FINAL REPORT

PEAK FLOW-CULVERT DESIGN STUDY: PENTICTON FOREST DISTRICT

Prepared for:

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Project 042-13.00

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SUMMIT
ENVIRONMENTAL CONSULTANTS LTD.

October 18, 2000
Reference: **042-13.00**

Mr. Nick Kleyn, R.P.F.
Weyerhaeuser Company Limited. – B.C. Timberlands
668 St. Anne Road
Armstrong, B.C.
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Dear Mr. Kleyn:

Re: Peak Flow – Culvert Design Study for Penticton Forest District

Summit Environmental Consultants Ltd. is pleased to provide you with two copies of the Final Report on the above-noted study. The report presents our peak flow regionalisation for the Penticton Forest District, and discusses the features, assumptions and limitations of the computer program *CULVERT* (Ver. 6.1) that uses the results of this regionalisation for estimating required culvert sizes.

We have enjoyed working on this project, and would be pleased to provide any additional assistance you may require. Please call me at 545-3672 if you have any questions or wish to discuss any aspect of the report.

Yours truly,

Summit Environmental Consultants Ltd.

Brian T. Guy, Ph.D., P.Geo., P.H.
President
Senior Geoscientist

Enclosures: 2 copies of Final Report

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1.0 INTRODUCTION

The *Forest Practices Code of British Columbia Act* (the Code) and Regulations provide design criteria for bridges and culverts. Culvert and bridge sizing is based on calculating the minimum opening required to pass the 100-year return period[1] peak flow for all stream culverts and permanent and semi-permanent bridges, and the 50-year return period peak flow for temporary bridges and all other culverts.

The Forest Road Engineering Guidebook (MoF/MELP, 1995) provides a simple default method of estimating the diameters of circular corrugated metal pipes (CMPs) needed to safely pass 100-year design flows. However, experience in some parts of British Columbia has suggested that the simple guidebook method tends to overestimate the required pipe sizes. In addition, there is little guidance for estimating 50-year design flows, or for sizing structures other than circular CMPs. The Community Watershed Guidebook (MoF/MELP, 1996) presents a method for estimating 100-year design flows which is based on an earlier hydrologic study of the entire province. However, this method does not reflect local or regional variations in peak flows, nor does it provide any information on the uncertainty of the design flow estimate.

The objectives of the present project are to develop a reliable method for estimating 50-year and 100-year design peak flows within the Penticton Forest District (Figure 1.1), based on local information and accepted techniques, and for sizing the structures necessary to safely pass these design flows.

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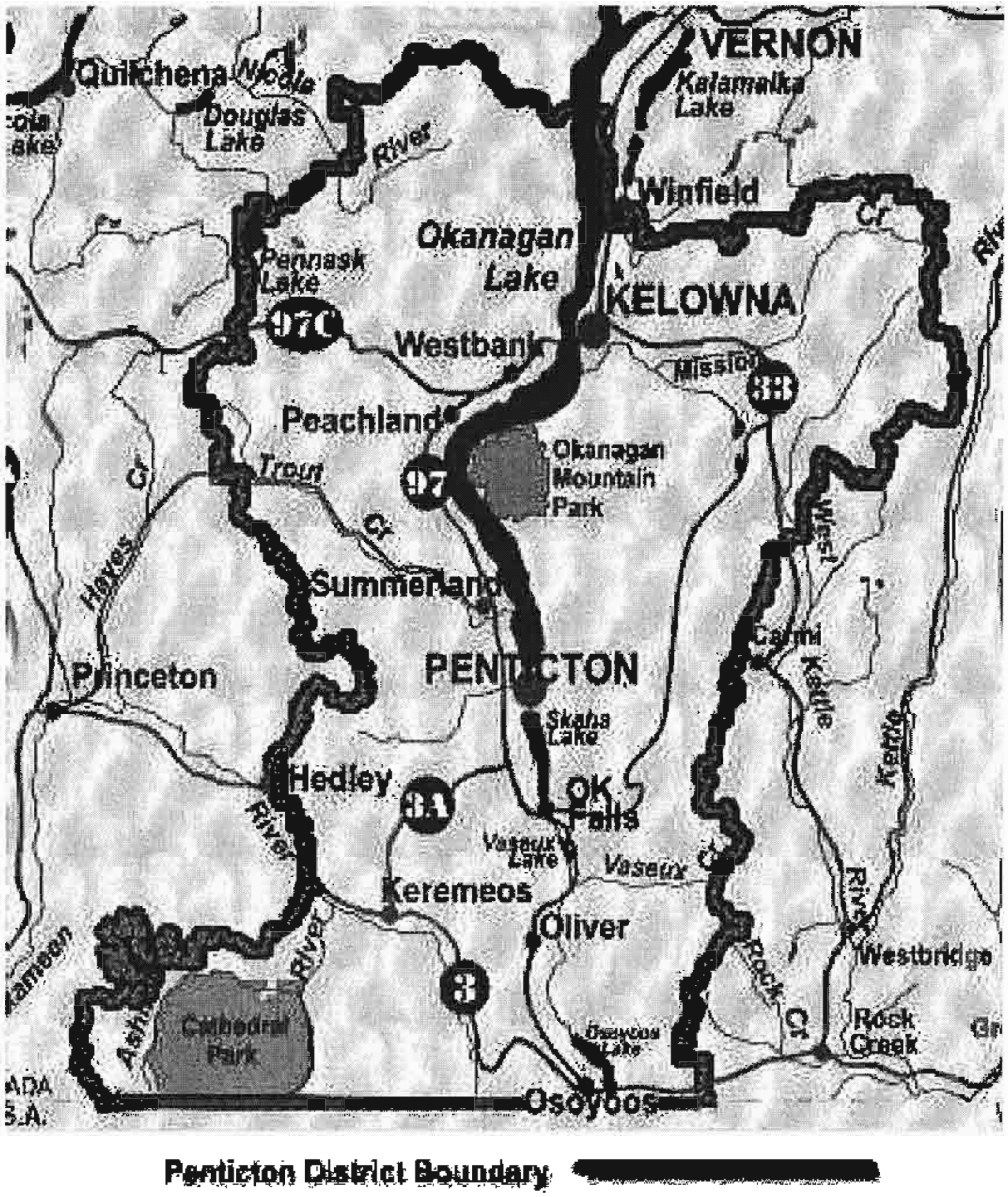
[1] Return period is a way of expressing the exceedance probability. The probability of the 100-year peak flow being equalled or exceeded in any given year is 1/100 or 0.01. In other words, in any year the odds that this flow will be equalled or exceeded are 1 in 100.

