8mm
- Maximum initial infiltration rate — 5 mm/h
- Minimum infiltration rate — 0.2 mm/h
- Decay rate of infiltration in Horton's Equation — 1.8 hour^{1}

4.8 Model Calibration

This section provides our review of the calibration of the storm water model using the January 15, 2010 storm event. The purpose of this section is to review the hydraulic model calibration and evaluate the suitability of the calibrated model for design purposes.

4.8.1 Background

As part of the ISMP process, stream flow monitoring stations were set up by Northwest Hydraulic Consultants (NHC) on Pipe, Cave and Godman Creeks. Two stations on each of these creeks were installed:

1) At the lower end of the watershed near tidewater; and
2) In the upper reaches of the creeks above the Upper Levels Highway. Flow data was collected every 10 minutes from March 2008 to April 2010.

Section 3.9 of the ISMP reviews a number of storm events. Rainfall data collected from the District of West Vancouver, Works Yard Gauge, situated on Cypress Bowl Road at the District's Works Yard provided the measured rainfall in 5 minute increments. This review determined the most suitable storm to develop the calibrated model is the January 15, 2010 event. This event was selected as there was an absence of complex snow fall and snowmelt, and the rainfall was more spatially and temporally consistent compared to the other events.

4.8.2 Sensitivity Analysis

Storm water models have a large number of flexible inputs that can be adjusted to calibrate the model. Fifty parameters were analyzed by increasing or decreasing the parameter to determine the sensitivity. The parameters are all from the model subcategories of Subcatchments and Conduits. The following Table 4-4 summarizes 11 of the parameters that have significant sensitivity.

TABLE 4-4
SENSITIVITY ANALYSIS

Item	Parameter	Subcategory
1	Area	Subcatchments
2	Pervious Depression Storage	Subcatchments
3	Maximum Infiltration Rate	Subcatchments
4	Minimum Infiltration Rate	Subcatchments
5	Infiltration Decay Rate	Subcatchments
6	Ground Water Flow Coefficient	Subcatchments
7	Ground Water Flow Exponent	Subcatchments
8	Surface Water Flow Coefficient	Subcatchments
9	Surface Water Flow Exponent	Subcatchments
10	Length	Conduits
11	Roughness	Conduits

Items 1 and 10 are obtained from spatial mapping and recorded contours, therefore these parameters should be minimally adjusted. Items 3 through 5 are components of the Horton's infiltration equation which simulates the rainfall infiltration in the subcatchments. Items 6 through 9 are components of a groundwater flow equation which is used to simulate the effects of interflow between the infiltrated rainfall from the subcatchments and the flow in the creeks. Other notable parameters include the subcatchment width and slope, which effect the time it takes for rainfall to reach a main tributary. All of these parameters were assessed in the model calibration.

4.8.3 PCSWMM Model Calibration

Model calibration is achieved by adjusting parameters within a reasonable range to meet a set of objectives. In the case of the ISMP model the objective is to simulate the watershed such that the modelled flows resulting from the recorded rainfall correlate with the recorded flow gauge data from Pipe, Cave and Godman Creeks. The calibration process is typically affirmed by assessing a calibrated model against several significant storm events (greater than 2-year return period). At this time the model has been calibrated to a single storm event of smaller magnitude (less than a 2-year return period).

The model was calibrated in a staged process, for each creek the upper catchments were calibrated followed by the lower catchments. PCSWMM's SRTC calibration tool was then used to adjust model parameters so that computed values would meet measured values more accurately.

Observing the January 15, 2010 event, it is clear that during the low intensity rainfall event, there is a noticeable hold-up of rainfall in the watershed which delays the resulting runoff and releases at a slower rate than was simulated in our original hydraulic model. To simulate this effect interflow was introduced to route a portion of the subcatchment infiltration into the creeks. Interflow is typically not required in a primarily piped system, which is isolated from groundwater conditions. However, as our study area consists of open channel creeks, interflow has a significant role on the routing of flow during low intensity events. This adjustment allowed for a better match between modelled and measured runoff volumes and hydrograph shape.

At the onset of the January 15, 2010 event there is a gradually diminishing flow present in the creeks. This diminishing baseflow in the creeks is a result of runoff from rainfall in the days prior to January 15th. To simulate these initial conditions, the model was run with the recorded rainfall from the five days prior to the onset of the event.

The parameters adjusted in the PCSWMM model are summarized in Table 4-5.

TABLE 4-5
PERCENT CHANGE IN MODEL PARAMETER

Calibration Parameters	Upper Godman	Lower Godman	Upper Cave	Lower Cave	Upper Pipe	Lower Pipe
Area	0	0	0	0	0	0
Maximum Infiltration Rate	0	0	0	45	15	25
Minimum Infiltration Rate	0	0	0	45	15	-15
Decay Constant	0	15	0	-25	10	-20
Ground Water Flow Coefficient	-5	0	0	0	0	0
Surface Water Flow Coefficient	-5	0	0	0	0	0
Length	10	5	0	15	10	10
Roughness	10	-5	0	20	15	10

At this time the model has only been calibrated to the January 15, 2010 storm. The characteristics targeted during the calibration included:

- Peak Flow;
- Total Flow Volume;
- Time to Peak Flow;
- Base Flow; and
- Curve Shape.

Figures 4-6 through 4-11 illustrate the calibrated model against each of the flow gauges. The blue hydrograph shows the initial modeled flow, the green shows the adjusted flows after calibration and the red hydrograph is the recorded flow at the monitoring station.

TABLE 4-6
CALIBRATION RESULTS

	Upper Godman			Lower Godman		
	Observed	Initial	Calibrated	Observed	Initial	Calibrated
Peak Flow (m^3/s)	1.278	1.328	1.269	2.719	2.149	2.155
Total Flow (m^3)	82,650	88,830	87,160	127,800	138,200	127,600
Peak Unit Area Runoff ($m^3/s-km^2$)	1.07	1.12	1.07	1.57	1.24	1.25
	Upper Cave			Lower Cave		
	Observed	Initial	Calibrated	Observed	Initial	Calibrated
Peak Flow (m^3/s)	0.1808	0.3348	0.3348	1.165	1.531	1.278
Total Flow (m^3)	20,810	13,840	13,840	62,660	67,440	66,910
Peak Unit Area Runoff ($m^3/s-km^2$)	1.06	1.97	1.97	1.31	1.72	1.44
	Upper Pipe			Lower Pipe		
	Observed	Initial	Calibrated	Observed	Initial	Calibrated
Peak Flow (m^3/s)	0.6532	0.6058	0.579	2.166	2.84	2.576
Total Flow (m^3)	27,240	27,570	27,590	146,000	130,900	129,800
Peak Unit Area Runoff ($m^3/s-km^2$)	1.98	1.84	1.75	1.27	1.67	1.52

The rainfall data is limiting as only one gauge is being used for the entire system which is not a suitable rainfall pattern for all the subcatchments. As a result, the peak flow and total flow volume did not correlate well for some of the gauge locations.

The Upper Cave Creek calibration zone was left unadjusted. It proved very difficult to calibrate to this gauge and adjustment of the input parameters was ineffective in simulating the measured response to the rainfall recorded at the Works Yard rainfall station. The catchment area of the Upper Creek gauge is significantly smaller than the other stations at only 17 hectares. This gauge would be of better use if it were located further downstream in the drainage reach to allow for the rainfall distribution to be averaged out over a larger area. For reference, the catchment areas and unit area runoffs for each of the key events during the flow monitoring

period are summarized in
Table 4-6.

When unregulated, the drainage capacity of these catchments is capable of conveying mean annual peak discharges from between 1 to 6 $m^3/s\text{-}km^2$ due to the small size of the drainage, the steepness of the terrain and straightness of the channels. (Guide to Peak Flow Estimation for Un-gauged Watersheds in the Lower Mainland Region, MOE&P). The relatively small watersheds of the West Vancouver drainages shown in the calculated unit area runoff of Tables 4-6 and 4-7, illustrate that the watershed is capable of greater capacity than can be simulated for the recorded flows. An attempt to simulate and calibrate a system that is below its effective carrying capacity will reduce ability to calibrate the model effectively. Repeatability for other events is therefore difficult to achieve.

TABLE 4-7
CATCHMENT AREAS AND UNIT AREA RUNOFFS

Event	Peak Recorded Unit Area Runoffs					
	Lower Pipe Creek ($m^3/s/km^2$)	Upper Pipe Creek ($m^3/s/km^2$)	Lower Cave Creek ($m^3/s/km^2$)	Upper Cave Creek ($m^3/s/km^2$)	Lower Godman Creek ($m/s/km^2$)	Upper Godman Creek ($m^3/s/km^2$)
Catchment Area (Ha)	170	33	89	17	173	119
Dec. 21, 2009	1.7	2.0	1.6	1.1	2.3	2.2
Jan. 15, 2010	1.3	2.0	1.3	1.1	1.6	1.1
Jan. 7, 2009	1.3	0.7	1.4	0.8	1.0	1.3
Sept. 6, 2009	0.2	0.7	0.5	0.0	0.2	0.0
Nov. 18, 2009	0.7	0.6	0.9	0.6	0.8	0.4

4.8.4 Calibration Conclusions

After significant effort spent in calibrating to the January 15, 2010 event, the resulting model is considered appropriate in evaluating lower intensity storms. Further efforts to calibrate will have diminishing returns, considering the lack of significant events during the flow monitoring period and size of the measured flow data relative to the watershed capacity. Further, it is evident that data from the Works Yard rain gauge does not accurately model all the creeks, primarily due to proximity and elevation variances. Discrepancies in the calibrated and recorded hydrographs are inevitable.

We recommend that the District consider the following:

1) Continue to review future rainfall data until another significant storm is recorded.

2) Install a new rain gauge between Pipe Creek and Cave Creek. An additional gauge in this vicinity can be used to improve the calibration of the eastern catchment.

3) Initiate further flow monitoring of the creeks in order to capture additional events suitable for additional future calibration of the hydraulic model.

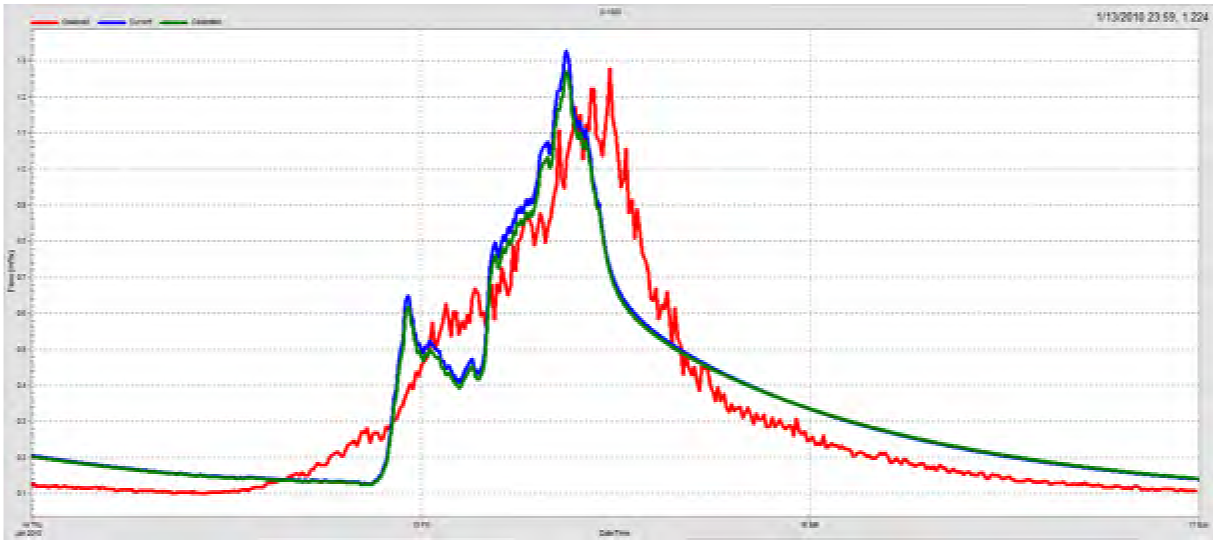


Figure 4-6: Upper Godman Creek Hydrograph - January 15th, 2010

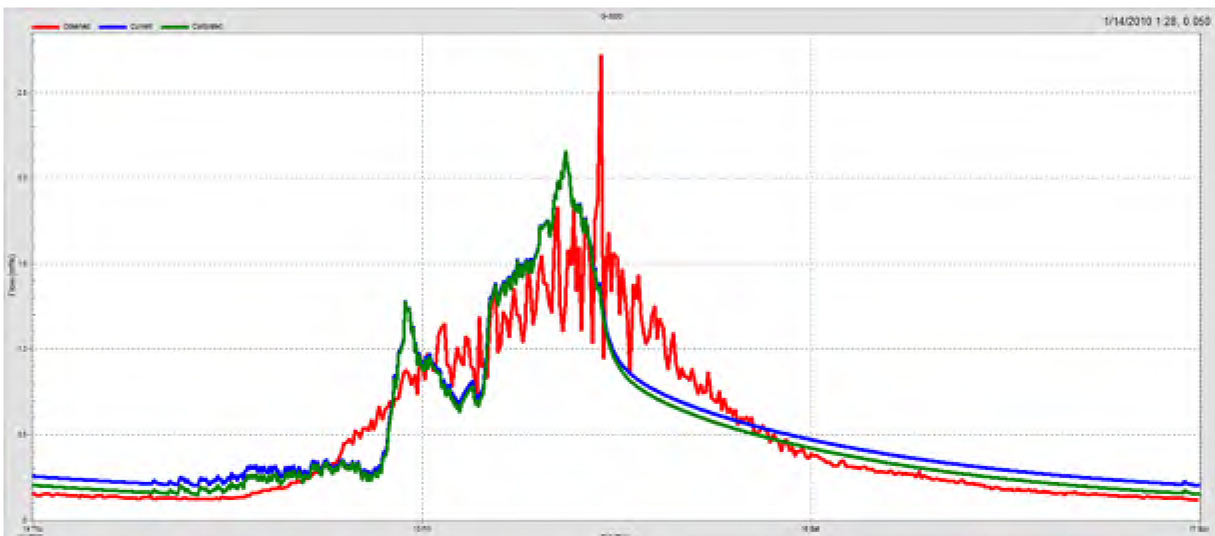


Figure 4-7: Lower Godman Creek Hydrograph - January 15th, 2010

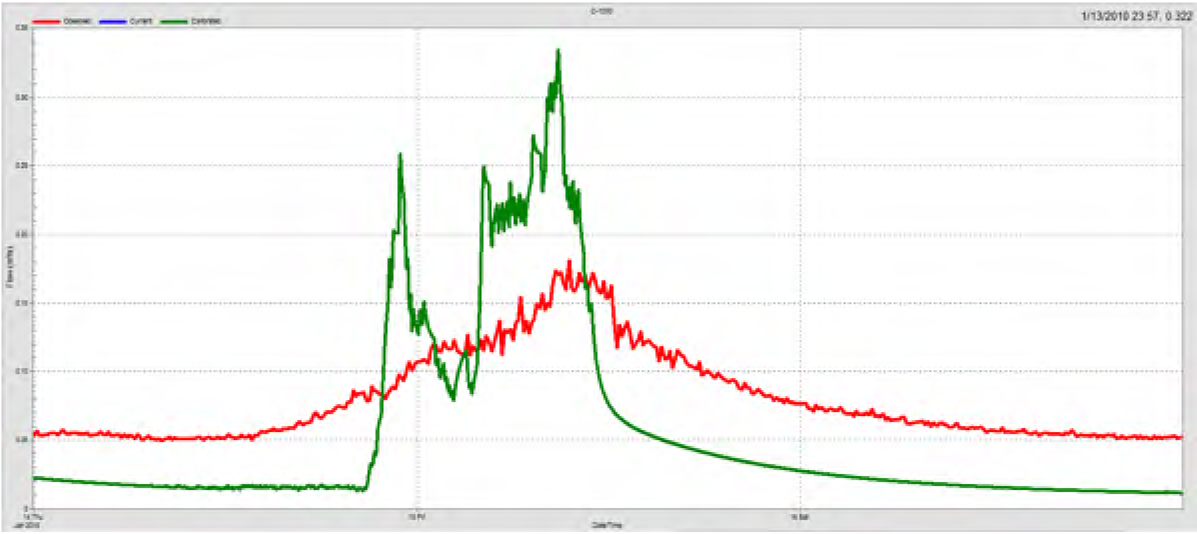


Figure 4-8: Upper Cave Creek Hydrograph - January 15th, 2010

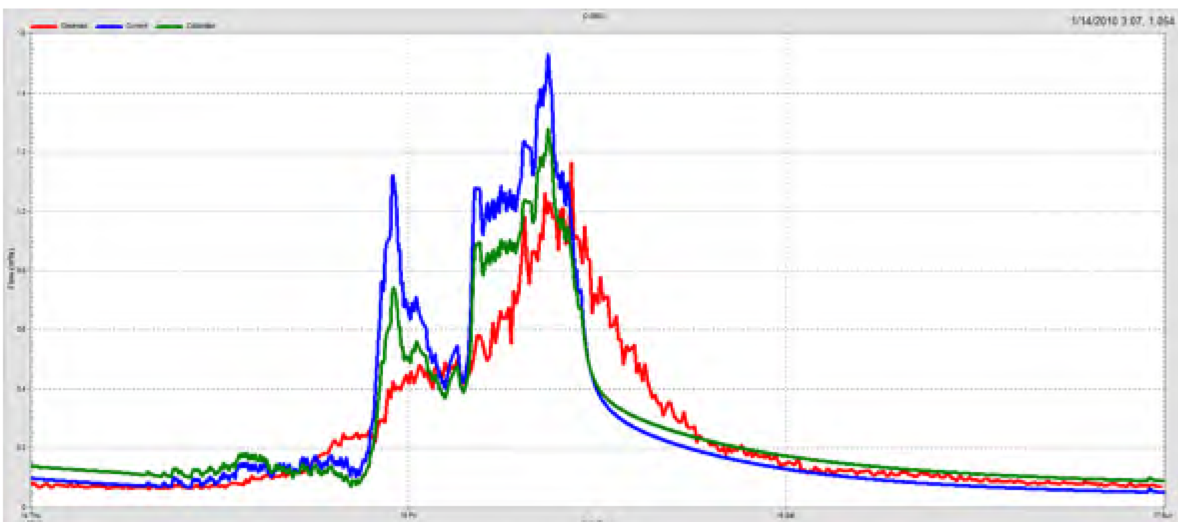


Figure 4-9: Lower Cave Creek Hydrograph - January 15th, 2010

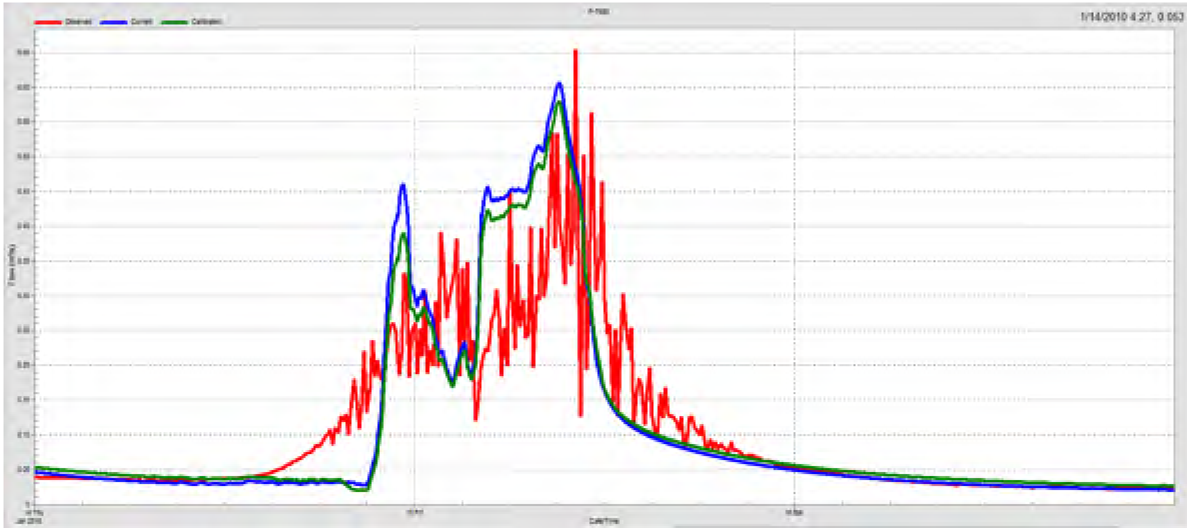


Figure 4-10: Upper Pipe Creek Hydrograph - January 15th, 2010

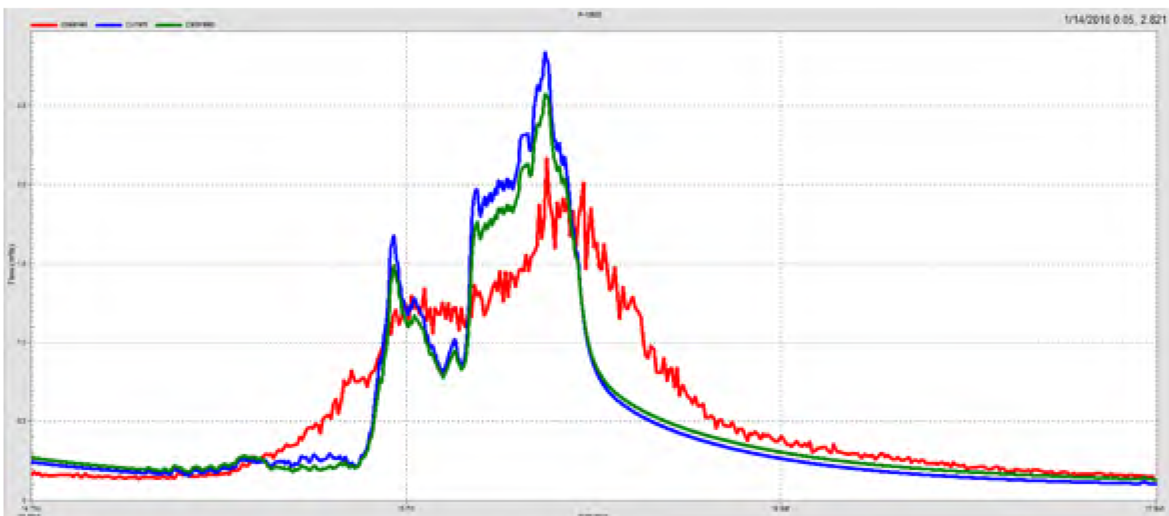


Figure 4-11: Lower Pipe Creek Hydrograph - January 15th, 2010

4.9 Model Verification

The model results were compared to historical peak unit area runoff rates developed for the Ministry of Transportation and Highways (Thurber 1983). The historical peak unit area runoff rates for southwest British Columbia and parts of Washington were compared for each of the five catchment areas based on unit runoff (expressed in $m^3/s/km^2$) versus the drainage area (expressed in km^2). Table 4-8 shows the unit area runoff rates as computed by the PCSWMM model for storms of 1-hour and 2-hour durations for the 100-year storm event. PCSWMM values ranging from 5.3 $m^3/s/km^2$ to 9.1 $m^3/s/km^2$ were found. The average value over the five catchments was 6.4 $m^3/s/km^2$ for the 1-hour storm, and 7.9 $m^3/s/km^2$ for the 2-hour storm. These values compare closely to the unit area runoff rates from historical data, attached as Figure A.2a in Appendix I, where the value for a 100-year storm with a catchment area of about 1 km^2 is about 5 to 9 $m^3/s/km^2$. The modelled unit area runoff is within the range of historic data.

TABLE 4-8
UNIT AREA RUNOFF RATES FOR 100-YEAR STORM EVENT

Creek	Area	1hr in 100 year		2hr in 100 year	
		Q	Q/A	Q	Q/A
	(km^2)	(m^3/s)	($m^3/s/km^2$)	(m^3/s)	($m^3/s/km^2$)
Godman	1.82	12.2	6.9	14.9	8.2
Turner	0.66	3.7	5.5	4.2	6.3
Cave	0.88	5.9	6.7	7.6	8.6
Westmount	1.06	5.7	5.3	7.7	7.3
Pipe	1.73	13.2	7.6	15.7	9.1
Average	**1.23**		**6.43**		**7.88**

4.10 Design Storms

Design storms were developed based on Atmospheric Environment Services (AES) statistical distributions as described in Section 4.2.4.

4.11 Estimated Peak Design Flows

Typically, areas with higher percentages of directly connected impervious areas are governed by shorter duration storm events (1-hour to 2-hour), and less developed areas tend to be governed by longer duration storms (6-hour to 24-hour).

The 200-year peak flows are recommended for use in mountainous areas to comply with provincial criteria. Estimated 200-year return period design peak flows from PCSWMM at selected points are summarized in Table 4-9. Peak design flows occur in the creek catchments during the 2 hour duration, 200-year storm.

TABLE 4-9
DESIGN 200-YEAR PEAK FLOWS (m^3/s)

Strategic Location	Godman	Turner	Cave	Westmount	Pipe
Upper Levels Hwy	15.5	3.4	6.8	6.9	15.4
Outlet to Burrard Inlet	17.0	4.7	8.6	9.1	17.5

5.0 SYSTEM REVIEW AND MITIGATION OPTIONS

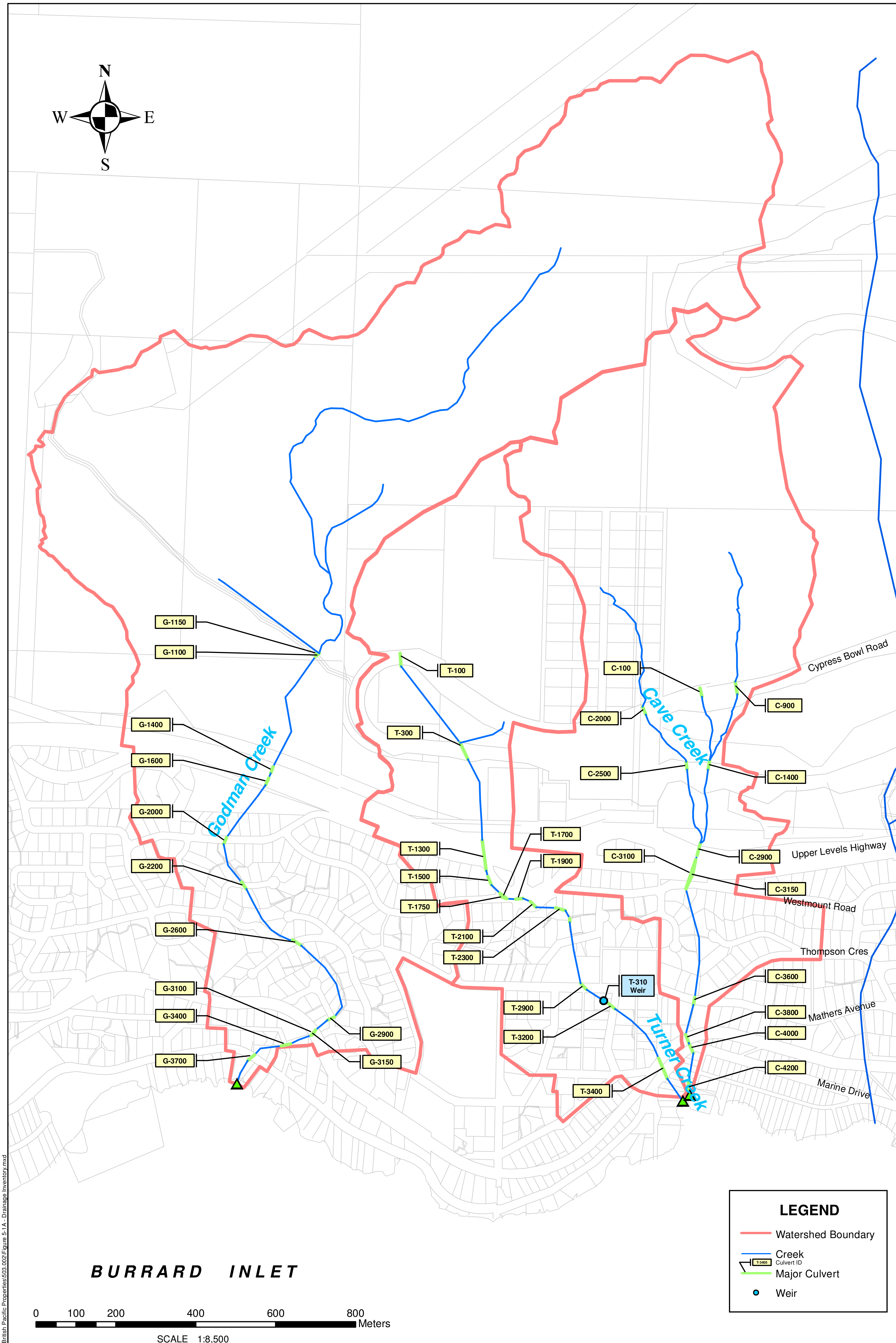
This chapter presents a record of existing drainage structures, known drainage problems and an estimate of major system limitations for the Pipe, Westmount, Cave, Turner and Godman Creek watersheds. An inventory of creek crossings, channel sections and other drainage works is included. Existing conditions are simulated to identify current drainage problems in the system. Selected drainage solutions are then analyzed to address future development and mitigation options.

5.1 Existing Drainage Inventory

Field visits were conducted to establish existing creek and watershed characteristics and to provide a thorough inventory of the drainage system. The following items were noted during the field work:

- culvert sizes, materials and types;
- typical channel cross section dimensions, and approximate slopes;
- photographs of channel sections to estimate channel roughness;
- detention facilities and any other hydraulic structures; and
- minor and major flow routes.

Figure 5-1A and 5-1B illustrate the main items from the drainage inventory.



Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 5-1A - Drainage Inventory.mxd

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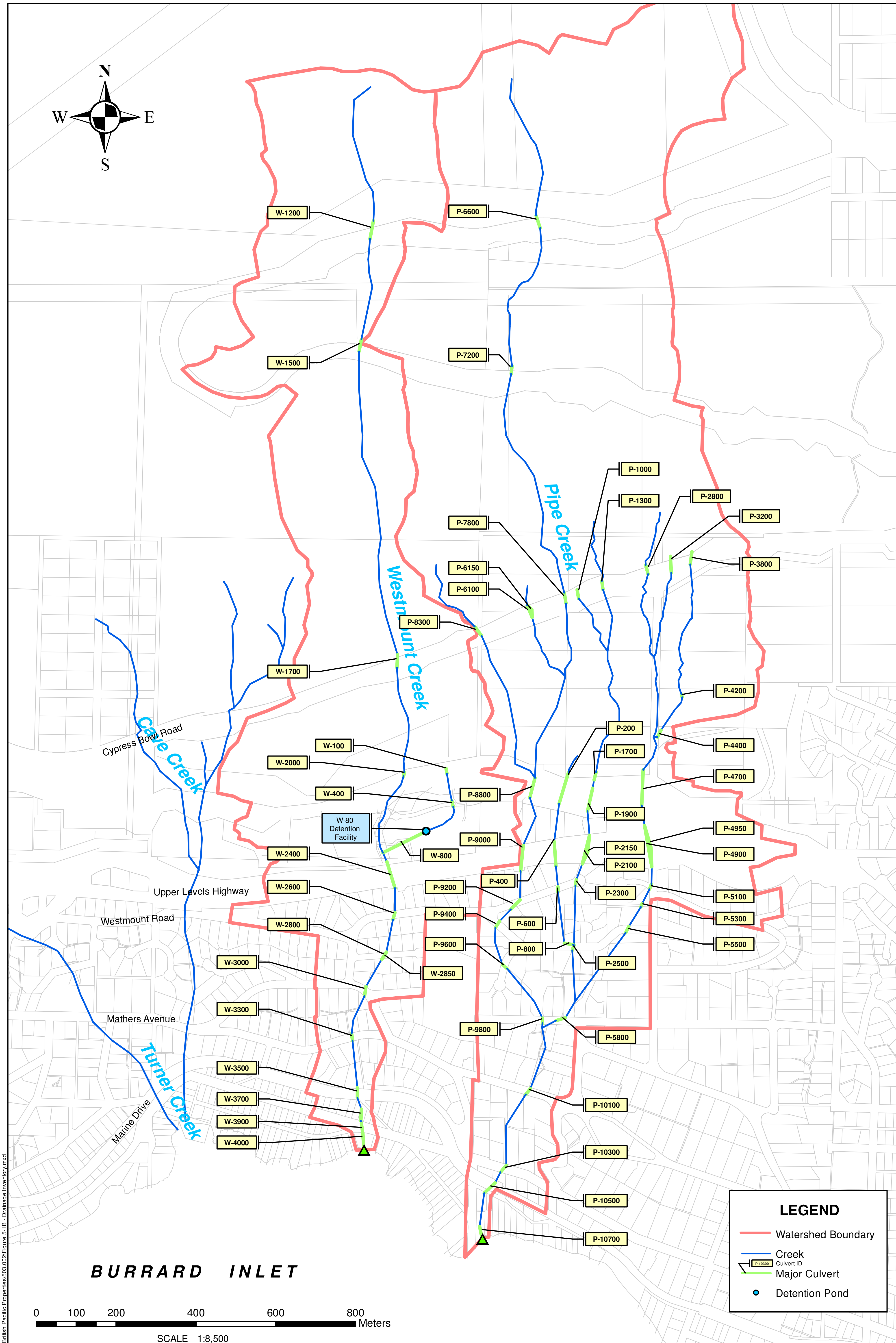
GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
DRAINAGE FACILITY INVENTORY

FIGURE 5-1A



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GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP

DRAINAGE FACILITY INVENTORY

FIGURE 5-1B

Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 5-1B - Drainage Inventory.mxd

The upper creek reaches should be hiked in the near future to complete the Pipe/Westmount/Cave/Turner/Godman field work and inventory. This is important to check for creek confinement, stability, and drainage issues.

5.1.1 Hydraulic Inventory

Table 5-1 summarizes the culvert information for Godman, Turner, Cave, Westmount and Pipe Creeks respectively. Section 5.4.4 provides an analysis of the existing capacities of culverts located below the Upper Levels Highway.

A control weir is on Turner Creek between Mathers Avenue and Hillcrest Street. Although the weir was reported to have been intended as habitat for waterfowl, it also serves to attenuate flows and act as a sedimentation basin.

A detention facility is on Westmount Creek between the Upper Levels Highway and Deer Ridge Drive. The detention facility was installed as part of the drainage solution for a subdivision development and is probably designed to reduce 100-year post-development peak flows to 10-year pre-development peak flows, since this is the design requirement set out in the District of West Vancouver Upper Lands Policy (DWV, 2004).

TABLE 5-1
HYDRAULIC STRUCTURE INVENTORY – ALL CREEKS

Culvert	Watercourse	Location	Size Dia. or WxH (mm)	Material and Type	Length (m)	Slope (%)
Godman Creek						
G-3700	Godman	Marine Drive	1550	Concrete	14.7	3.8
G-3400	Godman	Rose Crescent	1500	Concrete	25.4	5.8
G-3100	Godman	British Columbia Railway	1200	Concrete	14.7	7.8
G-3150	Godman	British Columbia Railway	900	CSP	14.7	7.6
G-2900	Godman	Sharon Place	3250 x 2000	CSP	17.5	4.2
G-2600	Godman	Bayridge Avenue	1350	Concrete	23.2	3.1
G-2200	Godman	Viewridge Place	1370	Concrete	22.0	5.2
G-2000	Godman	Westridge Avenue	1370	Concrete	16.1	2.7
G-1600	Godman	Upper Levels	1800	CSP	15.8	2.5
G-1400	Godman	Upper Levels	1800	CSP	18.3	2.7
G-1100	Godman	North of Upper Levels	600	CSP	10.3	7.5
G-1150	Godman	North of Upper Levels	600	CSP	10.3	6.8
Turner Creek						
T-3400	Turner	Marine Drive	900	Concrete	70.0	15.9
T-3200	Turner	Hillcrest Street	600	Concrete	18.3	3.3
T-2900	Turner	Mathers Avenue	900	Concrete	20.4	1.4
T-2300	Turner	Cedarridge Place	700	Concrete	27.0	5.1
T-2100	Turner	Westmount Road	1220	Concrete	18.9	4.5
T-1900	Turner	Southridge Place	1220	Concrete	19.2	7.2
T-1700	Turner	Southridge Avenue	770	Concrete	22.7	1.8
T-1750	Turner	Southridge Avenue	770	Concrete	22.7	1.6
T-1500	Turner	Westridge Avenue	1220	Concrete	21.5	7
T-1300	Turner	Upper Levels	1220	Concrete	73.2	17.9
T-300	Turner	Cypress Bowl Road	900	CSP	44.8	2.2
T-100	Turner	Cypress Bowl Road	600	-	30.6	16.2
Cave Creek						
C-4200	Cave	Seawall	950 x 1450	Concrete	24.3	14.2
C-4000	Cave	Marine Drive	1250 x 1250	Concrete	26.7	20.8
C-3800	Cave	British Columbia Railway	1200	Concrete	21.9	34.3
C-3600	Cave	Mathers Avenue	1050	Concrete	17.0	18.5
C-3100	Cave	Upper Levels	1400	CSP	76.2	17.4
C-3150	Cave	Upper Levels	900	Concrete	78.6	17.4
C-2900	Cave	Wentworth Avenue	1400	CSP	32.36	26
C-2500	Cave	Cypress Bowl Road	900	CSP	18.8	7.9
C-2000	Cave	Cypress Bowl Road	600	CSP	20.8	13.9
C-1400	Cave (east)	Cypress Bowl Road	900	CMP	24.4	20.6
C-900	Cave (east)	Cypress Bowl Road	600	CSP	32.8	25
C-100	Cave (middle)	Cypress Bowl Road	600	CSP	23.1	20.1
Westmount Creek						
W-4000	Westmount	Seawall	1220 x 1220	Concrete	38.6	22.9