Figure 1.1. Penticton Forest District.



The study involved two components:

1. a peak flow hydrology study, and
2. development of Version 6.1 of computer program *CULVERT*, in which the design flow hydrology for the Penticton Forest District (PFD) and an analysis of culvert hydraulics are combined to simplify the process of estimating design peak flows and culvert sizes.

This report provides a summary of the hydrology study, reviews the culvert hydraulic analysis, and introduces the computer program (*CULVERT*, Ver. 6.1). The hydrologic methodology is described in more detail in Appendix A, and a Users Manual for program *CULVERT* (Ver. 6.1) is provided in a separate report (Summit, 2000).

The hydrologic analysis and the *CULVERT* (Ver. 6.1) program reported herein have been developed specifically for use within the boundaries of the Penticton Forest District. ***CULVERT* (Ver. 6.1) is protected by copyright and its use is governed by a licence agreement. The program cannot be used by any party not holding a valid licence agreement, and cannot be used for any purpose other than the estimation of design flows and culvert sizes for locations within the boundaries of the Penticton Forest District.**

2.0 DESIGN PEAK FLOW HYDROLOGY

2.1 BACKGROUND

This section presents a summary of the design peak flow study. A detailed presentation is included in Appendix A. The peak flow hydrology study included two components:

1. review of existing data, information, reports, and analyses relevant to the hydrology of the Penticton Forest District, and
2. development of an appropriate methodology for estimating the required design flows, which comprises both the Index Flood Method and the Rational Formula Method.

2.2 INDEX FLOOD METHOD

The Index Flood method has been used to estimate design discharges for ungauged watersheds larger than 10 km^2 in area within the Penticton Forest District (the study area). This method is consistent with the methodology presented in the Community Watershed Guidebook (MoF/MELP, 1996). All of the streamflow gauging stations[2] in the southern interior of British Columbia (and several in Washington State) that were operational during the past century were examined for relevance to the project and the study area. Streamflow data from 62 of these stations was used for the analysis. These 62 stations were selected because they are not regulated and their watersheds contain no major lakes or wetlands that could significantly moderate the annual flood hydrograph. The locations of the relevant stations within and adjacent to the Penticton Forest District are indicated on Map 1, attached. The data for each station was used to calculate the mean annual peak daily discharge for the period of record. The estimate was then adjusted to a long term mean based on the flow records for several stations with records extending back to near the beginning of the 20th century. The adjusted mean annual peak daily discharges for the 62 stations are the "index floods" for those stations. These values were then transformed to a scale-independent estimate of the discharge per square kilometre[3], and plotted on a 1:250,000 scale base map. Spatial relationships between the streamflow gauging stations were then examined, and four zones homogeneous with respect to peak flows were delineated within the study area (Map 1). The zones are statistically different from each other at the 90% confidence level. The zones are:

1. Zone 1: Southern Okanagan Basin,
2. Zone 2: Northern Okanagan Basin,
3. Zone 3: Eastern Okanagan Highland and Okanagan Range, and

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[2] Maintained by Environment Canada

[3] Peak discharge per unit area (Q_{p2}/A) is known to be a function of drainage area (A). This effect must be removed before basins of different size can be compared. For the purpose of zone delineation, we have done this by calculating and mapping $k = Q_{p2}/A^{0.75}$, where Q_{p2} is the index flood. The exponent 0.75 is approximately correct throughout B.C.