TABLE 5-1 (cont'd.)
HYDRAULIC STRUCTURE INVENTORY – ALL CREEKS

Culvert	Watercourse	Location	Size Dia. or WxH (mm)	Material and Type	Length (m)	Slope (%)
W-3900	Westmount	Marine Drive	1220 x 1220	Concrete	20.9	11.1
W-3700	Westmount	Upstream of Marine Drive	1220	-	40.8	22.6
W-3500	Westmount	British Columbia Railway	1220	Concrete	23.6	9.2
W-3300	Westmount	Mathers Avenue	1220	Concrete	15.7	8.2
W-3000	Westmount	Thompson Crescent	1220	Concrete	23.6	0.8
W-2800	Westmount	Westmount Place	1050	Concrete	21.7	6.6
W-2850	Westmount	Westmount Place	1050	Concrete	21.7	6.6
W-2600	Westmount	Benbow Road	1050	Concrete	16.8	15
W-2400	Westmount	Upper Levels	1600	CSP	74.4	13.5
W-2000	Westmount	Cypress Bowl Road	1220	CSP	16.5	9.8
W-1700	Westmount	Cypress Bowl Road	600	CSP	32.2	22.3
W-1500	Westmount	Cypress Bowl Road	600	-	30.2	9.9
W-1200	Westmount	Cypress Bowl Road	600	-	46.1	6.5
W-800	Westmount (east)	Upstream of Upper Levels	900	-	87.2	15.5
W-400	Westmount (east)	Deer Ridge Drive	600	-	16.4	12.2
W-100	Westmount (east)	Cypress Bowl Road	600	-	13.2	3.8
Pipe Creek						
P-10700	Pipe	Seawall	1820 x 1820	Concrete	21.3	10.8
P-10500	Pipe	British Columbia Railway	1800	CSP	35.0	17.1
P-10300	Pipe	Marine Drive	1200 x 850	Concrete	20.3	12.7
P-10100	Pipe	Mathers Avenue	1200	Concrete	17.4	3.8
P-9800	Pipe (west)	Rosebery Avenue	1220	Concrete	12.3	27.7
P-9600	Pipe (west)	Spencer Place	1800	Concrete	16.1	7.1
P-9400	Pipe (west)	Spencer Drive	1050	Concrete	19.1	3.5
P-9200	Pipe (west)	Spencer Court	1500	Concrete	31.8	9
P-9000	Pipe (west)	Upper Levels	1500	CSP	56.1	11.4
P-8800	Pipe (west)	Cypress Bowl Road	1200	CSP	51.5	12
P-8300	Pipe (west)	Cypress Bowl Road	750	CSP	24.6	4.8
P-7800	Pipe (west)	Cypress Bowl Road	600	CMP	18.4	3.7
P-7200	Pipe (west)	Cypress Bowl Road	900	-	19.9	30.2
P-6600	Pipe (west)	Cypress Bowl Road	900	-	32.9	3
P-6100	Pipe (west)	Cypress Bowl Road	1150	CSP	23.3	7.9
P-6150	Pipe (west)	Cypress Bowl Road	600	CSP	23.6	7.3
P-5800	Pipe (east)	Rosebery Avenue	1220	Concrete	26.0	12.4
P-5500	Pipe (east)	Spencer Drive	1510 x 2000	Concrete	13.1	1.1
P-5300	Pipe (east)	Upstream of Spencer Drive	1200	-	8.8	14.1
P-5100	Pipe (east)	Gisby Street	1050 x 1050	Concrete	14.2	5
P-4900	Pipe (east)	Upper Levels	750 x 750	Concrete	111.6	22.7
P-4950	Pipe (east)	Upper Levels	920 x 920	CSP	113.0	22.5
P-4700	Pipe (east)	Cypress Bowl Road	900	CSP	90.1	32.1
P-4400	Pipe (east)	Cypress Bowl Lane	600	-	19.6	23.7
P-4200	Pipe (east)	Cypress Bowl Lane	600	-	8.5	8.9

TABLE 5-1 (cont'd.)
HYDRAULIC STRUCTURE INVENTORY – ALL CREEKS

Culvert	Watercourse	Location	Size Dia. or WxH (mm)	Material and Type	Length (m)	Slope (%)
P-3800	Pipe (east)	Cypress Bowl Road	600	-	28.8	22.4
P-3200	Pipe (east)	Cypress Bowl Road	600	-	40.1	31.2
P-2800	Pipe (east)	Cypress Bowl Road	750	CMP	20.7	3.8
P-2500	Pipe (middle)	Spencer Drive	850 x 850	Concrete	13.1	0.7
P-2300	Pipe (middle)	Spencer Court	750	Concrete	14.8	4.2
P-2100	Pipe (middle)	Upper Levels	750	CSP	80	16.4
P-2150	Pipe (middle)	Upper Levels	750	CSP	80	15.6
P-1900	Pipe (middle)	Cypress Bowl Road	900	CSP	60.8	18.8
P-1700	Pipe (middle)	Cypress Bowl Lane	900	CSP	19.7	4.1
P-1300	Pipe (middle)	Cypress Bowl Road	600	-	18.4	7.8
P-1000	Pipe (middle)	Cypress Bowl Road	600	-	18.7	6.1
P-800	Pipe (middle)	Upstream of Spencer Drive	700	Concrete	22.1	9.3
P-600	Pipe (middle)	Spencer Court	750	Concrete	14.8	3.8
P-400	Pipe (middle)	Upper Levels	1050	CSP	61	13.1
P-200	Pipe (middle)	Cypress Bowl Road	600	CSP/Concrete	84.1	21.9

5.1.2 Environmental Inventory

Godman, Turner, Cave, Westmount and Pipe Creeks are located within two biogeoclimatic subzones. The lower portions of the watersheds (below approximately 200 meters elevation) are within the Very Dry Maritime subzone (CWHxm1) and the upper portions are within the Dry Maritime subzone (CHWdm). CWHxm1 and CHWdm are subzones of the Coastal Western Hemlock zone (SLR, 2009).

A preliminary investigation of air photographs and creek profile photographs was compiled through data obtained from SLR, Golder, the District of West Vancouver, and from field work by Opus DK. An inventory of available creek characteristics is included in Appendix G.

5.1.3 Erosion Sites

Erosion is noted as a long-standing problem in these creeks and was noted in previous reports (D&K, 1973). Land erosion is a natural process, which is continually occurring and is exacerbated by sub-division development and/or increased flows from regularly occurring storm events or extreme events. In West Vancouver the steep grades within the watersheds produce high velocities of flows which can cause rapid erosion of some creek beds, depending on the soil conditions. This lowering of the creek bed results in slope instabilities and occurrences of slides in highly developed areas, and may cause property damage. The rate of erosion is unclear.

5.2 Existing Drainage Problems

Based on discussions with the District of West Vancouver, it was understood that no municipal records have been kept to identify past drainage problem areas within the selected watersheds.

Through field observations and dialogue with property owners, Opus DK identified one location with existing drainage problems. The area located is the roundabout of Cedaridge Place in the Turner Creek Watershed – driveway culverts in this area are undersized and overflow during large storm events. The property owner has experienced flooding on his property.

Aqua-Tex have conducted a review of the creek sections in the upper lands above Highway One. The review assesses the watershed conditions for proper functioning. The executive summary of the Aqua-Tex Report is included in Appendix M.

5.3 Existing Operation and Maintenance

The Operation and Maintenance of a storm water system are vital in ensuring the proper functioning of all components to their intended use. The District of West Vancouver does not have a published maintenance manual. District Operations staff provided the following details for this report of their storm water system maintenance program:

1) Roughly half of the District’s catch basins are cleaned each year. Catch basins identified as West Nile hazards (approximately 1000 catch basins) are cleaned yearly. To avoid leaf and tree debris accumulation in catch basins, the District’s regular street cleaning is increased in the fall.

2) Creeks are inspected yearly. Once a year, District staff walk the urban area creeks looking for debris, undermined roots/trees on the creek banks etc. and note necessary repairs or removal. Detention basins are cleaned out yearly. Screens are cleaned weekly from fall through spring and bi-monthly in the summer.

3) Culvert headwalls are maintained reactively. Public complaints, e.g. blockages, soggy lawns etc., are addressed as they are received and dealt with on an individual basis.

5.4 Hydrotechnical Assessment of Existing Conditions

The existing drainage system was simulated using the calibrated PCSWMM model for the design storms to assess current capacity problems, and to identify areas where flood risk mitigation is required. The flood risk within the Godman, Turner, Cave, Westmount, and Pipe Creek watersheds was previously documented in the 1973 Drainage Survey prepared by Dayton & Knight Ltd.

The following sub-sections describe the results of the hydrotechnical assessment. The locations of the hydraulic structures discussed are illustrated on Figures 5-1A and 5-1B.

5.4.1 Detention Pond Assessment

One detention pond lies within the Westmount Creek watershed, which is located above the Upper Levels Highway downstream of recently developed subdivisions. It was assumed these facilities were designed to reduce the 100-year post-development peak flow to the pre-development 10-year peak flow in accordance with the District’s Upper Lands Policy. This design philosophy may attenuate peak flows but will not necessarily provide an environmental benefit to the downstream creeks.

Model results showed that the existing pond is adequately sized to attenuate the 100-year peak flows. The pond should be maintained in its existing configuration because it reduces peak flows conveyed to the lower portion of the Westmount watershed.

5.4.2 10-Year Peak Flow Analysis

The existing drainage infrastructure is not able to convey the 10-year peak flows. Results of channel and culvert assessments are shown in Appendix J and K. From the upper levels Highway to the outfalls at Burrard Inlet, five channels and two culvert sections are undersized for the 10-year flows.

The model shows that the channel sections T-2400 in Turner Creek G-1700, G-1800 and G-2800 in Godman Creek and C-3200 in Cave Creek are undersized during the 10-year peak loading conditions. However, flooding may or may not occur if there is additional capacity along the creek (i.e. freeboard). These channel sections should be considered for improvement through channel widening or slope restoration by the District. All of these channels should be monitored, and where constrictions in flow are demonstrated, should be corrected as appropriate. Culvert capacities should also be addressed through infrastructure upgrades.

5.4.3 25-year, 50-year and 100-year Peak Flow Analysis

The existing drainage infrastructure was assessed for subsequent return periods, 25-year, 50-year, and 100-year return periods. Results for all return periods assessed are shown in Appendix J and K, Tables J-1 and K-1. From the Upper Levels Highway to the outfalls at Burrard Inlet: thirteen channels and nine culverts are undersized for the 25-year flows, fourteen channels and twelve culverts are undersized for the 50-year flows; and nineteen channels and fourteen culverts are undersized for the 100-year flows. As the return period and intensity of the design event increases the amount of deficient drainage infrastructure increases notably.

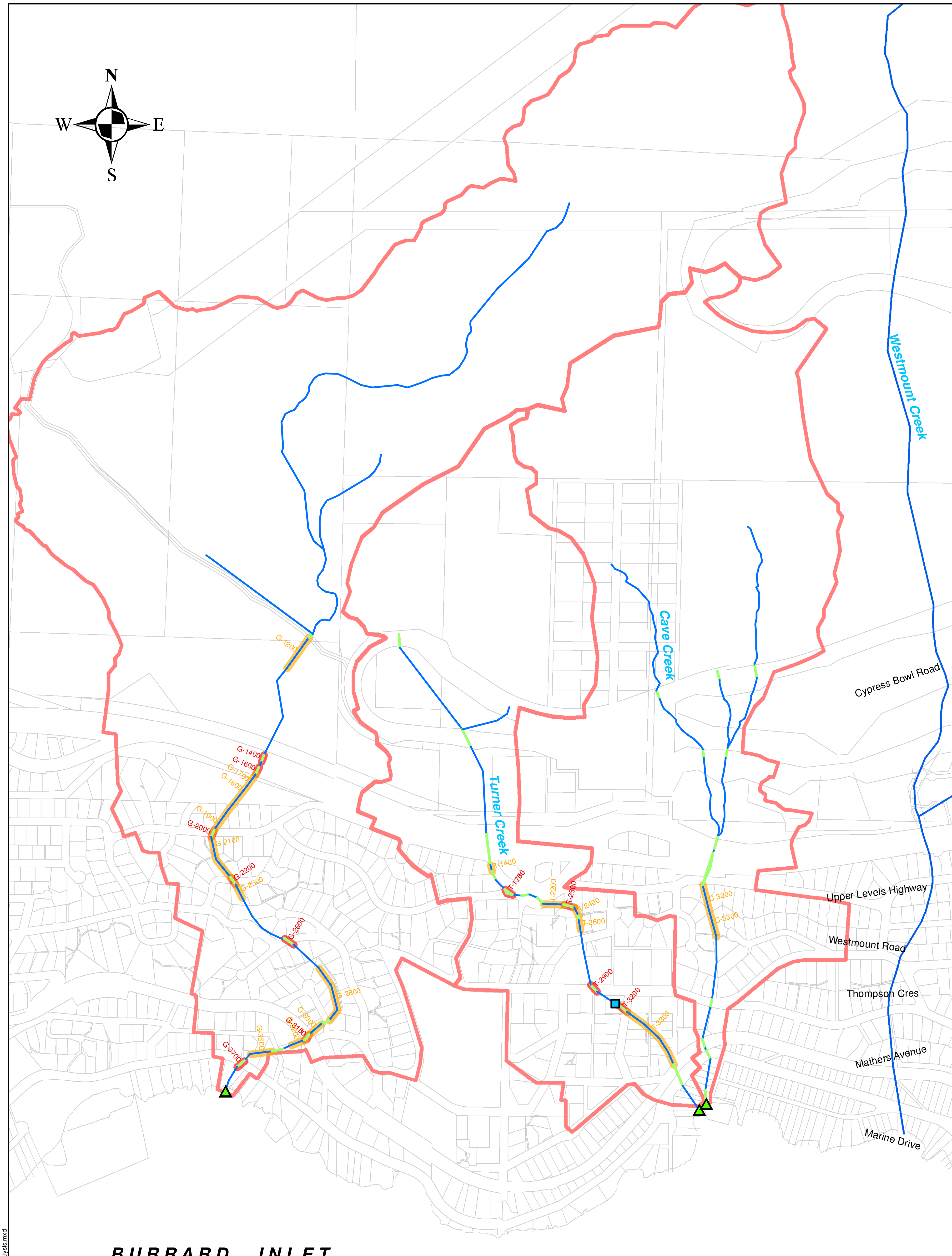
5.4.4 Creek Channel Assessment

In the absence of a high flow diversion or upland detention basins, the Godman, Turner, Cave, Westmount and Pipe Creek channels must safely convey the 200-year peak runoff flow. The channel capacity was determined using the Manning's equation based on the assumed channel cross-sections used in the model. The capacity was then compared to the estimated 200-year peak flow for each channel segment. There are 173 channel sections in the five creeks, of which 24 are under capacity. These inadequate channel sections were noted within each of the five creeks, below the Upper Levels Highway. The results of the channel capacity assessment are included in Appendix J and are shown in Figures 5-2A and 5-2B.

For environmental protection, channels were also assessed on existing flow velocities at 50% of the Mean Annual Rainfall. These frequently occurring rainfall events are responsible for greater than 90% of the annual runoff and hence contribute significantly to the erosion within a stream channel. Velocities during these events are known as Frequent Event Velocities (FEV). Maximum Permissible Velocities (MPV's) were developed for each channel reach as per the criteria outlined in Section 4.1.2. The MPV's and FEV's for each channel are given in Appendix J, Table J-3. The channel FEV's were then compared to the MPV's. Six channels in Godman Creek, two channels each from Cave and Westmount Creeks and seven in Pipe Creek are shown to have FEV's exceeding their MPV's. The FEV analysis is discussed further in Section 5.5.3.4 Stream Bank Protection.

5.4.5 Culvert Assessment

The 200-year return period design flows were used to assess the existing conveyance system. The assessment was based on examining the capacity of each structure under unrestricted 200-year peak flow (i.e., no constrictions in the system to attenuate flows and



Inadequate Hydraulic Structures	G-1400, G-1600, G-2000, G-2200, G-2600, G-3100, G-3150, G-3700, T-1700, T-1750, T-2300, T-2900, T-3200
Inadequate Channel Sections	G-1200, G-1700, G-1800, G-1900, G-2100, G-2300. G-2800, G-3000, G-3200, G-3500, T-1400, T-2200, T-2400, T-2600, T-3300, C-3200, C-3300

LEGEND

- Watershed Boundary
- Creek
- Major Culvert
- Storage Node
- Inadequate Channel Sections
- Inadequate Hydraulic Structures

0 100 200 400 600 800 Meters

SCALE 1:8,500

Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 5-2A - 200 Year Analysis.mxd

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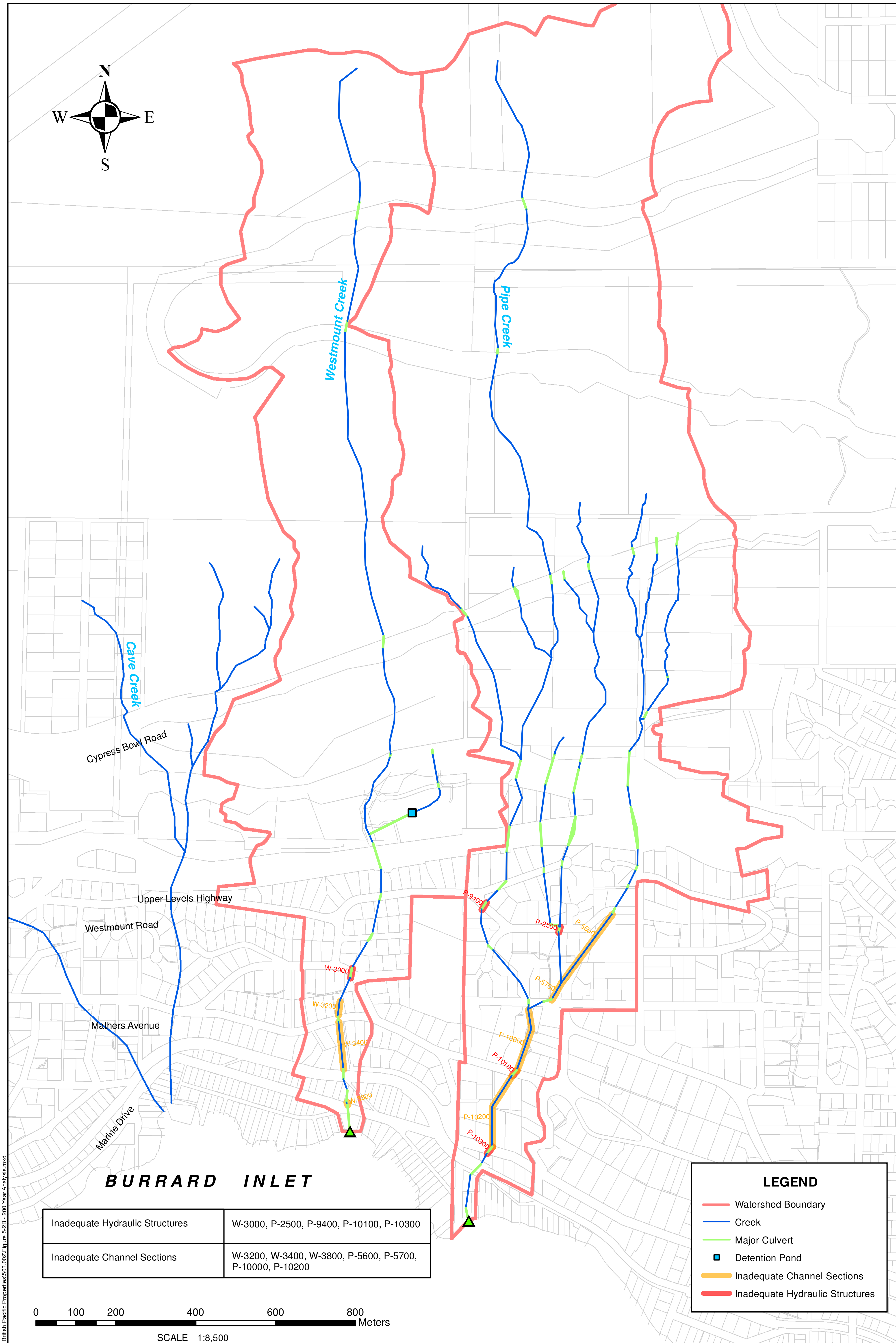
GODMAN, TURNER, CAVE, WESTMOUNT AND PIPE ISMP
HYDROTECHNICAL ASSESSMENT - EXISTING CONDITIONS
200-YEAR STORM ANALYSIS

FIGURE 5-2A



Inadequate Hydraulic Structures	W-3000, P-2500, P-9400, P-10100, P-10300
Inadequate Channel Sections	W-3200, W-3400, W-3800, P-5600, P-5700, P-10000, P-10200

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HYDROTECHNICAL ASSESSMENT - EXISTING CONDITIONS
200-YEAR STORM ANALYSIS

FIGURE 5-2B

Path: H:\Projects\500\503 British Pacific Properties\503.002\Figure 5-2B - 200 Year Analysis.mxd no flow diversion or detention facilities in operation). The culverts assessed were required to convey the 200-year flow without surcharging to be considered adequate.

Culvert analysis involves determining whether the flow condition within the culvert is inlet or outlet controlled during peak events. Inlet-controlled flow occurs when the culvert inlet capacity is less than that of the culvert itself. The headwater at the culvert entrance must then increase to force the flow through the culvert entrance. Outlet-controlled flow occurs when the capacity of the culvert is less than that of the capacity of the culvert inlet structure. The tailwater elevation and energy losses through the culvert determine the culvert capacity. PCSWMM uses the dynamic wave equation to compute the dynamic headwater and tailwater elevations during the simulated storm event. Thus, depending on the culvert type, geometry, and upstream and downstream conditions, the culvert experiences inlet- or outlet-controlled flows at various times during the simulation.

The results of the culvert assessments are summarized in Appendix K. There are 94 structures in the Godman, Turner, Cave, Westmount and Pipe Creek systems, of which 18 are estimated to be under capacity. These culverts include those listed in the 10-year peak flow analysis. Figures 5-2A and 5-2B show the inadequate structures.

5.5 Mitigation Options

The Integrated Stormwater Management Plan (ISMP) is designed to facilitate stormwater-sensitive land development and redevelopment, protect life and properties from flood and erosion hazards, and maintain public safety through creek management.

The Godman/Turner/Cave/Westmount/Pipe Creek ISMP consists of a number of components including:

- Flood Protection
- Environmental Protection

- Diversion Pipe Construction
- Streambank Protection
- Individual Lot Development Guidelines

This section identifies the flood protection level for protection of property and defines the minor and major drainage components that are required to meet ultimate development, unless otherwise stated. The improvements form the ISMP. Each improvement is presented in a table, given an identifier and illustrated on a plan of the study area to show location.

Flooding is a major risk to the lower reaches of the creeks due to limited channel and culvert capacities, and the close proximity of high density developments in West Vancouver.

Flood protection for life and property (major drainage system) is to be provided by ensuring adequate conveyance for the 200-year return period through a combination of the construction of the diversion and improvements to the existing drainage infrastructure.

For protection from nuisance flooding (minor drainage systems), the 10-year storm was selected for design.

Environmental impacts such as increased channel erosion with the proposed new developments on the Godman/Turner/Cave/Westmount/Pipe Creeks can be mitigated by reducing flows through diversion and reducing peak flows through low impact development (LID) techniques for residents of the respective watersheds. These measures will also help to minimize erosion associated with low flow events.

5.5.1 Stormwater Management Options for Protection of Life and Property

Analysis was undertaken to analyze stormwater management upgrade options to mitigate the effects of new development in the Godman, Turner, Cave, Westmount, and Pipe Creek watersheds. These options will provide attenuation of the peak flows in the existing drainage system such that the 200-year flows are adequately serviced and flood risks are minimized.

Potential solutions include increasing conveyance capacity of the existing creeks, bypassing excess flows to Burrard Inlet, and attenuating peak flows with detention storage facilities. Since the creeks run directly through private property, the work to increase conveyance capacity would be difficult.

Modeling results are given for both major and minor drainage. Drainage analysis was undertaken with the PCSWMM model (see Section 4 Model Development and Calibration). Appendices J and K contain the PCSWMM model results.

The model was run for a 10 year storm event on the minor drainage system. The 200-year storm event was used in sizing the creek drainage system works with the detention storage and diversion pipe solutions.

5.5.1.1 Detention Storage

Attenuation of peak flows through the use of detention storage facilities can reduce peak flows to the lower reaches of drainage networks and to provide protection to drainage infrastructure downstream. Storage facilities are designed to provide a controlled release of flow into the drainage system and to limit peak flows and velocities to channels and culverts such that the velocities are below the maximum permissible values and flows do not cause infrastructure downstream to overflow.

Detention storage was considered as a potential solution for stormwater management in the Godman, Turner, Cave, Westmount and Pipe Creeks. A preliminary analysis included sizing detention ponds at the upper reaches of the creeks at or above the Upper Levels Highway to attenuate flows. These detention storage facilities were modelled to be 1 metre in depth (for safety) and attenuate the 200-year post development flows to flows which ensure downstream infrastructure are protected while maintaining baseflow. The 24 hour, 200-year storm event proved critical in sizing the detention storage facilities.

The required areas for detention storage are summarized in Table 5-2. Appendix L contains the critical output hydrographs for the 200 year storm at the eight detention facilities.

TABLE 5-2
REQUIRED DETENTION STORAGE AREAS

Location	Downstream Node	Required Detention Storage Area (m^2)	Flow Released (m^3/s)
Godman	G-110	270,000	1.39
Turner West Branch	T-30	60,000	0.45
Cave	C-290	130,000	1.28
Westmount	W-240	110,000	0.67
Pipe West Branch	P-900	30,000	0.35
Pipe Middle Branch	P-40	5,000	0.07
Pipe East Branch 1	P-210	140,000	1.21
Pipe East Branch 2	P-490	150,000	1.16

A linear sequence of storage ponds is often considered to provide the cumulative total. This can prove to be disadvantageous if releases are not synchronized to avoid a recurrence of a new peak. Due to the steep mountainous profile of the upper reaches of the creeks and the large areas required for storage, the use of detention storage facilities was deemed impractical.

5.5.1.2 Flow Diversion

Re-routing of peak flows to avoid restricted outflows such as confined channels or high freshet backwater may provide the necessary protection against flooding. Flow diversion can permit base flows to continue in the existing stream channels, but prevent peak storm runoff flows from entering. Peak flows are typically re-routed at a diversion structure into a storm sewer interceptor, which discharges into a lower reach or other receiving water. Diverted discharges may flow in open channels or closed conduits. In steep terrain, energy dissipaters are required, either as waterfalls in the open channels or as sections in the closed conduits where hydraulic jumps are allowed to occur.
Stormwater diversions are not intended to capture base flows or runoff from frequent low intensity rainfalls. The management of these flows is discussed in Section 5.5.3.1 related to Low Impact Development.

The solution for bypassing excess flows and allowing baseflows to remain in the creek is to be provided through the construction of a pipe which diverts 200-year flows from the mountainside and transports the flows to Burrard Inlet.

For the diversion pipe solution, the 2-hour 200-year storm was used, since the event proved critical in the sizing of the minor and major drainage system.

A general design concept for the diversion pipe is illustrated in Figures 5-3A and 5-3B. The diversion pipe will have two branches at its upper reaches. The west branch will collect flows from Godman, Turner, and Cave Creeks above and along the Upper Levels Highway. The pipe then connects with the eastern branch of the diversion pipe at the Westmount Road off ramp from the Upper Levels Highway. The eastern branch collects a number of the Pipe Creek tributaries along the Upper Levels Highway and crosses the Highway at the Westmount Road off ramp, after which it connects with the western branch. The pipe then runs along Westmount Road, Benbow Road, Thompson Crescent and Mathers Avenue until it reaches 31st Street, which it follows until the outfall at Burrard

Inlet. The alignment shown in the figures is preliminary and may change due to property issues, right-of-ways, etc.

The construction of the diversion pipe is an integral part of the ISMP. The diversion provides flood and major event erosion protection, and environmental and minor event erosion protection to the drainage system it protects downstream.

The following four scenarios were developed to compare diversion requirements under four possible flood protection methodologies:

- Scenario 1 – Diversion for Existing Conditions Only
- Scenario 2 – Diversion for Post-Development Conditions above Highway One
- Scenario 3 – Diversion for Post-Development Conditions above Highway One with a 25% increase in impervious area to the developed lands below Highway One.
- Scenario 4 – Diversion for Post-Development conditions above Highway One, but only diverting flows greater than the 25-year flow.

The locations of the diversion structures are illustrated on Figures 5-3A and 5-3B.
The diversion pipe in Scenarios 1 to 3 is sized to accommodate the full 200-year flows. This design allows for the safe passage of the 200-year flow even with the low-flow by-pass into the creek blocked at the diversion inlet by potential debris carried down during a large storm event. This safety factor will help to ensure protection of life and property downstream of the diversion inlet structures and provide allowance for increased flow due to climate change.

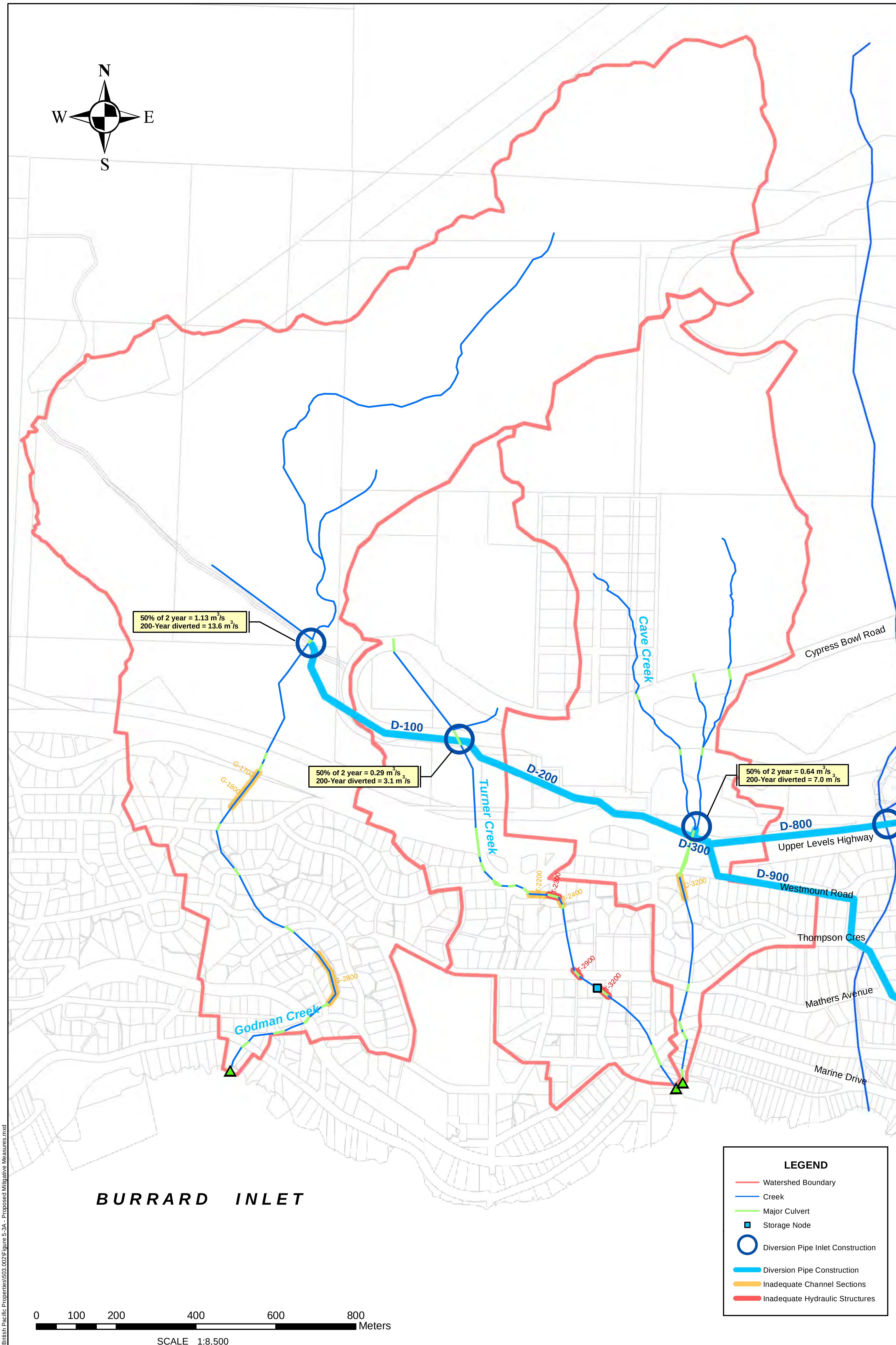
The objective of Scenario 3 is to illustrate the effect of increasing the density (i.e. imperviousness) of the existing developed areas below Highway One. This assumed increase in density has little effect on the size of the diversion pipe because it is below the diversion structures. It does however increase the runoff from the areas below the diversion structure. The scenario shows how the resultant flow increase impacts downstream erosion, culvert capacity and potential flooding and how the upstream diversion helps to mitigate these impacts.

The diversion pipe in Scenario 4 is sized for the 200-year storm event, but only for flows greater than the 25-year design flow. For Scenario 4, during the 200-year storm event, flows up to the 25-year level will remain in the creek. The objective of Scenario 4 is to determine the level of impact on the downstream infrastructure should a larger amount of the post-development flows continue to be conveyed by the natural creek system. The sizing of the pipe for this scenario shows the reduction in pipe size and potential cost savings by allowing the 25-year flows to remain in the creek. If significant upgrades are required to convey this higher flow then the cost savings associated with a reduced diversion pipe size may be negated.

5.5.1.3 Diversion Inlet Design

The diversion system is intended to divert flows from small frequently occurring flow events in addition to diverting flows during high flow events. Baseflows set at 50% of MAR (50% of 2-year flows) are to be maintained in the creeks. For environmental protection, and to ensure the system is well maintained through adequately sized flushing events, post-development frequently occurring flow events (above the baseflow level) up to and including the 10-year return period are to be partially diverted out of the creek (e.g. by using a V-notch weir). High flows however, will be conveyed mainly by the diversion with a portion of the flows remaining in the creek. High flows occur infrequently and do not contribute to erosion as much as smaller more frequent storms. While the diversion pipe has been sized to handle the full 200 year flow, the exact magnitude of the flows to be diverted and to remain in the creek during high flow events is to be determined during detailed design.

The diversion structure is sized such that, under normal operating conditions, up to 50 % of the 2-year flow (baseflow) is to remain in the creek with no water being diverted. The diversion structure was simulated as a compound orifice / v-notch / broad crest weir where



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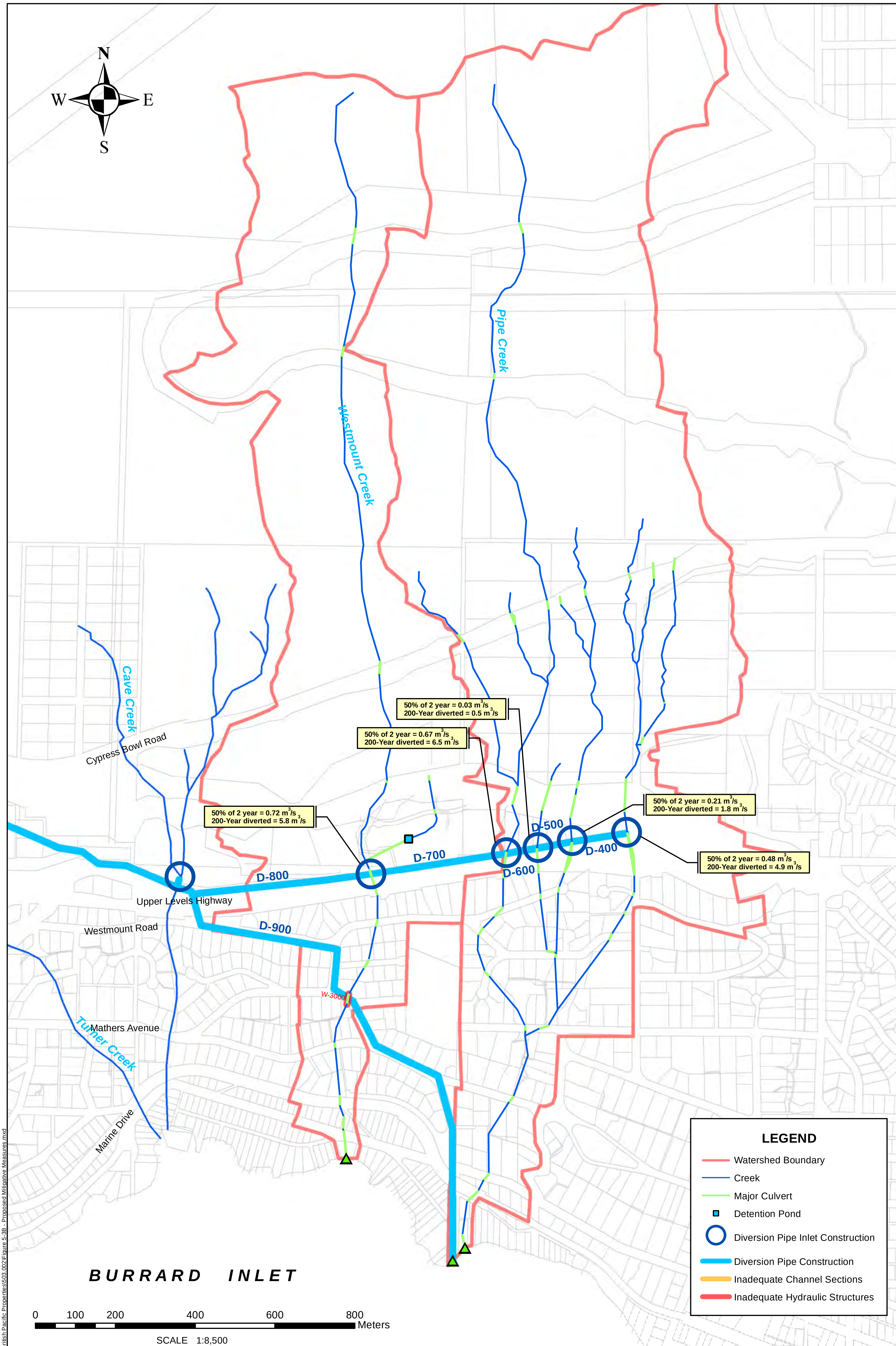
HYDROTECHNICAL ASSESSMENT - FUTURE CONDITIONS
PROPOSED MITIGATIVE MEASURES - SCENARIO 2

FIGURE 5-3A



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HYDROTECHNICAL ASSESSMENT - FUTURE CONDITIONS
PROPOSED MITIGATIVE MEASURES - SCENARIO 2

FIGURE 5-3B

the baseflow is conveyed by an orifice and the diverted water is conveyed by the weirs. General assumptions were made in the model to simulate this effect.