4. Zone 4: Upper Kettle River.

The driest part of the Penticton Forest District is located in the valley bottoms of the Okanagan and Similkameen River valleys and has been designated *Zone 1*. This zone extends from Summerland at the northern end to Osoyoos at the south. The main water bodies in this zone include Okanagan River, Similkameen River and Trout Creek, as well as Skaha Lake, Osoyoos Lake and the southern end of Okanagan Lake. The dominant biogeoclimatic zones include Bunchgrass, Ponderosa Pine and Interior Douglas Fir.

Zone 2 includes the Okanagan valley north of Zone 1, around Kelowna. It also forms a narrow band east and west of Zone 1 along the upper valley sides of the Okanagan and Similkameen River valleys, reflecting the rapid climatic transition from valley bottom to plateau in the southern part of the Penticton Forest District. The larger water bodies in Zone 2 include Trepanier Creek, Powers Creek, Lambly Creek and the lower reaches of Mission Creek. The Interior Douglas Fir biogeoclimatic zone is dominant, though the Ponderosa Pine zone is found along the valley bottom.

Zone 3 is comprised primarily of the plateau east of the Okanagan valley, but also includes the mountainous terrain west of Penticton and Osoyoos. The main water bodies include the headwaters of Ellis Creek, Penticton Creek, Mission Creek, Vaseau Creek and McNulty Creek, as well as the Ashnola River. The dominant biogeoclimatic zones in Zone 3 are Montane Spruce and Engelmann Spruce – Subalpine Fir.

Zone 4 is limited to the eastern-most part of the Penticton Forest District. It includes the relatively wet headwaters of the West Kettle River. The biogeoclimatic zone is typically Interior Cedar Hemlock, with Engelmann Spruce – Subalpine Fir at higher elevations.

Using the daily peak flow data from the gauged streamflow stations, three separate analyses were undertaken for each peak discharge zone.

1. Regression-fit curves relating mean annual peak daily discharge[4] (Q_{p2}) to drainage area (A) were derived.
2. Representative ratios of the 50-year (Q_{p50}) and 100-year (Q_{p100}) return period peak daily discharge to the mean annual peak daily discharge (Q_{p2}) were estimated.
3. A representative estimate of the ratio between peak instantaneous and peak daily discharge was determined.

Mean estimates of the 50-year and 100-year return period design peak flows for a given location are determined using the results of each of the three analyses, as follows:

$$Q_{p50i} = (I/D)(Q_{p50}/Q_{p2})(Q_{p2}) \qquad (1)$$

$$Q_{p100i} = (I/D)(Q_{p100}/Q_{p2})(Q_{p2}) \qquad (2)$$

where:

- Q_{p50i} = 50-year return period annual maximum instantaneous discharge to be estimated [m^3/s],
- Q_{p100i} = 100-year return period annual maximum instantaneous discharge to be estimated [m^3/s],
- I/D = representative ratio of instantaneous to daily peak discharge,
- Q_{p2} = index flood (mean annual maximum daily discharge) based on the peak flow zone and the drainage area upstream of the proposed bridge or culvert [m^3/s],
- Q_{p50}/Q_{p2} = representative ratio of 50-year return period annual maximum daily discharge to the index flood, and
- Q_{p100}/Q_{p2} = representative ratio of 100-year return period annual maximum daily discharge to the index flood.

[4] This is the "index flood".

Values of I/D, Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} for each peak flow zone were determined from a subset of the available data. Uncertainty in the design flow estimate is quantified by computing a one standard error confidence interval about the mean estimate. The standard error is based on the combined errors in the mean annual peak daily discharge, instantaneous to daily peak discharge ratio, and the ratios of the 50-year and 100-year return period to the mean annual peak daily discharge.

2.3 RATIONAL FORMULA METHOD

It is generally accepted that the Index Flood Method (as described in section 2.2) is not reliable for estimating peak flows in watersheds smaller than about 10 km^2 in area. Accordingly, the Rational Formula Method (as presented by Reksten and Davies, unpub.) was applied to the four peak flow zones. Precipitation data from Oliver, Vernon and Revelstoke were used. The results were used to modify the slope of the design curves derived by the Index Flood Method for areas smaller than 10 km^2. Standard error estimates for watersheds less than 10 km^2 in size are assumed to be equivalent to those derived using the Index Flood Method for drainage areas greater than 10 km^2.

2.4 RECOMMENDED DESIGN PEAK FLOW CURVES

Recommended design peak flow curves, developed using the methods described in Sections 2.2 and 2.3, are presented in Figures 2.1 through 2.8. Dashed lines indicate the estimated one standard error (68%) confidence interval about the mean design flow estimate. For new installations, the recommended design curve is the upper dashed line in each figure. Using this curve, there is an 84% chance that the "true" design flow is less than the recommended value. This is considered a reasonable level of design conservatism. The suitability of existing installations can be evaluated by assuming that the "true" design flow falls within the one standard error confidence interval about the mean design curve. On each curve, the discharge passable by single 400 mm and 2000 mm CMPs are indicated, which facilitates identification of required culvert sizes according to watershed area, as outlined in Table 2.1.