The design of the infrastructure surrounding the diversion is critical in providing adequate flows and protection to the inlets within the mountainous creeks of West Vancouver. Due to potentially high kinetic energies around the inlets as water flows along the creeks, the diversion inlets should have wide mouths to capture flows into the diversion pipe.

Due to the high velocities and high potential of carrying natural materials down the steep mountainous terrain, protection of the diversion inlets and creek control structures are required through the construction of rock pits, ‘grizzly’ screens, and rock traps directly upstream of the inlets. A ‘grizzly’ screen with 0.5 meter bar spacing should be adequate to prevent trees, vegetation, and large boulders from traveling downstream and causing obstruction or damage to the inlets and control structures. A road should also be constructed for district crews to access and clean out the screens and rock pits after every high flow event.

Examples of inlet protection have been used in the Marr, Lawson, and McDonald Creeks for stream and diversion protection. Appendix N contains previous Opus DK designs for diversion and diversion inlets in nearby streams. These should be discussed with DWV operators for improvements and approach to the design.

5.5.1.4 Scenario 1 – Diversion for Existing Conditions Only

This diversion scenario is sized to capture existing flows from catchments above the highway to divert most of the 200-year flood waters out of the system and maintain minimum base flows in the creek. This scenario assumes no new development in the watersheds.

The diversion inlets should be configured to divert flows as described in Table 5-3. See Appendix P for a detailed schematic of the modeled diversion system and creeks.

Included in Appendix P are flow tables showing the simulated flows in the creek above the diversion, below the diversion, within the diversion, and at the outlet of the creek during the 200 year event.

TABLE 5-3
CRITERIA FOR DIVERSION INLETS – SCENARIO 1

Inlet	Pre-development 50% of 2-Year	Pre-development 10-Year	Pre-development 200 Year	Diverted 200-Year	Remaining in Creek During 200 –Year
	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)
Godman Inlet	1.1	6.0	14.6	11.8	2.2
Turner Inlet	0.3	1.4	3.5	2.8	0.5
Cave Inlet	0.6	2.3	6.9	5.5	1.1
Westmount Inlet	0.7	2.6	6.8	5.6	1.1
Pipe West Branch Inlet	0.7	3.7	7.4	6.2	1.0
Pipe Middle Branch Inlet	0.03	0.2	0.5	0.5	0.05
Pipe East Branch 1 Inlet	0.2	1.0	2.0	1.7	0.3
Pipe East Branch 2 Inlet	0.5	2.2	5.6	4.7	0.85

Table 5-4 provides the approximate pipe sizes for each section of the diversion pipe under Scenario 1.

TABLE 5-4
DIVERSION PIPE SIZING – SCENARIO 1

	D-100	D-200	D-300	D-400	D-500	D-600	D-700	D-800	D-900
Size	1800	1350	1500	1050	1200	1350	1350	1800	1800x1800

5.5.1.5 Scenario 2 – Diversion for Post-Development Conditions above Highway One

This diversion scenario is sized to capture future, post-development flows from catchments above the highway to divert most of the 200-year flood waters out of the system and maintain minimum baseflows in the creek. This scenario assumes new development in the watersheds with the future land use assumptions as outlined in Section 2.2.

The modeled flow results from Scenario 2 (post-development) do not appear to vary significantly from the results of Scenario 1 (pre-development). This is not to suggest that the proposed development has no effect on the watershed runoff, rather it suggests that other factors are more significant when simulating such an intense storm in these watersheds. Some of these factors could include:

- the relatively small area of each watershed being redeveloped when compared to the entire watershed;
- the size of the storm will overwhelm the ability for natural and man-made processes to reduce runoff as the peak of the storm passes; and
- the steep grades that result in a rapid response time which when combined with the reduction in attenuation opportunities reduces the difference between pre and post development.

Table 5-5 shows the inlet configurations under Scenario 2. See Appendix P for a detailed schematic of the modeled diversion system and creeks. Included in Appendix P are flow tables showing the simulated flows in the creek above the diversion, below the diversion, within the diversion, and at the outlet of the creek during the 200 year event.

TABLE 5-5
CRITERIA FOR DIVERSION INLETS – SCENARIO 2

Inlet	Pre-development 50% of 2-Year (m^3/s)	Post-development 10-Year (m^3/s)	Post-development 200 Year (m^3/s)	Diverted 200-Year (m^3/s)	Remaining in Creek During 200 –Year (m^3/s)
Godman Inlet	1.1	6.5	15.4	12.4	2.2
Turner Inlet	0.3	1.4	3.5	2.8	0.5
Cave Inlet	0.6	3.4	6.9	5.5	1.1
Westmount Inlet	0.7	2.7	6.8	5.6	1.1
Pipe West Branch Inlet	0.7	3.7	7.4	6.2	1.1
Pipe Middle Branch Inlet	0.03	0.3	0.5	0.5	0.05
Pipe East Branch 1 Inlet	0.2	1.0	2.0	1.7	0.3
Pipe East Branch 2 Inlet	0.5	2.3	5.6	4.7	0.8

Table 5-6 provides the approximate pipe sizes for each section of the diversion pipe under Scenario 2. While there are increases in the modeled flows from pre-development to post-development, they do not translate into a need for an increased pipe size for the diversion. For large diameter pipe the difference in capacity between standard pipe sizes can be significant in relation to the total pipe capacity. In this case the difference in flows is not enough to warrant an upgrade to the next standard pipe size.

TABLE 5-6
DIVERSION PIPE SIZING – SCENARIO 2

	D-100	D-200	D-300	D-400	D-500	D-600	D-700	D-800	D-900
Size	1800	1350	1500	1050	1200	1350	1350	1800	1800x1800

5.5.1.6 <u>Scenario 3 – Diversion for Post-Development Conditions above Highway 1 with a 25% increase in impervious area to the developed lands below Highway One</u>

The purpose of simulation Scenario 3 is to evaluate the potential impact of future development within the existing developed areas. Recent trends indicate that older smaller houses are being redeveloped with significant increases to site imperviousness.

This scenario assumes the same development conditions as Scenario 2 above the highway, which is where the diversion structures were simulated to be located. As such, there is no difference in the size of the diversion proposed for this scenario.

Similar to the conclusion drawn for Scenario 2, the effect of the increased percent imperviousness modeled in Scenario 3 is not indicated by the flows estimated at the outlet of the creek. This does not suggest that there will not be an impact to the creek systems or the minor system infrastructure due to the increased runoff for the less intense storms. Rather it suggests that the same factors as described above are also a factor under this scenario.

See Appendix P for a detailed schematic of the modeled diversion system and creeks. Included in Appendix P are flow tables showing the simulated flows in the creek above the diversion, below the diversion, within the diversion, and at the outlet of the creek during the 200 year event.

5.5.1.7 Scenario 4 – Diversion for Post-Development conditions above Highway One, but only diverting flows greater than the 25-year flow.

The purpose of Scenario 4 is to determine the impact to the system if the proportion of flow being bypassed in Scenario 2 is reduced thereby requiring the creek to convey more water. As this anticipates a decrease in the size of the diversion there could be substantial cost savings. However, the additional flow in the creek could also increase the amount of work required downstream. A review of the potential cost savings, additional costs, and impact to risk for private and public infrastructure can then be reviewed to determine an acceptable balance.

Table 5-7 shows the inlet configurations under Scenario 4. See Appendix P for a detailed schematic of the modeled diversion system and creeks. Included in Appendix P are flow tables showing the simulated flows in the creek above the diversion, below the diversion, within the diversion, and at the outlet of the creek during the 200 year event.

TABLE 5-7
CRITERIA FOR DIVERSION INLETS – SCENARIO 4

Inlet	Post-development 200 Year (m^3/s)	Diverted 200-Year (m^3/s)	Remaining in Creek During 200 – Year (m^3/s)
Godman Inlet	15.4	3.8	9.91
Turner Inlet	3.5	1.5	2.0
Cave Inlet	6.9	2.2	4.7
Westmount Inlet	6.8	2.6	4.2
Pipe West Branch Inlet	7.4	1.2	5.9
Pipe Middle Branch Inlet	0.5	0.1	0.4
Pipe East Branch 1 Inlet	2.0	0.5	1.5
Pipe East Branch 2 Inlet	5.5	1.5	3.9

As expected, the proposed diversion pipe is 1-2 pipe sizes smaller than the proposed pipe in Scenario 2. The approximate diversion pipe size is shown in Table 5-8. The additional downstream works that result from this scenario are presented below in Table 5-9. The number of works required downstream increases by a factor of 3.6 when compared with Scenario 2.

TABLE 5-8
DIVERSION PIPE SIZING – SCENARIO 4

	D-100	D-200	D-300	D-400	D-500	D-600	D-700	D-800	D-900
Size	1350	1050	1200	750	900	900	1050	1500	1500x1800

5.5.1.8 Additional Improvements

The diversion pipe as modeled will attenuate peak flows and resolve most of the inadequate channel and hydraulic structures downstream of the intakes. However, there are still culverts and channels, which remain hydraulically deficient and require improvements for handling the 200-year peak flows even with the diversion in place. The proposed alignment of the diversion pipe is such that the inlets for both the Turner and Godman Creeks are located above the Upper Levels Highway. The flow diverted at these locations is not always sufficient to solve all capacity issues downstream of the inlet. For the four diversion scenarios, the deficient channels are highlighted in Appendix J, Table J-2. Upon further investigation, these channel sections may not require improvement as flooding may or may not occur if there is additional capacity along the creek (i.e. freeboard) to convey flows. If needed, these channels are to be considered for improvement by the District through channel widening or slope restoration. Figures 5-3A and 5-3B show the inadequate channel sections under diversion Scenario 2.

For the four diversion scenarios, deficient culverts are highlighted in Appendix K, Table K-2. Capacity increases should be provided through culvert replacements or alternative solutions such as twinning. However, field investigations should be carried out at these locations to verify the size of the pipes and conditions of the culverts before replacement is confirmed. Figures 5-3A and 5-3B show the inadequate hydraulic structures under diversion Scenario 2.

The number of deficient channels and culverts below the highway is shown in Table 5-9 for each of the diversion scenarios. As expected, the number of deficient culverts and channels increases from pre-development (Scenario 1) to post-development (Scenarios 2 & 3), however, the increase is not significant because the increase in impervious area from the new development above the highway is small in relation to the overall catchment area. The 25% increase in impervious does not appear to significantly increase the number of deficient culverts and channels. This is likely because the areas below the highway are a small portion of the total catchment area. There is a significant increase in deficient culverts and channels in Scenario 4 compared to Scenarios 1-3. This is due to the increase in flows allowed to remain in the creek. Diverting only the flows above the 25-year flow decreases the required size of the diversion pipe, but also increases the number of deficient channels and culverts that require improvements.

TABLE 5-9
HYDRAULIC DEFICIENCIES WITH DIVERSION IN PLACE

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
No. of Deficient Culverts	3	4	4	16
No. of Deficient Channels	6	6	7	20

Several culverts located above the Upper Levels Highway have also been identified as under capacity during the 200-year storm event. However, these culverts act only as routes of conveyance in the model, and do not provide an accurate representation of the existing culverts above the Upper Levels Highway. Detailed analysis of these culverts is beyond the scope of this report. Culverts above the Upper Levels Highway were identified previously by Dayton and Knight Ltd. as requiring upgrades (DK 1973). These have been extensively reviewed by InterCAD Consulting Services Ltd. recently, including an inventory of all existing culverts and subsequent analysis. There is also a risk of creek flows 'jumping' catchments along Cypress Bowl Road as a result of culvert backups. This is not reviewed in this report, but should be considered in detailed design.

5.5.1.9 Diversion Options

Based on the four scenarios evaluated as part of this study, two diversion options have been identified to provide perspective for future watershed management decisions. The options are described below:

- Option A – Construct the diversion pipe as defined in Scenario 2. This diversion pipe would be sized for maximum risk aversion and would minimize the number of downstream works required.

- Option B - Construct the diversion pipe as defined in Scenario 4. This diversion pipe would be smaller than in Option A and hence less expensive to build initially. However, it would result in additional downstream works as well as the need to accept a higher risk of damages to private and public property.

5.5.2 Protection from Nuisance Flooding

The construction of the diversion pipe will address the smaller, more frequent 10-year storms. The remaining culvert and channel improvements below the Upper Levels Highway are as noted in the previous section.

5.5.3 Environmental Protection

5.5.3.1 Individual Lot Development Guidelines

Recognizing that the diversion manages peak flows and provides flood protection on the creek systems, it is still necessary to manage the low intensity, frequent storm flows through other stormwater management methods. The following guidelines are provided for the selection of Low Impact Development (LID) measures that promote environmentally sensitive development or redevelopment on individual lots in the Pipe, Westmount, Cave, Turner and Godman Creek drainage areas.

The January 2009 Golder Hydro-Geotechnical report identifies the geomorphologic and physical constraints associated with the drainage areas within the development study area. Conditions vary laterally across the study area and impact the selection of LID solutions that may be reasonably selected for specific developments. For example, the Pipe Creek catchment above the Upper Levels Highway in the development area represents a sheet flow run-off pattern, corresponding to a high drainage density, on steep slopes of 30% to 50% and through shallow deposits overlying bedrock. This compares to the Godman Creek catchment at the same contours below Eagle Lake Road and to the west, which collects drainage in low lying sandy alluvial deposits of about 10% gradient. These conditions offer considerably different degrees of LID opportunity for development to achieve low intensity rainfall collection and management.

The development of a planning tool that can be used to select LID's for the differing conditions in the five drainage areas that are impacted by the development is needed. The upper level development represents about 31 ha or 28% of the overall development study area of 111 ha. The upper level development represents about 5% of the full drainage area for the five creeks of 616 ha.

The LID measures are the first to collect and treat the excess surface runoff from a development, but are designed and constructed to manage low intensity, frequent storms only. The low intensity storms contribute to the greatest annual volume of runoff and their capture and management, through infiltration or other techniques, can improve stream interflow and reduce stream erosion. Excess runoff from more intense, less frequent storms that exceed the capacity of the LID systems are collected by minor or major storm collection systems described earlier in this section. These latter systems secure protection for life and property. All systems in this way contribute to the protection of the social-economic and environmental needs of the development and the community.

The selection of the best LID solutions will depend on local environmental constraints. Each development area of the five drainage basins will have different conditions under which the LID will function.

The five LID options shown in Table 5-10 were identified for their ability to satisfy the goals and constraints. Example design details for the LID measures are included in

Appendix O. These example designs were completed by the development industry independently for lands to be developed and are included for information only.

TABLE 5-10
LID OPTIONS CONSIDERED

Option	LID Options
1	Absorbent Soils
2	Permeable Pavers
3	Redirect Roof Leaders to Rock Pits
4	Rain Barrels
5	Wetland Infiltration Areas / Rain Gardens

The goals and constraints for LID measures within the developed area of each catchment are shown in Tables 5-11 through 5-15. Goals and constraints are rated equally in the five tables for Importance. The satisfaction that each option provides when evaluated for each creek drainage area is shown in the tables and recognizes the character of the development area in the drainage basin. The sum of the ratings gives the relative satisfaction of the LID for the respective development.

Table 5-16 provides the comparisons for the ranking LID and drainage areas. As shown, the use of LIDs in the Godman Creek basin can be seen to be more productive than the other basins since, the Godman basin development areas have higher scores. Similarly, the use of rain gardens (Option 5) appears to offer better opportunities and is of greater benefit for LIDs than rain barrels (Option 4).

TABLE 5-11
LID OPTION SCORING PIPE CREEK

No		Importance 1 low 9 high	Satisfaction of Objectives[1]									
			Option 1 Absorbent Soils		Option 2 Permeable Pavers		Option 3 Redirect Roof Leaders to Rock Pits		LID Option 4 Rain Barrels		Option 5 Wetland Infiltration Areas	
	Objectives		Rank	Product	Rank	Product	Rank	Product	Rank	Product	Rank	Product
1	Sustain seasonal flows in creek	5	5	25	5	25	4	20	1	5	8	40
2	Reduce bank erosion from low intensity storm flows	5	4	20	4	20	6	30	1	5	7	35
3	Mimic infiltration capacity of pre-development conditions	5	8	40	7	35	5	25	1	5	8	40
4	Protect aquatic life	9	3	27	3	27	3	27	1	9	8	72
5	Protect fisheries habitat	2	3	6	3	6	3	6	1	2	9	18
	Constraints											
1	Soil porosity and absorption capacity	2	5	10	5	10	6	12	3	6	6	12
2	Site gradients	2	6	12	6	12	7	14	7	14	1	2
3	Creek bank and base instability and erosion potential	4	5	20	5	20	7	28	1	4	8	32
4	Creek lateral movement potential	7	5	35	5	35	7	49	1	7	8	56
5	Permanence of Solution: landowner compliance; public ownership; enforcement	8	7	56	7	56	1	8	1	8	8	64
6	Cluster development restrictions	4	6	24	6	24	6	24	6	24	2	8
TOTAL				**275**		**270**		**243**		**89**		**379**

[1] Options are ranked from 1 to 10 based on ability to satisfy the watershed objectives for each specific watershed, with 10 being the highest score. The product is the importance x rank for each option and objective.

TABLE 5-12
LID OPTION SCORING WESTMOUNT CREEK

No		Importance 1 low 9 high	Satisfaction of Objectives[1]									
			Option 1 Absorbent Soils		Option 2 Permeable Pavers		Option 3 Redirect Roof Leaders to Rock Pits		Option 4 Rain Barrels		Option 5 Wetland Infiltration Areas	
	Objectives		Rank	Product	Rank	Product	Rank	Product	Rank	Product	Rank	Product
1	Sustain seasonal flows in creek	7	5	35	5	35	4	28	1	7	8	56
2	Reduce bank erosion from low intensity storm flows	5	4	20	4	20	6	30	1	5	7	35
3	Mimic infiltration capacity of pre-development conditions	6	8	48	7	42	5	30	1	6	8	48
4	Protect aquatic life	9	3	27	3	27	3	27	1	9	8	72
5	Protect fisheries habitat	2	3	6	3	6	3	6	1	2	9	18
	Constraints											
1	Soil porosity and absorption capacity	4	5	20	5	20	6	24	3	12	6	24
2	Site gradients	2	6	12	6	12	7	14	7	14	1	2
3	Creek bank and base instability and erosion potential	3	5	15	5	15	7	21	1	3	8	24
4	Creek lateral movement potential	4	5	20	5	20	7	28	1	4	8	32
5	Permanence of Solution: landowner compliance; public ownership; enforcement	8	7	56	7	56	1	8	1	8	8	64
6	Cluster development restrictions	5	6	30	6	30	6	30	6	30	2	10
TOTAL				**289**		**283**		**246**		**100**		**385**

1 Options are ranked from 1 to 10 based on ability to satisfy the watershed objectives for each specific watershed, with 10 being the highest score. The product is the importance x rank for each option and objective.