Table 2.1. Relationship between design culvert size and watershed area for 400 mm and 2000 mm CMPs.

Zone	Return Period	Watershed area upstream of new[1] culverts of the following diameter:	
		400 mm	2000 mm
1	50	0.152 km^2 (15.2 ha)	72.2 km^2
	100	0.116 km^2 (11.6 ha)	57.1 km^2
2	50	0.087 km^2 (8.7 ha)	42.4 km^2
	100	0.072 km^2 (7.2 ha)	35.6 km^2
3	50	0.040 km^2 (4.0 ha)	19.9 km^2
	100	0.033 km^2 (3.3 ha)	16.9 km^2
4	50	0.028 km^2 (2.8 ha)	12.1 km^2
	100	0.025 km^2 (2.5 ha)	11.0 km^2

1: data based on "recommended" design flow (mean design flow plus one standard error)

2.5 GUIDANCE FOR SELECTION OF PEAK FLOW ZONE

The boundaries of the homogeneous peak flow zones shown on Map 1 have been drawn to represent as closely as possible the hydrologic variations indicated by the data and, where data are sparse, have been inferred based on experience and professional judgement. Despite the fact that the zones are statistically different from each other, hydrologic conditions vary continuously, grading from drier zones to wetter ones along a continuum. Conditions do not change instantaneously along precisely identifiable boundaries, as suggested by a line drawn on a map. This fundamental spatial variability of hydrologic properties must be considered when choosing which zone to use for design discharge estimation, especially at or near zone boundaries.

The key consideration is the location of the drainage basin upstream of the proposed (or existing) installation, not the location of the installation itself. For drainage basins completely within one peak flow zone and well away from a zone boundary, one can be confident that that zone represents the drainage basin, and no other zones need be considered. For basins within one peak flow zone but close to a zone boundary, it is still likely that that zone represents the basin. However, it may also be appropriate to calculate a design discharge based on the adjacent peak flow zone, and to consider the suitability of both design discharge estimates based on knowledge of the site conditions.

Drainage basins that cross zone boundaries must be addressed carefully. Since snowmelt dominates peak flow hydrographs in the Penticton Forest District, a zone covering the higher elevations is more appropriate that a zone covering the lower elevations of a basin. As a general rule basins that have more than about 40% of the area within an upstream zone should be represented by the upstream zone. However, it is appropriate to calculate the design discharge based on both zones and choose the most appropriate one, based on the site conditions. Basins that have less than 40% of their area in the upstream zone will most likely be best represented by the downstream zone, but this should be confirmed based on the site conditions. In some cases, site conditions may not help identify which zone is most representative of a drainage basin close to or crossing a peak flow zone boundary. The recommended course of action in these circumstances is to choose the larger design discharge (i.e. the wetter hydrologic zone). This is a considered to be a conservative approach, and should prevent unintentional under-sizing of an installation.

2.6 Guidance for Model Use in Areas Affected by Hydrologic Storage

Storage in lakes and wetlands tends to attenuate the peak of the flood hydrograph, generating lower but longer floods. Because the data set used to develop the peak flow models presented herein excludes stations significantly influenced by lake and wetland storage, the design flows produced by the model will overestimate the design discharge for installations downstream of significant lake and wetland storage. The magnitude of this effect on Q_{100} was estimated by analysing data for 13 gauged rivers that have lakes affecting one third or more of their watershed but are not regulated by dams. Table 2.2 presents the ratio of the actual 100-year design flow (Q_{actual}) to that predicted by the model ($Q_{predicted}$) for each of the 13 stations.

Table 2.2. Ratio of actual to predicted 100-year design flows for streams affected by natural lakes.

Station Name	Station No.	Ratio of Q_{actual} to $Q_{predicted}$
Murtle River Above Dawson Falls	08LA004	0.86
Celista Creek Near Albas	08LE025	0.76
Scuitto Creek Near Barnhart Vale	08LE036	0.79
Spa Creek Above Cowpersmith Diversion	08LE042	0.77
Clark Creek Near Winfield	08NM146	0.72
Horn Creek Near Olala	08NM147	0.57
Camp Creek Near Thirsk	08NM134	0.52
Wolfe Creek at Outlet of Issitz Lake	08NL041	0.43
Bowron River near Wells	08KD001	0.53
Quesnel River at Likely	08KH001	0.84
Barriere River at the Mouth	08LB020	0.60
Barriere River Below Sprague Creek	08LB069	0.90
Vernon Creek below Arda Dam	08NM175	0.55
Mean		**0.69**
Standard Deviation		**0.15**

The "actual" discharges were estimated from the data, and the "predicted" discharges were produced by CULVERT (Ver. 6.1).

It is suggested that users apply a $Q_{actual}/Q_{predicted}$ ratio of 85% to streams affected by lake storage (equal to the mean value plus one standard deviation), which is considered to be an appropriate level of design conservatism. This suggestion does not apply to crossings downstream of reservoirs.

Figure 2.1. Recommended design curves for the 50-year return period annual maximum instantaneous discharge – Zone 1: Southern Okanagan Basin.

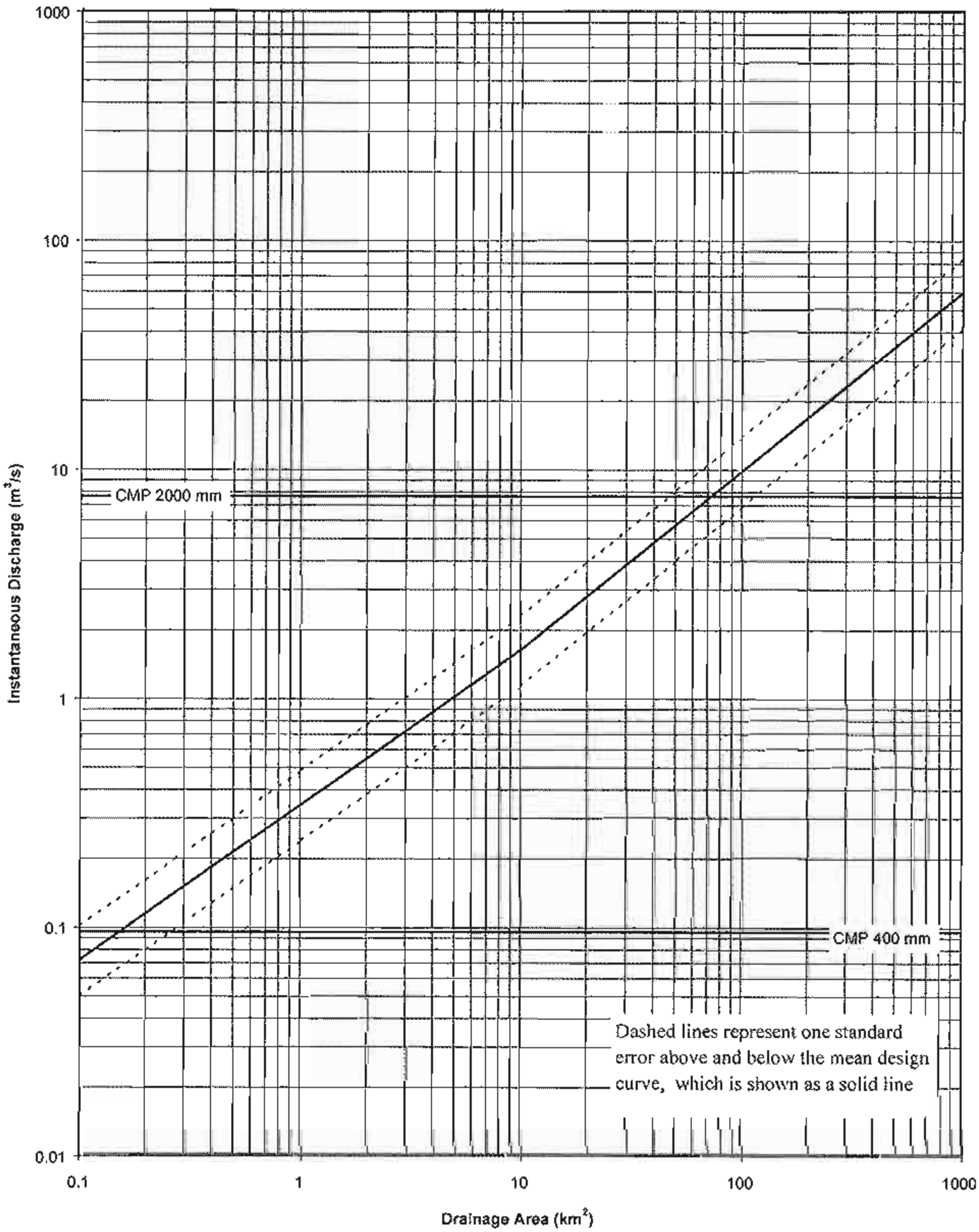


Figure 2.2. Recommended design curves for the 50-year return period annual maximum instantaneous discharge – Zone 2: Northern Okanagan Basin.