TABLE 5-13
LIO OPTION SCORING CAVE CREEK

No		Importance 1 low 9 high	Satisfaction of Objectives[1]									
			Option 1 Absorbent Soils		Option 2 Permeable Pavers		Option 3 Redirect Roof Leaders to Rock Pits		Option 4 Rain Barrels		Option 5 Wetland Infiltration Areas	
	Objectives		Rank	Product	Rank	Product	Rank	Product	Rank	Product	Rank	Product
1	Sustain seasonal flows in creek	7	5	35	5	35	4	28	1	7	8	56
2	Reduce bank erosion from low intensity storm flows	5	4	20	4	20	6	30	1	5	7	35
3	Mimic infiltration capacity of pre-development conditions	5	8	40	7	35	5	25	1	5	8	40
4	Protect aquatic life	9	3	27	3	27	3	27	1	9	8	72
5	Protect fisheries habitat	2	3	6	3	6	3	6	1	2	9	18
	Constraints											
1	Soil porosity and absorption capacity	4	5	20	5	20	6	24	3	12	6	24
2	Site gradients	2	6	12	6	12	7	14	7	14	1	2
3	Creek bank and base instability and erosion potential	4	5	20	5	20	7	28	1	4	8	32
4	Creek lateral movement potential	4	5	20	5	20	7	28	1	4	8	32
5	Permanence of Solution: landowner compliance; public ownership; enforcement	8	7	56	7	56	1	8	1	8	8	64
6	Cluster development restrictions	6	6	36	6	36	6	36	6	36	2	12
TOTAL				**292**		**287**		**254**		**106**		**387**

[1] Options are ranked from 1 to 10 based on ability to satisfy the watershed objectives for each specific watershed, with 10 being the highest score. The product is the importance x rank for each option and objective.

TABLE 5-14
LID OPTION SCORING TURNER CREEK

No		Importance 1 low 9 high	Satisfaction of Objectives[1]									
			Option 1 Absorbent Soils		Option 2 Permeable Pavers		Option 3 Redirect Roof Leaders to Rock Pits		Option 4 Rain Barrels		Option 5 Wetland Infiltration Areas	
	Objectives		Rank	Product	Rank	Product	Rank	Product	Rank	Product	Rank	Product
1	Sustain seasonal flows in creek	8	5	40	5	40	4	32	1	8	8	64
2	Reduce bank erosion from low intensity storm flows	5	4	20	4	20	6	30	1	5	7	35
3	Mimic infiltration capacity of pre-development conditions	5	8	40	7	35	5	25	1	5	8	40
4	Protect aquatic life	9	3	27	3	27	3	27	1	9	8	72
5	Protect fisheries habitat	2	3	6	3	6	3	6	1	2	9	18
	Constraints											
1	Soil porosity and absorption capacity	4	5	20	5	20	6	24	3	12	6	24
2	Site gradients	2	6	12	6	12	7	14	7	14	1	2
3	Creek bank and base instability and erosion potential	1	5	5	5	5	7	7	1	1	8	8
4	Creek lateral movement potential	4	5	20	5	20	7	28	1	4	8	32
5	Permanence of Solution: landowner compliance; public ownership; enforcement	8	7	56	7	56	1	8	1	8	8	64
6	Cluster development restrictions	7	6	42	6	42	6	42	6	42	2	14
TOTAL				**288**		**283**		**243**		**110**		**373**

1 Options are ranked from 1 to 10 based on ability to satisfy the watershed objectives for each specific watershed, with 10 being the highest score. The product is the importance x rank for each option and objective.

TABLE 5-15
LID OPTION SCORING GODMAN CREEK

No		Importance 1 low 9 high	Statement of Objectives[1]									
			Option 1 Absorbent Soils		Option 2 Permeable Pavers		Option 3 Redirect Roof Leaders to Rock Pits		Option 4 Rain Barrels		Option 5 Wetland Infiltration Areas	
	Objectives		Rank	Product	Rank	Product	Rank	Product	Rank	Product	Rank	Product
1	Sustain seasonal flows in creek	8	5	40	5	40	4	32	1	8	8	64
2	Reduce bank erosion from low intensity storm flows	9	4	36	4	36	6	54	1	9	7	63
3	Mimic infiltration capacity of pre-development conditions	9	8	72	7	63	5	45	1	9	8	72
4	Protect aquatic life	9	3	27	3	27	3	27	1	9	8	72
5	Protect fisheries habitat	9	3	27	3	27	3	27	1	9	9	81
	Constraints											
1	Soil porosity and absorption capacity	8	5	40	5	40	6	48	3	24	6	48
2	Site gradients	6	6	36	6	36	7	42	7	42	1	6
3	Creek bank and base instability and erosion potential	6	5	30	5	30	7	42	1	6	8	48
4	Creek lateral movement potential	3	5	15	5	15	7	21	1	3	8	24
5	Permanence of Solution: landowner compliance; public ownership; enforcement	8	7	56	7	56	1	8	1	8	8	64
6	Cluster development restrictions	4	6	24	6	24	6	24	6	24	2	8
TOTAL				**403**		**394**		**370**		**151**		**550**

[1] Options are ranked from 1 to 10 based on ability to satisfy the watershed objectives for each specific watershed, with 10 being the highest score. The product is the importance x rank for each option and objective.

TABLE 5-16
RANKING OF LID OPTIONS AND DRAINAGE AREAS

	Option 1 Absorbent Soils	Option 2 Permeable Pavers	Option 3 Roof Leaders to Rock Pits	Opption 4 Rain Barrels	Option 5 Wetland Rain Gardens	TOTAL	RANK
Pipe Creek	275	270	243	89	379	1256	5
Westmount Creek	289	283	246	100	385	1303	3
Cave Creek	292	287	254	106	387	1326	2
Turner Creek	288	283	243	110	373	1297	4
Godman Creek	403	394	370	151	550	1868	1
TOTAL	1547	1517	1356	556	2074		
RANK	**2**	**3**	**4**	**5**	**1**		

1 Is Highest Rank vs. 5 is Lowest Rank

5.5.3.2 LID Performance Target

To narrow the selection of LID measures for the development sites it is necessary to evaluate the LID performance in capturing rainfall volume from the lower intensity, frequent storms. Rainfall volume capture targets quantify this performance for the various LID measures. The BC Provincial Guidebook for Stormwater Planning (Reference) sets this target at 50% of the Mean Annual Rainfall (MAR) over a 24 hour period. Roughly 75% of rainfall events in a given year are at or below 50% of the MAR, so in capturing this amount, the majority of rainfall volume is allowed to infiltrate or evaporate and mimic the predevelopment hydrological process.

Because of the physical constraints in infiltrating rainfall within the Pipe to Godman watersheds, we recommend a rainfall capture target of 30% MAR. The capacity for infiltration is highly limited by the steep gradients, shallow bedrock or impermeable till and low hydraulic conductivity of the underlying soils. This is consistent with the current development and ISMP study in the neighbouring Rogers and Marr Creek watersheds where a 30% MAR capture target was deemed a more practical goal.

The methodology for calculating the MAR is given in the BC Provincial Guidebook for Stormwater Planning (Province of British Columbia, 2002) as follows:

1) Calculate the peak daily rainfall (24-hour rainfall depth) for each year of record from an appropriate rainfall gauge (nearby location with available historical data, preferably 30 years or more).
2) Rank the rainfall maxima from highest to lowest and calculate a return period (T) for each, using a standard plotting position formula (e.g. Weibull formula, T = [total # of rainfall maxima +1]/rank).
3) Create a logarithmic plot of rainfall maxima vs. return period.

4) From this plot determine the rainfall maxima with a 2-year return period. This is approximately equal to the MAR (the statistical definition of MAR is the rainfall with a 2.33 return period).

Determining the effective rainfall capture of the various LID measures is a simple volume calculation. For absorbent soils, permeable pavers or rock pits, the rainfall capture is calculated by the following formula:

$$Vc = A \times d \times p$$

Where:

Vc = the effective capture volume
A = the area covered by the absorbent soil / pavers / rockpit
d = the depth of the absorbent soil / pavers / rockpit
p = the effective porosity of the material

For larger wetland or infiltration areas the volume captured is simply the effective storage capacity of the facility. For larger sites, the volume captured in rain barrels is likely to be negligible in relation to the total rainfall volume, making rain barrel use a more practical solution for sustainable practice than for LID solutions in a high rainfall area.

The cumulative volume captured by the various LID measures is then divided by the total site area to determine the effective capture in millimetres. This effective capture should be at or above 30% of the MAR. The total site area is defined as the developed area. Undeveloped areas are not considered in the calculation as these are deemed neutral and unchanged from predevelopment conditions.

Other site specific opportunities for infiltration should also be explored when meeting the capture targets. For example, in the Godman Creek watershed there is opportunity for a larger wetland facility in the flat areas to the west of the first Cypress Bowl switchback and directly south of the Eagle Lake access Road.

5.5.3.3 Recommendations for LIDs

The selection of LID management solutions for the upper level development areas should follow the following five categories reflecting the opportunity and practicality of their use:

1. Select a 30% MAR for the low intensity frequent rainfall LID capture

2. Select the appropriate LID for the development that reflects a practical solution fitting the geology and topography of the area

3. Select LID use as a means of greatest value for ISMP optimization from best to least as follows:

    a. Rain gardens and wetland solutions Option 5
    b. Absorbent soils, Option 1
    c. Permeable pavers, Option 2
    d. Roof leaders to rock pit, Option 3

4. The basins in which the LIDs should be developed for greatest effectiveness for low intensity frequent storms, from best to least are:

    a. Godman
    b. Cave
    c. Westmount
    d. Turner, and
    e. Pipe

The above ranking should be considered as a general guide. Site specific opportunities for LID implementation, such as the large flat area at the District Operations Centre in the Turner Creek catchment should be considered during detailed design.

5.5.3.4 Stream Bank Protection

The proposed diversion will minimize the erosive impact for a range of flows from frequently occurring events to extreme events. Stream bank erosion should be monitored over time to evaluate the effectiveness of the diversion on the reduction of erosion.

Modeling of the diversion shows that there are capacity problems for some of the creek channel sections. The current alignment of the diversion pipe is such that the inlets for the diversion are located above the Upper Levels Highway. The flow diverted at these locations is not always sufficient to solve all capacity issues downstream of the inlets. The inadequate channel sections are discussed in Section 5.5.1.8. Figures 5-3A and 5-3B show the inadequate channel sections under diversion Scenario 2.

In order to evaluate potential erosion problems, the Frequent Event was modelled to approximate channel velocities under the following scenarios:

1) Existing conditions with no diversion
2) Post-development conditions with diversion
3) Post-development conditions with diversion and an increase in impervious area by 25% in the developed area below Highway One.
4) Post-development conditions with diversion and a decrease in impervious area by 10% and increase in pervious area depression storage by 10% in the developed area below Highway One.

The first two scenarios are included to assess the effect the proposed development above Highway One has on channel velocities and the resulting potential for erosion. The third scenario is included to assess the effect future densification of the developed area below

Highway One has on the channel velocities and erosion potential. It is possible that densities may increase in this area as the housing stock is replaced over time and the resulting effects on the creek channels needs to be considered as part of the ongoing management of the watershed. The fourth scenario considers future implementation of LID's in the developed area below Highway One. Decreasing the impervious area and increasing the depression storage simulates the increased infiltration and hold-up in these areas which would be created by implementation of LID's. This scenario is included so that the District can predict the potential benefits of LID implementation.

The results of the Frequent Event Analysis are included in Appendix J, Table J-3. The frequent event velocities are shown for all four of the scenarios. Maximum Permissible Velocities for the channel reaches are shown for comparison. The MPV's were developed based on the criteria outlined in Section 4.1.2.

The results of the frequent event analysis indicate that there is minimal variance in the modelled velocities between the four scenarios. The increase in impervious area from existing to post-development conditions is small in relation to the overall catchment area. The resulting increase in run-off is not enough to affect the modelled velocities in Scenarios 1 & 2. The modelled velocities in Scenarios 3 & 4 are also not affected by the variance in the model inputs. Because a much larger portion of the overall catchment area lies above the Highway, changes to the impervious area or depression storage to the area below the Highway are not significant enough to affect the modelled velocities.

Based on the modelled velocities for the four scenarios, it appears the model may not be sensitive enough to detect minor changes in channel velocities resulting from minor adjustments to the catchments impervious area or depression storage. As noted in Section 3.4, the drainage areas contain undulating topography and variably sized surficial barriers and depressions where water can be rerouted or retained. These and other heterogeneous soil/slope conditions throughout the watershed are not easily simulated in generic modelling tools, which assume relatively homogeneous conditions throughout defined areas. These limitations make it difficult to accurately simulate velocities in constantly varying creek channel sections with a hydraulic model which assumes constant channel sections and roughness averaged out for the entire length of a given channel reach. A more detailed study of the creek channel reaches may be required to determine the creeks' sensitivity to changes in land use such as densification and LID implementation.

More reliable evidence of channel erosion than the modelled velocities is actual observed erosion problems in the channel reaches. Table J-3 in Appendix J includes the observed conditions from both the Golder and Aqua-Tex reports. The Aqua-Tex report only covers the creek Sections in the upper lands above Highway One. The observed conditions are shown for each channel reach and compared with the FEV's and MPV's.

Based on the modelled FEV's, MPV's and observed creek conditions, the channel reaches were assigned erosion monitoring priorities. This analysis provides a ranking system that can be used to prioritize the monitoring of the creeks for potential erosion problems and highlights the most at-risk sections of the creek which should be considered for rehabilitation. The priorities assigned are as follows:

1) Golder observed erosion problems and Aqua-Tex 'Non-functional' or 'Functional at Risk'
2) FEV > MPV and Golder observed erosion problems
3) Golder observed erosion problems
4) Aqua-Tex 'Non-Functional'
5) Aqua-Tex 'Functional at Risk'
6) FEV > MPV, no observed erosion problems

Table 5-17 shows the number of channel reaches for each erosion monitoring priority.

TABLE 5-17
EROSION MONITORING PRIORITIES

Erosion Monitoring Priority	No. of Channel Reaches
1	5
2	2
3	20
4	14
5	2
6	15

5.5.4 Capital Cost Estimates – Stormwater Diversion Options

Costs for the proposed works identified in both Option A and Option B were developed. A preliminary alignment for these improvements is referenced on Figures 5-3A and 5-3B in Section 5.5.1.2.

Cost estimates (2010 dollars) used for the purpose of this report were based on previous and recent experience with similar projects. Actual (tendered) costs in the future may vary. Langley Concrete Group and Woseley Inc. Canada were contacted to provide supply costs for concrete and HDPE pipe respectively. Supply costs for pipe materials were then doubled to develop initial unit costs for construction. While High Density Polyethylene (HDPE) was considered, the unit costs were four times the cost of concrete which made this option cost prohibitive without sufficient offsetting performance benefits. HDPE was therefore not recommended for primary construction though it may be applicable in select situations.

5.5.4.1 <u>Capital Costs - Option A</u>

The capital costs estimated for Option A is provided in Table 5-18. Costs are provided for both the installation of the linear pipe as well as the diversion structures identified for each intake. Estimates for future operational costs are also presented as 1% of the initial construction cost.

TABLE 5-18
STORMWATER DIVERSION – OPTION A

ID	Description	Diameter (W x H)	Length	Unit Cost ($/m)	Major Cost (Const.)	Annual O&M (1% of Capital)
D-100	Diversion Pipe	1800 mm	500 m	$1,838	$919,000	$9,190
D-200	Diversion Pipe	1350 mm	650 m	$1,100	$715,000	$7,150
D-300	Diversion Pipe	1500 mm	75 m	$1,354	$101,550	$1,016
D-400	Diversion Pipe	1050 mm	150 m	$694	$104,100	$1,041
D-500	Diversion Pipe	1200 mm	100 m	$852	$85,200	$852
D-600	Diversion Pipe	1350 mm	100 m	$1,100	$110,000	$1,100
D-700	Diversion Pipe	1350 mm	375 m	$1,354	$507,750	$5,078
D-800	Diversion Pipe	1800 mm	450 m	$1,838	$827,100	$8,271
D-900	Diversion Pipe	1800 mm X 1800 mm	1350 m	$3,230	$4,360,500	$43,605
D-10	Diversion Structure - Godman	1800 mm			$200,000	$2,000
D-20	Diversion Structure - Turner	1350 mm			$150,000	$1,500
D-30	Diversion Structure - Cave	1500 mm			$150,000	$1,500
D-80	Diversion Structure - Westmount	1800 mm			$200,000	$2,000
D-70	Diversion Structure - Pipe West	1500 mm			$150,000	$1,500
D-60	Diversion Structure - Pipe Middle	1350 mm			$150,000	$1,500
D-50	Diversion Structure - Pipe East 1	1200 mm			$150,000	$1,500
D-40	Diversion Structure - Pipe East 2	1050 mm			$150,000	$1,500
TOTAL					**$9,030,200**	**$90,302**

5.5.4.2 <u>Capital Costs - Option B</u>

The capital costs estimated for Option B are provided in Table 5-19. Costs are provided for both the installation of the linear pipe as well as the diversion structures identified for each intake. Estimates for future operational costs are also presented as 1% of the initial construction cost.

TABLE 5-19
STORMWATER DIVERSION – OPTION B

ID	Description	Diameter (W x H)	Length	Unit Cost ($/m)	Major Cost (Const.)	Annual O&M (1% of Capital)
D-100	Diversion Pipe	1350 mm	500 m	$1,100	$550,000	$5,500
D-200	Diversion Pipe	1050 mm	650 m	$694	$451,100	$4,511
D-300	Diversion Pipe	1200 mm	75 m	$852	$63,900	$639
D-400	Diversion Pipe	750 mm	150 m	$350	$52,500	$525
D-500	Diversion Pipe	900 mm	100 m	$474	$47,400	$474
D-600	Diversion Pipe	900 mm	100 m	$474	$47,400	$474
D-700	Diversion Pipe	1050 mm	375 m	$694	$260,250	$2,603
D-800	Diversion Pipe	1500 mm	450 m	$1,354	$609,300	$6,093
D-900	Diversion Pipe	1800 mm X 1500 mm	1350 m	$3,060	$4,131,000	$41,310
D-10	Diversion Structure - Godman	1350 mm			$150,000	$1,500
D-20	Diversion Structure - Turner	1050 mm			$150,000	$1,500
D-30	Diversion Structure - Cave	1200 mm			$150,000	$1,500
D-80	Diversion Structure - Westmount	1500 mm			$150,000	$1,500
D-70	Diversion Structure - Pipe West	1050 mm			$150,000	$1,500
D-60	Diversion Structure - Pipe Middle	900 mm			$150,000	$1,500
D-50	Diversion Structure - Pipe East 1	900 mm			$150,000	$1,500
D-40	Diversion Structure - Pipe East 2	750 mm			$150,000	$1,500
TOTAL					**$7,412,850**	**$74,129**

5.5.5 Capital Cost Estimates – Minor Drainage Works

Cost estimates (2010 dollars) used for the purpose of this report were based on previous and recent experience with similar projects. Actual (tendered) costs in the future may vary. Costs for the culvert improvements assume upsizing by one pipe size. Actual size requirements should be confirmed during pre-design of each culvert upgrade.

A summary of the costs estimated for the channel sections and culvert upgrades identified under Option A is provided in Table 5-20.