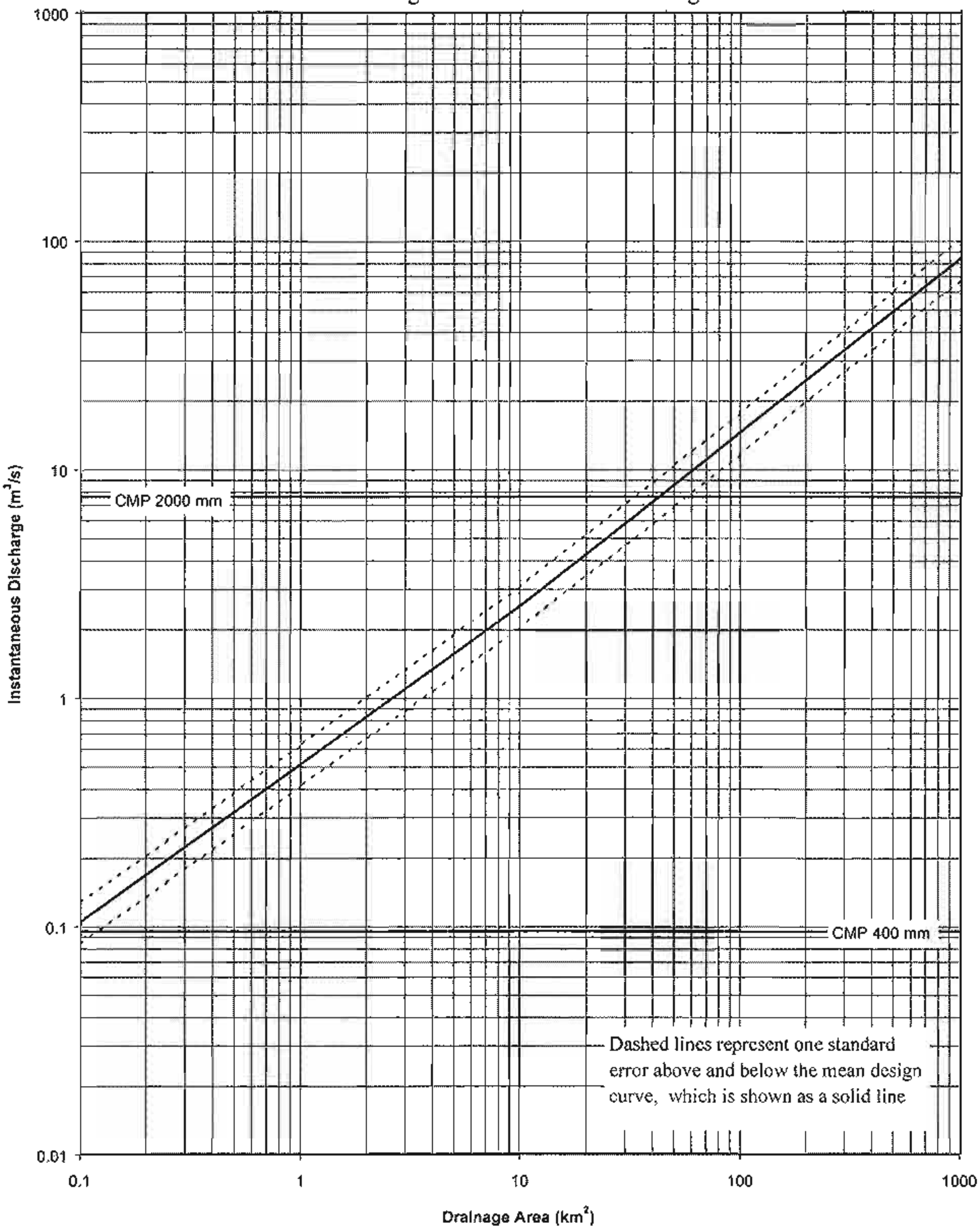


Figure 2.3. Recommended design curves for the 50-year return period annual maximum instantaneous discharge – Zone 3: Eastern Okanagan Highland and Okanagan Range.