TABLE 5-20
MINOR DRAINAGE WORKS – OPTION A

ID	Type	Location	Existing Dia.	Required Dia.	Length (m)	Unit Cost ($/m)	Minor Cost (Const.)	Annual O&M (1% of Capital)
T-3200	Culvert	Below Diversion	600	750	24	350.00	$8,400	$84
T-2900	Culvert	Below Diversion	900	1050	22	694.00	$15,268	$153
T-2300	Culvert	Below Diversion	700	900	26	474.00	$12,324	$123
W-3000	Culvert	Below Diversion	750	900	24	474.00	$11,376	$114
G-2800	Channel	Below Diversion	-	-	142	1,000.00	$142,000	$1,420
G-1800	Channel	Below Diversion	-	-	77	1,000.00	$77,000	$770
G-1700	Channel	Below Diversion	-	-	34	1,000.00	$34,000	$340
T-2400	Channel	Below Diversion	-	-	74	1,000.00	$74,000	$740
T-2200	Channel	Below Diversion	-	-	48	1,000.00	$48,000	$480
C-3200	Channel	Below Diversion	-	-	57	1,000.00	$57,000	$570
TOTAL							**$479,368**	**$4,794**

As shown in Table 5-9 of Section 5.5.1.8, the number of downstream works required under Option B increases by a factor of 3.6 when compared tothe number of locations identified under Option A. Individual cost estimates were not calculated for each of the Option B locations. If the cost of the upgrades is assumed to also increase by a factor of 3.6 when compared to those of Option A, this additional work would be approximately $1,742,983 ($1,725,725 capital and $17,258 O&M)

5.5.6 Capital Cost Estimates – Summary

Table 5-21 summarizes the estimated costs associated with the Stormwater Diversions and downstream Minor Drainage works for Options A and B. The total initial cost savings on the diversion system suggested by choosing Option B is approximately $1.6 million. This represents a potential cost savings of 18%. However, the savings calculated are for the diversion pipe and inlet structures only and does not include the additional improvements required to the downstream culverts and channel section. As illustrated in Table 5-21, the cost savings associated with the reduced pipe size for Option B is approximately offset by the additional costs for the downstream works under Option B. These values do not include O&M costs.

TABLE 5-21
SUMMARY OF TOTAL COSTS FOR DIVERSION SYSTEM OPTIONS

Description	Diversion	Minor Drainage	Total
Option A	$9,030,200	$479,368	$9,509,568
Option B	$7,412,850	$1,725725	$9,138,575
Difference	$1,617,350	-$1,246,357	$370,993

5.6 Operation and Maintenance

Operation and maintenance for the diversion and its inlets should include spring and fall inspections and removal of debris at the diversion pipe inlets, as well as the outfalls at the seawall, and bi-weekly inspections of the diversion inlets during the rainy season from October to May.

6.0 STAKEHOLDER CONSULTATION

Consultation with stakeholders to date is summarized below:

1) July 17, 2008 – ISMP Initialization Meeting: See meeting minutes attached in Appendix S, Items 3.1 to 3.6 address public consultation and stakeholders group.

2) August 5, 2008 – Letter distributed to residents living adjacent to the creeks. See Appendix S for letter notifying residents of field work taking place along the creek channels as part of the information gathering for the ISMP.

3) December 3, 2008 – Progress Meeting and Presentation of ISMP Criteria. See Appendix S for the meeting minutes and presentation slides from Opus DaytonKnight, SLR Consulting and Golder Associates.

4) June 3, 2009 – Presentation to Stakeholders:

    - Location - Sentinel High School Auditorium, West Vancouver.
    - Time - 4:00pm to 6:00pm.
    - Presenters - British Pacific Properties Ltd., Opus DaytonKnight, SLR Consultants and Golder Associates. See Appendix S for presentation slides.

- Invitees – West Van Streamkeepers, North Shore Coho Society, DFO, MOE, District Staff and General Public. See Appendix S for advertisement posted as a quarter page add in the North Shore News on Friday May 29th and Sunday May 31st.

- Attendee sign-up sheet included in Appendix S.

5) September 30, 2013 – Stakeholders Consultation Meeting:

- Location – West Vancouver Community Centre, Cedar Room, 3rd floor

- Time 1:30 pm to 4:00 pm

- Presenters - Opus DaytonKnight Ltd. See Appendix S for summary notes of meeting and presentation slides.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

1. The five watersheds in the Pipe/Westmount/Cave/Turner/Godman Creeks comprise roughly 616 ha.

2. The District of West Vancouver annual precipitation averages 221 cm of which the largest rainfall occurs in November. The recorded average total precipitation for the month of November is 35.2 cm. The maximum November day recorded is 10.4 cm, over 24 hours.

3. Climate change is predicted to increase high and low extremes in rainfall, temperature, and wind.

4. The study area is characterized by a thin layer of dense, relatively low permeability sediments overlying dense till and/or granitic bedrock topography. The steep slopes also contribute to very little water being retained in the drainage area.

5. The drainage area contains mainly single family residential housing below the Upper Levels Highway. Most of the new residential development that will occur in the study area is between the Upper Levels Highway and the 365 m GSC elevation.

6. Rainfall events are complex in nature and vary both spatially and temporally across the study area. During the flow monitoring period of the ISMP (March 2008 – April 2010), the rainfall event most suitable for calibration is January 15, 2009.

7. The conclusions of the SLR Ecological Overview Report (SLR, 2009) are summarized as follows:

   a) The minimum width of riparian areas in the five creeks ranges from 10 to 17 meters. The RFI values of the five creeks above Highway 1 range from 71% to 92% compared to an RFI of 0% for the portions of the creeks below Highway1.
   b) In situ water quality measurements, taken at two locations along each of the five creeks, generally show results typical of fast-flowing mountain streams of BC coastal areas.
   c) The Benthic Invertebrate sampling conducted on a 52 meter reach of Godman Creek below Highway 1 resulted in an 'Acceptable Site Assessment Rating' for the Module 4 Stream Invertebrate Survey and a B-IBI rating of 38 – Good Stream Condition.
   d) The development area consists mostly of second growth trees, a result of re-growth after clear-cutting in the early 20th century. Within the study area, there are no known rare element occurrences of vascular plants or ecological communities and sensitive ecosystems are mainly limited to riparian areas, wetlands and rock outcrops.
   e) There are five listed bird species and five listed mammal species that have the potential for occurrence within the study area but whose presence has not been confirmed. Two listed species of frog, the coastal tailed frog and the red-legged frog have been confirmed present. The area provides only low-quality habitat for listed dragonfly and butterfly species.

8. Environmental and public protection requires flood protection up to the 200-year storm event. Base flows enable the creeks to be scoured on a regular basis to prevent build-up of materials. Water quality and habitat protection necessitate the

development of Best Management Practices including Low Impact Development guidelines.

9. The drainage issues in the study area are summarized as follows:

   a) The existing storm drainage system is under capacity for 200-year storm runoff event.
   b) As referenced in the Golder and Aqua-Tex reports, there are existing erosion concerns due to a lack of management of peak flows and water quality that comes from poor collection of drainage (on roofs, overland flow and in drainage conduits).

10. The stormwater model constructed in PCSWMM was calibrated to the January 15, 2010 storm event and verified to historical unit runoff rates. Calibration was performed using streamflow data collected at three creeks. Data was calibrated such that the peak flow, time to peak flow, and total flow volume of the response hydrographs were accurately matched.

11. The selected drainage solution provides the following:

    a) Installation of diversion to collect and drain excess flows from above the Upper Levels Highway during high flow events.
    b) Preservation of baseflow in creeks to maintain necessary flows to clean creeks.
    c) Provision of Low Impact Development guidelines to attenuate peak flows and promote improved water quality.

12. The scenario results suggest the increase in flow after development (Scenario 2) is not sufficient to warrant an increase in the size of the proposed diversion pipe.

13. Based on the results of Scenario 4 which allows for a higher flow to remain in the creeks, the required diversion pipe would be reduced by 1-2 standard pipe sizes.

14. The number of downstream culvert and channel deficiencies recommended under Scenario 4 is approximately triple the number required under Scenario 2.

15. The modeled velocities for the four Frequent Event scenarios did not indicate significant differences. Given the challenges associated with the range of conditions on each of the creeks, actual observed erosion problems in the channel reaches is recommended as a more reliable source of information to support mitigation works to address channel erosion.

16. Routine maintenance of the drainage system is vital to proper operation and to ensure sufficient capacity in times of severe rainfall. Scheduled cleaning and maintenance is essential.

17. Two management options were developed to address concerns related to life and property safety. The two options included:

    - Option A - Construct the diversion pipe as defined in Scenario 2. This diversion pipe would be sized for maximum risk aversion and would minimize the number of downstream works required.
    - Option B - Construct the diversion pipe as defined in Scenario 4. This diversion pipe would be smaller than in Option A and hence less expensive to build initially. However, it would result in additional downstream works as well as the need to accept a higher risk of damages to private and public property.

18. A summary of the cost estimates for Options A and B are presented in Table 7-1 below.

TABLE 7-1
SUMMARY FOR DIVERSION OPTIONS

Description	Major Cost	Minor Cost	O&M	Total
Option A	$9,030,200.00	$479,368.00	$95,096.00	$9,604,664.00
Option B	$7,412,850.00	$1,725,725.00	$91,386.00	$9,229,961.00

7.2 Recommendations

1. Confirm priorities and financing of the Integrated Stormwater Management Plan; add allowances for financing and administration costs as appropriate.

2. Continue to review future rainfall data for significant storm events that could be used for additional calibration of the hydraulic model. Consider installation of a new rain gauge between Pipe and Cave Creek. An additional gauge in this vicinity can be used to improve the calibration of the eastern catchments.

3. Initiate further flow monitoring of the creeks in order to capture events suitable for additional calibration of the hydraulic model.

4. Conduct future watershed health assessments to compare with the base measurements from SLR's Ecological Overview Report and assess future development effects on watershed health and the effectiveness of LID practices.

5. Monitor the highlighted erosion problems in the channel reaches. Prioritize upgrades with the Erosion Monitoring Priorities provided in the ISMP. Areas of observed erosion problems are of the highest concern.

6. Coordinate road improvements and other utility work with the storm utility improvements.

7. Refine the concept diversion solutions as development plans are finalized. Undertake PCSWMM model refinements to confirm staging of work.

8. Pre-design reports detailing geologic, hydrologic, ecological and civil requirements for each drainage improvement should be undertaken prior to detailed design and costing of any of the Integrated Stormwater Management Plan structural improvements.

9. Diversion Option A is recommended for implementation as it provides a higher level of protection to downstream life and property while not resulting in a significant increase to overall costs.

10. Prioritize the upgrades as outlined in Section 8.0 Implementation Strategy.

8.0 IMPLEMENTATION STRATEGY

The Implementation Strategy prioritizes the proposed works outlined in the mitigation options. This section includes a prioritized list of the recommended improvements and associated cost estimates. Diversion Option A is the ISMP's recommended mitigation option as it provides the highest level of protection to downstream life and property. The Implementation Strategy follows Diversion Option A (Post-Development Conditions).

8.1 Implementation Strategy - Priority 1

The proposed development above the highway is proceeding gradually from east to west across the study area. Priority 1 of the Implementation Strategy is to construct the diversion system which will capture flows from the Pipe, Westmount and Cave Creek catchment areas. Ideally, the Priority 1 proposed upgrades should be constructed prior to new development in these catchments areas. However, as these upgrades will require time and resources to implement, interim measures to address the impact of development will be permitted subject to the approval of the Director of Engineering and Transportation.

The main branch of the diversion pipe, from Burrard Inlet up to the Westmount interchange at Highway One, should be constructed first, followed by the branches to the Pipe, Westmount and Cave creek inlet structures. Construction of the diversion inlet structures should follow an east to west order. Figures 5-3A and 5-3B show the proposed diversion layout. Table 8-1 provides a sequenced list of the proposed upgrades and associated cost estimates for Priority 1 of the Implementation Strategy.

TABLE 8-1
IMPLEMENTATION STRATEGY- PRIORITY 1

ID	Description	Diameter (W x H)	Length	Unit Cost ($/m)	Major Cost (Const.)	Annual O&M (1% of Capital)
D-900	Diversion Pipe	1800 mm X 1800 mm	1350 m	$3,230	$4,360,500	$43,605
D-800	Diversion Pipe	1800 mm	450 m	$1,838	$827,100	$8,271
D-700	Diversion Pipe	1350 mm	375 m	$1,354	$507,750	$5,078
D-600	Diversion Pipe	1350 mm	100 m	$1,100	$110,000	$1,100
D-500	Diversion Pipe	1200 mm	100 m	$852	$85,200	$852
D-400	Diversion Pipe	1050 mm	150 m	$694	$104,100	$1,041
D-40	Diversion Structure - Pipe East 2	1050 mm			$150,000	$1,500
D-50	Diversion Structure - Pipe East 1	1200 mm			$150,000	$1,500
D-60	Diversion Structure - Pipe Middle	1350 mm			$150,000	$1,500
D-70	Diversion Structure - Pipe West	1500 mm			$150,000	$1,500
D-80	Diversion Structure - Westmount	1800 mm			$200,000	$2,000
D-300	Diversion Pipe	1500 mm	75 m	$1,354	$101,550	$1,016
D-30	Diversion Structure - Cave	1500 mm			$150,000	$1,500

The total costs for Priority 1 of the Implementation Strategy are as follows:

Total Construction Costs	$7,046,200.00
Total O&M Costs	$70,462.00
Total Cost	**$7,116,662.00**

8.2 Implementation Strategy – Priority 2

Priority 2 of the Implementation Strategy is to construct the diversion system which will capture flows from the Turner and Godman Creek catchment areas. The Priority 2 proposed upgrades should be constructed prior to new development in these catchment areas.

The diversion pipe branch to Turner Creek and inlet structure should be constructed first, followed by the pipe branch and inlet structure for Godman Creek. Table 8-2 provides a sequenced list of the proposed upgrades and associated cost estimates for Priority 2 of the Implementation Strategy.

TABLE 8-2
IMPLEMENTATION STRATEGY – PRIORITY 2

ID	Description	Diameter (W x H)	Length	Unit Cost ($/m)	Major Cost (Const.)	Annual O&M (1% of Capital)
D-100	Diversion Pipe	1800 mm	500 m	$1,838	$919,000	$9,190
D-200	Diversion Pipe	1350 mm	650 m	$1,100	$715,000	$7,150
D-10	Diversion Structure - Godman	1800 mm			$200,000	$2,000
D-20	Diversion Structure - Turner	1350 mm			$150,000	$1,500

The total costs for Priority 2 of the Implementation Strategy are as follows:

Total Construction Costs	$1,984,000.00
Total O&M Costs	$19,840.00
Total Cost	**$2,003,840.00**

8.3 Implementation Strategy – Priority 3

Priority 3 of the Implementation Strategy includes the additional improvements to the downstream channels and culverts that are required even after diversion is in place for the Pipe, Cave and Westmount creeks. These upgrades should be constructed following the construction of the upgrades outlined in Priority 1.

The upgrades should progress from east to west across the study area as the diversion intake structures are built. The locations of the creek channel and culvert upgrades are shown in Figures 5-3A and 5-3B. Table 8-3 provides a sequenced list of the proposed upgrades and associated cost estimates for Priority 3 of the Implementation Strategy.

TABLE 8-3
IMPLEMENTATION STRATEGY – PRIORITY 3

ID	Type	Location	Existing Dia.	Required Dia.	Length (m)	Unit Cost ($/m)	Minor Cost (Const.)	Annual O&M (1% of Capital)
W-3000	Culvert	Below Diversion	750	900	24	$474	$11,376	$114
C-3200	Channel	Below Diversion	-	-	57	$1,000	$57,000	$570

The total costs for Priority 3 of the Implementation Strategy are as follows:

Total Construction Costs	$68,376.00
Total O&M Costs	$684.00
Total Cost	**$69,060.00**

Site specific changes to the prioritization to be considered during detailed design. For example, due to its proximity to the proposed diversion route, it may be more cost effective to upgrade W-3000 during construction of the diversion (Priority 1).

8.4 Implementation Strategy – Priority 4

Priority 4 of the Implementation Strategy includes the additional improvements to the downstream channels and culverts that are required even after diversion is in place for the Godman and Turner creeks. These upgrades should be constructed following the construction of the upgrades outlined in Priority 2.

The upgrades should progress from east to west across the study area as the diversion intake structures are built. The locations of the creek channel and culvert upgrades are shown in Figures 5-3A and 5-3B. Table 8-4 provides a sequenced list of the proposed upgrades and associated cost estimates for Priority 4 of the Implementation Strategy.

TABLE 8-4
IMPLEMENTATION STRATEGY – PRIORITY 4

ID	Type	Location	Existing Dia.	Required Dia.	Length (m)	Unit Cost ($/m)	Minor Cost (Const.)	Annual O&M (1% of Capital)
T-3200	Culvert	Below Diversion	600	750	24	$350	$8,400	$84
T-2900	Culvert	Below Diversion	900	1050	22	$694	$15,268	$153
T-2400	Channel	Below Diversion	-	-	74	$1,000	$74,000	$740
T-2300	Culvert	Below Diversion	700	900	26	$474	$12,324	$123
T-2200	Channel	Below Diversion	-	-	48	$1,000	$48,000	$480
G-2800	Channel	Below Diversion	-	-	142	$1,000	$142,000	$1,420
G-1800	Channel	Below Diversion	-	-	77	$1,000	$77,000	$770
G-1700	Channel	Below Diversion	-	-	34	$1,000	$34,000	$340

The total costs for Priority 4 of the Implementation Strategy are as follows:

Total Construction Costs	$410,992.00
Total O&M Costs	$4,110.00
Total Cost	**$415,102.00**

8.5 Implementation Strategy – Summary

The total costs of the Implementation Strategy are shown in Table 8-5.

TABLE 8-5
IMPLEMENTATION STRATEGY- SUMMARY

Priority	Construction Cost	O&M Cost	Total Cost
1	$7,046,200.00	$70,462.00	$7,116,662.00
2	$1,984,000.00	$19,840.00	$2,003,840.00
3	$68,376.00	$684.00	$69,060.00
4	$410,992.00	$4,110.00	$415,102.00
Total	**$9,509,568.00**	**$95,096.00**	**$9,604,664.00**

Prior to implementation of the prioritized works, a preliminary design process should confirm the general solutions proposed in the ISMP. The preliminary design process should include:

right-of-way assessments; finalize alignments of the diversion pipes; diversion intake design (including bedload transport issues); and detailed staging of the works.

The implementation of the ISMP should also include the environmental protection discussed in Section 5.5.3. This includes the implementation of LID's in new development areas as outlined in Section 5.5.3.3 and the monitoring of erosion problems as prioritized in Section 5.5.3.4. The detailed cost associated with these measures is beyond the scope of the ISMP. Further investigation into the effectiveness of LID implementation and channel erosion mitigation should be considered.

GLOSSARY

Antecedent Moisture

In drainage modeling or hydrologic calculations, moisture from previous rainfalls may be accounted for in runoff calculations. This rainfall is referred to as antecedent moisture.

Apron Swale

An impervious circular collection area forming a depression around a catch basin to encourage flow into the catch basin. Asphalt aprons are common construction.

Avulsion

A rapid abandonment of a river channe. Avulsions occur as a result of channel slopes that are much lower than the slope that the river could travel during its new course

Basin Study

A plan of a watershed, or group of watersheds, that sets out the City's intent for that watershed. It will then be used to guide development or redevelopment in that watershed. Its primary thrust will be preventative, but it will also set out remedial measures that are technically sound, socially desirable and financially viable.

Buffer Strip

Vegetation fringe left intact along a stream, river or lake after logging. Can be deciduous or a mix of deciduous and coniferous species, including a complete assemblage of natural forest. e.g., leave strip, riparian zone.

Catchment - See Drainage Area.

Critical Depth

Unstable, turbulent depth where the flow has a Froude number equal to one, (velocity equal to the square root of the depth times gravitational acceleration product).

For any specific energy, other than minimum, two depths of flow are possible and separate: subcritical (deep) and supercritical (shallow) flow. At minimum specific energy, the two depths coincide and flow is critical at critical depth.

Critical Flow - See Critical Depth

Debris Flow

Failure of predominantly coarse saturated material which deforms continuously as a more or less viscous slurry usually in a pre-existing channel.

Debris Slide (Debris Avalanche)

Failure of predominantly coarse unsaturated material which slips down a hillside in rapidly disintegrating blocks.

Debris Torrent

A debris flow of predominantly coarse materials characterized by a high water content and confined in a steep channel.

Designated Flood

A flood, which may occur in any given year, of such magnitude as to equal a flood having a 200-year recurrence interval, based on a frequency analysis of unregulated historic flood records or by regional analysis where there is inadequate stream flow data available.

Detention Pond (see also Storage)

An open or closed impoundment intended to store and release surface runoff for a period of 24 hours or more to attenuate peak flows.

Design Frequency

The average lapsed time between the occurrence of two events (storms, floods, etc.) equal to or exceeding a specified value (intensity, low, etc.).

Discharge

The rate of flow, or volume of water flowing in a stream; usually expressed as cubic metres per second (formerly cubic feet per second).

Discretize

To break up into separate distinct parts.

Doline

A closed depression, often basin-shaped or roughly conical, funnel-shaped depressions, usually formed in karst land surface of carbonate rock strata.

Downcutting

Lowering of stream bed due to stream erosion.

Drainage Area or Drainage

(1) An area surrounded by a continuous height of land within which all runoff is expected to join into a single flow stream, and which extends to the point of junction of the flow stream with some predefined point of discharge at the lowest height of land of the drainage.

(2) The area served by a drainage system receiving storm and surface water, or by a watercourse.

Drainage Basin

See Drainage Area.

Drainage Density

A parameter measuring the ratio of the watershed channel length to drainage area (km/km^2).

Drainage Zone, Discharge Area

A groundwater source, supply or spring.

Ecosystem

Any complex of living organisms together with all the other biotic and abiotic (non-living) factors which affect them. For example, a forest ecosystem is that part of a forest area which is uniform in climate, parent materials, physiography, vegetation, soils, animals and micro-organisms.

Ephemeral Stream

Refers to flows of water which occur only after precipitation or snowmelt and which do not flow long enough or with sufficient volumes to create well-defined channels.

Epikarstic

Pertaining to upper/outer layer of karsified carbonate rock in this unsaturated zone, immediately below the soil areas.

Epiphreactic

Referring to water movement with some speed in the intermittently or seasonally saturated or flood water zone on top of the phreatic zone or in the zone liable to be temporarily part of the phreatic zone in flood time.

Episodic Erosion

or PERIODIC EROSION is an abrupt channel response to slow cumulative effects of a progressive sedimentologic change when a threshold is exceeded.

Erosion

or PROGRESSIVE EROSION is the slow change in grades and landscape adjustments due to river activity. For this study, it is restricted to changes in riverbank or bankline recession.

Fisheries Stream Classification

Fisheries classify the streams or watercourses in various orders of 1 through 5 or higher which represent as 1, the extremities of a watercourse in a watershed and as 5 or higher, a major receiving stream. Each of these are classified as Perennial, (year round flow, well defined channels), Intermittent, (half year flow only), and Ephemeral, (flow occurs only on snow melt or heavy precipitation, not well defined channels, low order streams 1 or 2).

Fissure

An open crack in rock or soil.

Floodplain

The relatively flat or lowland area adjoining a river, stream, watercourse, ocean, lake, or other body of standing water which has been or may be covered temporarily with floodwater. For administrative purposes, the floodplain may be defined as the area that would be inundated by the 1:100 or 1:200 year storm flows.

Flood Proofing

A combination of structural changes and adjustments to properties subject to flooding primarily for the reduction of flood damages.

Floodway

The strip of land that would be flooded by the 1:100 or 1:200 year storm flow.

Freshet

A sudden rise in the level and streamflow of a stream or river, due to heavy rains or rapid melting of snow and ice.

Gullying

The formation of scars in a landscape by erosion.

HADD

Harmful alternation, disruption, or destruction of stream habitat. If determined but DFO-Habitiat under section 35(2) of the Fisheries Act may trigger an Environmental Review under Canadian Environmental Assessment Act (CEAA)

Hard Point

Durable rock or compact soil not readily eroded by stream action and representing a boundary condition in the valley flood plain or valley flat.

High Density Polyethylene (HDPE)

A polyethylene thermoplastic made of petroleum. HDPE is resistant to many different solvents, and has been applied in a variety of stormwater drainage projects.

Hydrograph

A graph showing the discharge of water with respect to time for a given point on a stream or conduit.

Hydrology

The science of engineering that deals with the aspects of rainfall and the nature of its subsequent collection or discharge.

Hyetograph

A graph showing average rainfall, rainfall intensities or volume over specified areas with respect to time.

Ice Contact Materials

Materials in content with ice during glacial activity.

Impervious

A term applied to a material through which water cannot pass, or through which water passes very slowly.

Imperviousness Ratio

The ratio of impervious surfaces to total surface area within a watershed or drainage area. If rainwater leaders are not connected directly to the storm sewer system, but are discharged onto splash pads or into soak-away pits, the impervious roof area may be neglected in calculating the imperviousness ratio. Similarly, if rainwater is temporarily stored on a flat roof or in underground storage to simulate the pre-development agricultural condition of runoff, such impervious surfaces may also be neglected.

Incision

Downcutting of stream bed, sometimes due to lack of sediments (e.g. Mission Creek incision into older fan deposits due to now lower lake level). In local areas, this is sometimes referred to as gullying during an active incision process.

Infiltration

(1) The entering of water through the interstices or pores of a soil or other porous medium.

(2) The entrance of water from the ground into a sewer or drain through porous walls, breaks, defective joints.

(3) Absorption of water by the soil either as it falls as precipitation, or from a stream flowing over the surface.

Integrated Resource Management

The process of setting goals, objectives, strategies and policies in a cooperative framework among all watershed resources and resource uses.

Intensity

As applied to rainfall, the rate at which precipitation falls in a given period, usually expressed in millimetres per hour or inches per hour.

Intermittent Stream

A stream with a defined channel, but which is dry for periods of the year; usually the late summer and fall period of low precipitation and no snow melt.

Isohyetals (compare also Hyetograph)

The graphed distribution of rainfall intensity for a particular storm duration.

Karst

Terrain with special landforms and drainage characteristics due to greater solubility of certain rocks (notably carbonate rocks such as limestone, dolomite or magnesite) in natural waters. Derived from the geographic name "krs" from part of the karst terrain in Slovenia.

Karstic

Pertaining to karst.

Karstification

A periodic or cyclic process, where phases of active solutional development of karst are followed by infilling of karst conduits and voids, depending on global climate regimes.

Lag Time

The time difference between two occurrences, such as between rainfall and runoff.

Major Drainage System (see also Storm, Major Design)

That storm drainage system which carries the runoff from the major design storm. The major system will function whether or not it has been planned and designed, and whether or not developments are situated wisely with respect to it.

The major system usually includes many features such as streets, gulches, and major drainage channels. Storm sewer systems may reduce the flow in many parts of the major system by storing and transporting water underground.

Mean Annual Rainfall (MAR)

The Mean Annual Rainfall is the 24 hour duration, 2 year return period storm for a particular area. 50% of MAR gives the minimum required baseflow in a stream for wildlife protection.

Meander (bendway)

The loop in a river representing the alignment of the river thalweg.

Minor Drainage System (see also Storm, Minor Design)

That storm drainage system which is frequently used for collecting, transporting, and disposing of snowmelt, miscellaneous minor flows, and storm runoff up to the capacity of the system. The capacity should be equal to the maximum rate of runoff to be expected from the minor design storm which may have a frequency of occurrence of once in 2, 5 or 10 years.

The minor system is sometimes termed the "convenience system", "initial system", or the "storm sewer system".

The minor system may include many features ranging from curbs and gutters to storm sewer pipes and open drainage ways.

Morphology

The study of the physical character of a watershed's initial and present relief, drainage network and valley character, and its components which form the majority of variables of the fluvial system.

Mud Flow (earth flow)

Failure of predominantly fine saturated material or an earth slide characterized by a high water content.

Natural Boundary

The water course limits recognized as the wetted stream flow width occurring at bankfill flow conditions in a modified or unmodified channel or section throughout the full length of the stream.

Neap Tide

A tide of minimum range occurring at the first and third quarter of the moon.

Orogeny

The affects of mountainous terrain on climate.

Overcompetent Streams

Streams capable of transporting and eroding sediments (down grading) (contrast undercompetent streams).

Overcontrol

Use of detention or retention storage to reduce storm flow below some minimum such as pre-development flows.

Overland Flow

The flow of water over the ground surface before it flows to channels, swales and ditches.

Perennial Stream

A stream which has flowing water all year.

Pervious

Applied to a material through which water passes relatively freely.

Pipe (karst geology)

A tubular cavity projecting as much as several meters down from the surface into karst rocks and often filled with earth, sand, gravel, breccia, etc.

Planning

The process of determining of the goals and objectives of an enterprise, and the selection, through a systematic consideration of alternatives, of the policies, programs and procedures for achieving them. An activity devoted to clearly identifying, defining and determining courses of action necessary to achieve predetermined goals and objectives.

Precipitation

Any moisture that falls from the atmosphere, including snow, sleet, rain and hail.

Rainfall Excess

That part of a rain of a given storm which falls at intensities exceeding the infiltration capacity and is thus available for direct runoff.

Rainfall Mass Curve

Plot of accumulated precipitation against time from the beginning of the storm.

Retention Pond

An open or closed pond or empoundment primarily intended to promote exfiltration into the groundwater flow.

Riparian Zone

See Buffer Strip.

Routing, Hydraulic

(1) The derivation of an outflow hydrograph of a channel or stream from known values of upstream inflow.

(2) The process of determining progressively the timing and shape of a flood wave at successive points along a stream or channel.

Runoff

That part of the precipitation which reaches a stream, drain, sewer, etc., directly or indirectly.

Direct Runoff

The total amount of surface runoff and subsurface storm runoff which reaches stream channels.

Indirect Runoff

The total amount of surface and sub-surface storm runoff which reaches stream channels after detention underground or in open bodies of water for a substantial length of time.

Sinkhole

A word of American origin used to describe sites of sinking water in a carbonate rock (karst) area; often formed in doline. Sinkholes also include swallets, and like dolines can be mantled by subsequent glacial drift deposits.

Sinuosity

Curvature of a river defined by the ratio of the thalweg length and the valley length.

Specific Energy

In a channel flow, specific energy is the energy per unit weight of water in any section with respect to the bottom of the channel, and includes both depth of water and velocity components.

Spillway

A waterway in or about a dam or other hydraulic structure, for the escape of excess water.

Spring Tide

A tide of greater-than-average range around the times of new and full moons.

Stochastic Models

Hydrologic drainage models (particular to a study site) developed from statistical combinations of probably relevant parameters.

Storage (With Respect to Runoff Analysis)

Detention Storage

That water that is detained on the surface during a storm and does not become runoff until sometime after the storm has ended.

Depression Storage

That portion of the rainfall that is collected and held in small depressions and does not become part of the general runoff.

Storage (With respect to Runoff Controls)

Upstream Storage

The storage of storm runoff water near the points of rainfall occurrence.

Downstream Storage

The storage of storm runoff water at some distance from the points of rainfall occurrence but before it reaches areas where it may endanger lives or property.

Off-line Storage

The temporary storage of storm runoff water away from the main channel of flow.

On-line Storage

The temporary storage of storm runoff water behind embankments or dams located on the channel.

Storm Drainage System

All facilities used for conducting the stormwater through and from a drainage area to be point of final outlet, consisting of any or all of the following: conduits and appurtenant features, canals, channels, ditches, streams, ravines, gullies, flumes, culverts, streets, and pumping stations.

Storm, Major Design

That storm used for design purposes, the runoff from which is used for sizing the major storm drainage works.

Storm, Minor Design

That storm used for precipitation running off from the surface of a drainage area during and immediately following a period of rain.

Stream

A watercourse which has a flow of water for all or part of the year and has a defined channel showing signs of scouring and washing.

Streambank

The rising ground bordering a stream channel.

Streambed

The bottom of the stream below the usual water surface.

Stream Management Area

Any area related to a natural stream or course of water susceptible of being part of the major flood path, where any modification of natural conditions of flow is restricted to protect the environment and private or public properties. The restrictions applicable are defined in the Water Act, Land Titles Act, City Master Drainage Plan or City By-Laws.

Stream Reach

A section of stream of reasonably uniform gradient, streambed, streambank and flow pattern.

Subcritical Flow

Tranquil laminar flow with a Froude number less then one (velocity less than the square root of the depth times gravitational acceleration product) and water depth greater than critical depth.

Supercritical Flow

Turbulent rapid flow with a Froude number greater than one, (velocity greater than the square root of the depth times gravitational acceleration product) and water depth less than critical depth.

Surcharge

The flow condition occurring in closed conduits when the hydraulic gradeline is above the conduit crown, or the transition from open channel to pressure flow.

Surficial Materials

Naturally occurring unconsolidated materials including soil which cover the earth's surface.

Suspended Load

Material held in suspension by the river flow and transported at a velocity virtually identical to that of the water. Movement is unrelated to other particles in suspension.

Sustainable Development Goal

A balance between environment and natural resource systems on which human life and well-being depend, brought about by economic activity that does not undermine or impair tomorrow's economic prospects or quality of life.

Synthetic Unit Hydrograph

A unit hydrograph developed for an ungauged drainage area, based on known physical characteristics of the basin.

Thalweg

The route of deepest river and main channel flow.

Time of Concentration

The time required for storm runoff to flow from the most remote point of a watershed or drainage area to the outlet or point under consideration. It is not a constant, but varies with depth of flow, grades, length and condition of conduit and/or channel.

Topography

A general term to include characteristics of the ground surface such as plains, hills and mountains, degree of relief, steepness of slopes, and other physiographic features.

Transmissibility

The flow rate of an aquifer (or pumped recharge into an aquifer) defined as the flow divided by the aquifer depth for a unit width of aquifer, (Volume per unit of time per unit of projected area).

Trash Rack

A barrier constructed to catch debris and exclude it from a downstream conduit. An improperly maintained trash rack may render a conduit useless.

Undercompetent Streams

Streams unable to transport sediment inflows (aggrading) (contrast overcompetent streams).

Underfit Stream

A river cut valley presently occupied by a much smaller river or stream.

Ungulate

A hoofed mammal, such as a deer or goat.

Unit Hydrograph

A runoff hydrograph resulting from one inch of excess rainfall applied to a given watershed over some specified time interval; also called unit graph.

Valley Flat

Relatively flat surfaces on the valley floor subject to flooding.

Watercourse

A channel in which a flow of water occurs, either continuously or intermittently, and if the latter, with some degree of regularity. Such flow must be in a definite direction. Watercourses may be either natural or artificial, and the form may occur either on the surface or underground.

Artificial

A surface watercourse constructed by human agencies, usually referred to as channel, canal or ditch.

Natural

A surface watercourse created by natural conditions.

Watercourse Storage

The volume of water stored in a watercourse. Generally considered in the attenuation of the peak of a flood hydrograph moving downstream.

Watershed

See Drainage Area.

REFERENCES

AESL, AXYS,(2008), Rogers and Marr Creeks Integrated Stormwater Management Plan, for District of West Vancouver, Draft February 2008.

Aqua-Tex Scientific Consulting Ltd. (2012), 2011 Proper Functioning Condition (PFC) Assessment for Pipe, Westmount, Cave & Turner Creeks, January 2012.

British Columbia Ministry of Transportation and Highways (1983), Debris Torrent and Flooding Hazards – Highway 99, Howe Sound, Thurber Consultants Ltd., April 1983.

Chow, Ven Te, Ph.D. (1959), Open-Channel Hydraulics, McGraw-Hill Book Company, Toronto, ON, 1959.

Cook D.J., and Dickenson, (1986), "Impact of Urbanization on Hydrological Response of a Small Watershed", Can. Jour. Civ. Eng., 13, (6), Dec., pp 620-630.

District of West Vancouver (1973), District of West Vancouver Drainage Survey, Dayton & Knight, December 1973.

District of West Vancouver (2004), Official Community Plan – Bylaw No. 4360, 2004, June 2004.

Forestry Practice Code (2002), Fish-stream Crossing Guidebook, Ministry of Forests, Ministry of Environment, Ministry of Energy and Mines, Province of British Columbia, March 2002.

Golder Associates Ltd. (2009), Hydro-Geotechnical Stream Assessment, January 2009.

GVRD (1999), Assessment of Current and Future GVS&DD Area Watershed and Catchment Conditions, Greater Vancouver Sewerage & Drainage District, LWMP, Storm Water Management Technical Advisory Task Group, Vancouver, B.C.

GVRD (2002), Integrated Stormwater Management Planning, Terms of Reference Template, Kerr Wood Leidel, Working Draft Report for Greater Vancouver Regional District, May 2002.

GVRD (2005), Template for Integrated Stormwater Management Planning 2005, Terms of Reference Template Draft Report, Kerr Wood Leidel Ltd., Greater Vancouver Sewerage & Drainage District, Vancouver, B.C. December 2005.

GVS&DD (1998), Options for Municipal Stormwater Management Governance, Dayton & Knight Ltd., Economic & Engineering Services, Inc., Environmental Engineering and Science, ECL Envirowest Consultants Limited, April 1998.

National Topographic System (NTS) (1983), *1:50,000 Map 92G/6 North Vancouver*, *Edition 4*, Department of Energy, Mines and Resources, Ottawa, ON.

Province of British Columbia (2002), A Guidebook For British Columbia – Stormwater Planning, Ministry of Water, Land and Air Protection, May 2002.

Province of British Columbia (2004), Flood Hazard Area Land Use Management Guidelines, Ministry of Water, Land and Air Protection, May 2004.

Rantz, S.F. (1971) Suggested Criteria for Hydrologic Design of Storm Drainage Facilities in the San Francisco Bay Region, California, Geological Survey Open File Rep., U.S., November 1971.

Roesner, Larry A., Brian P. Bledsoe, and Robert W. Brashear (2001) "Are Best-Management-Practice Criteria Really Environmentally Friendly", ASCE – Journal of Water Res. Plan and Manage, Vol. 127 (3), May, June 2001.

Scheckenberger, Ronald B., and Raymond T. Guther (1998) "Stormwater Management of Somebody Else's Land" Advances in Modeling the Management of Stormwater Impacts – Vol. 6 Ed William James Pub. CHI, Guelph, Ontario, ISBN: 09697422-8-2, February 1998, 504 pp.

Shumm, S.A. (1984) Incised Channels, Water Resources Publications, Littleton, Colorado, U.S.A.

Shumm, S.A., (1977) The Fluvial System, John Wiley & Sons, New York, 1977.

SLR Consulting (Canada) Ltd. (2009), Ecological Report, March 2009.

Thorne, C.R., Bathhurst, J.C., and Hey, R.D. (1987) editors, Sediment Transport in Gravel-Bed Rivers, John Wiley & Sons, ISBN 0 471 90914 9.