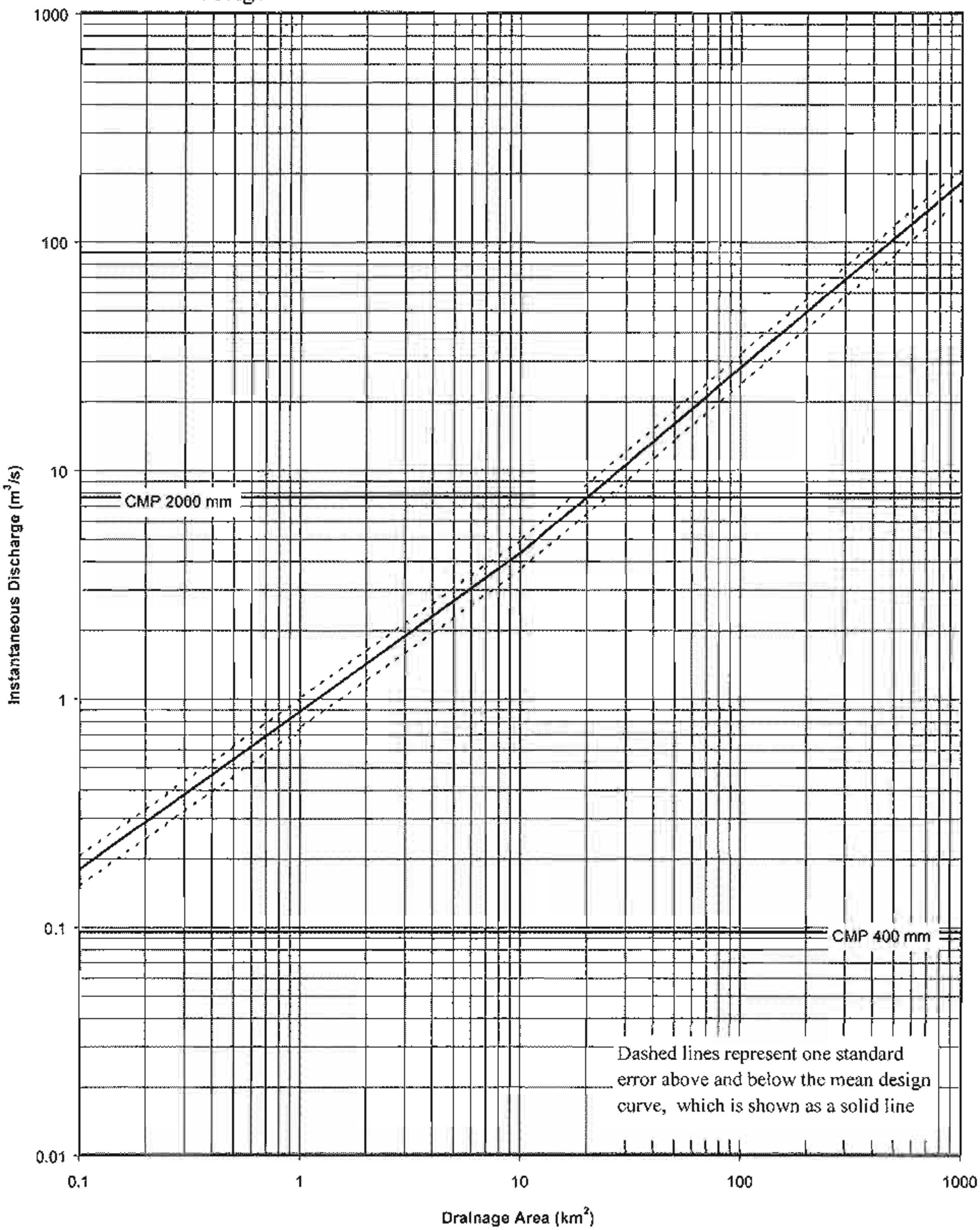


Figure 2.4. Recommended design curves for the 50-year return period annual maximum instantaneous discharge – Zone 4: Upper Kettle River.

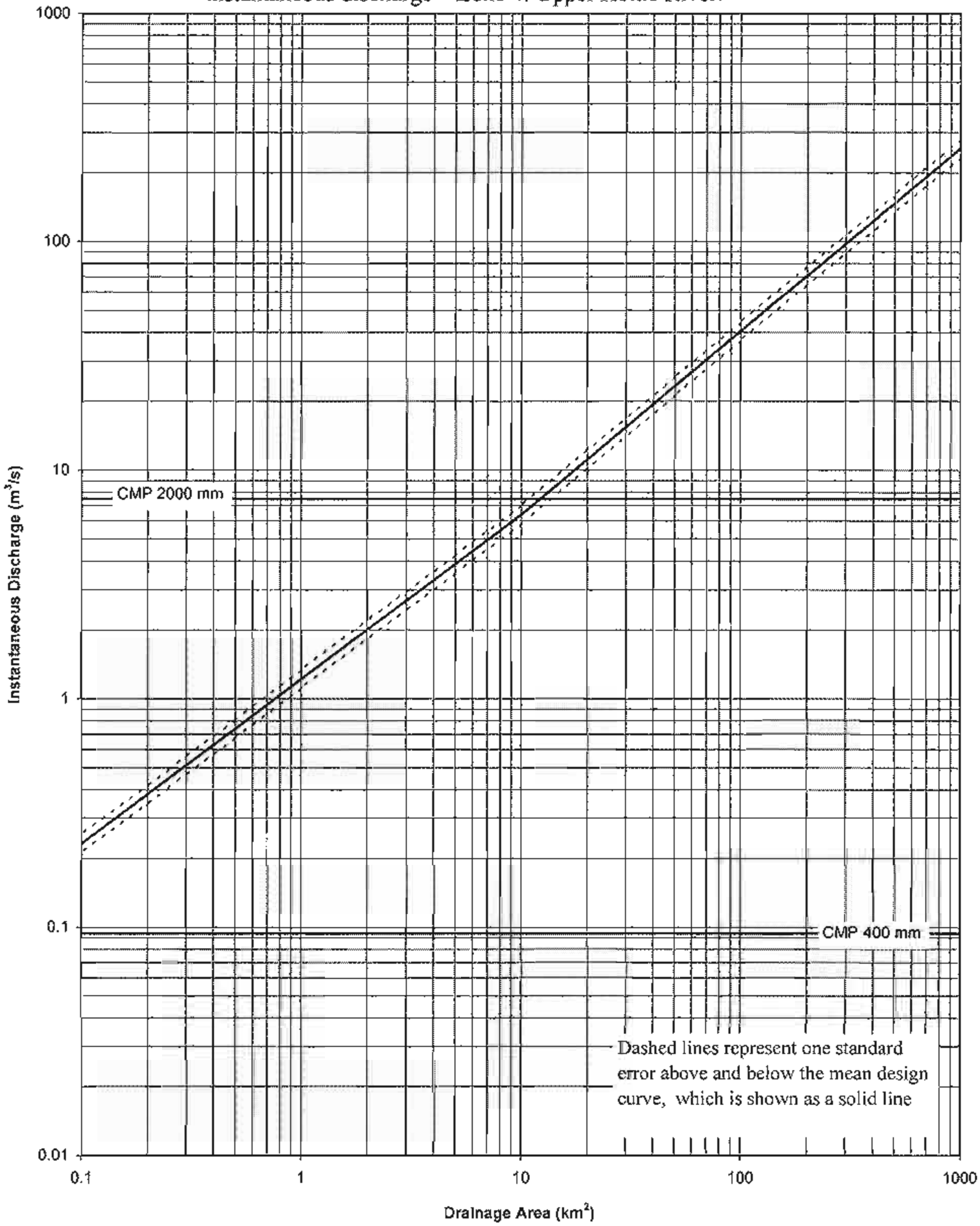


Figure 2.5. Recommended design curves for the 100-year return period annual maximum instantaneous discharge – Zone 1: Southern Okanagan Basin.

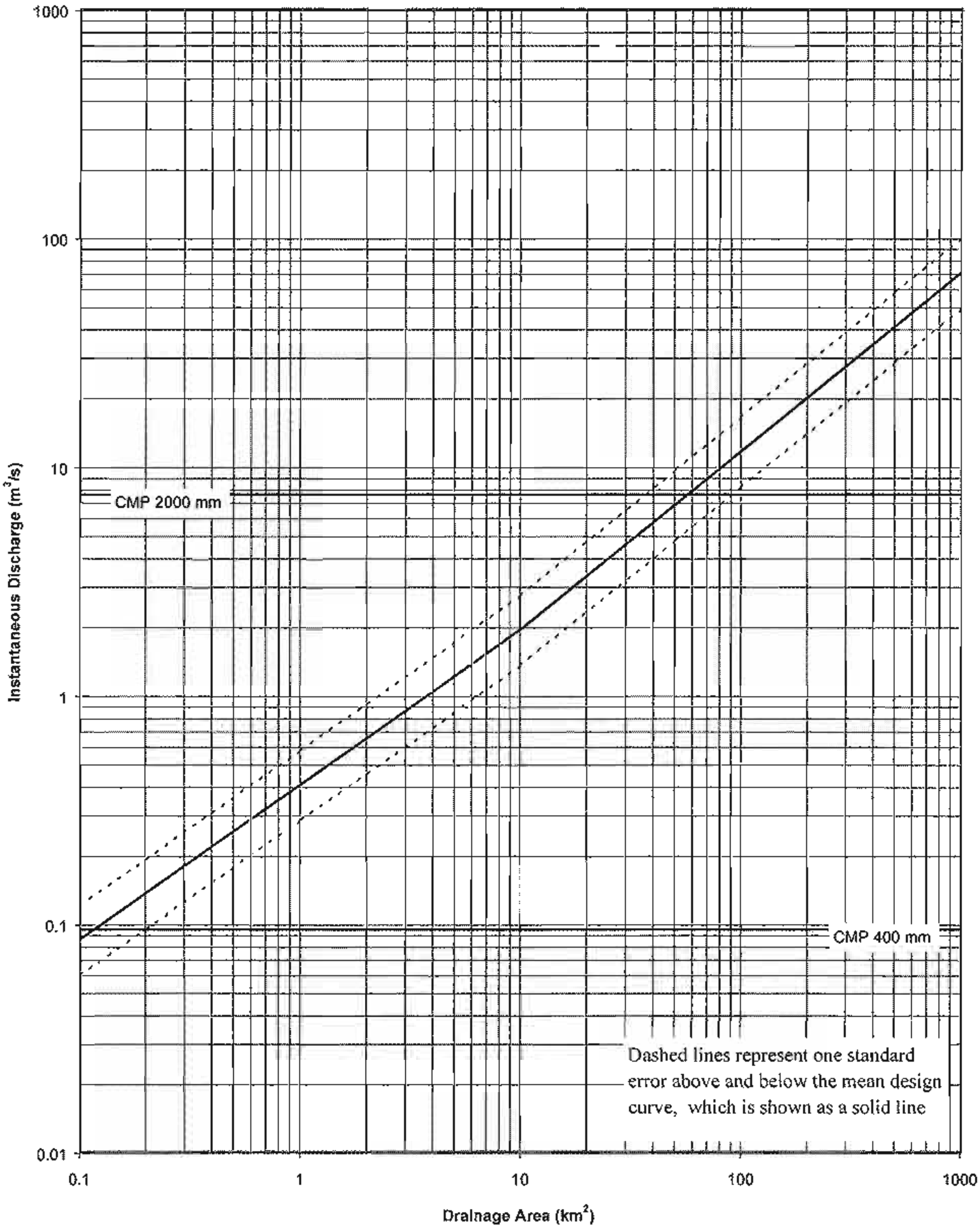


Figure 2.6. Recommended design curves for the 100-year return period annual maximum instantaneous discharge – Zone 2: Northern Okanagan Basin.

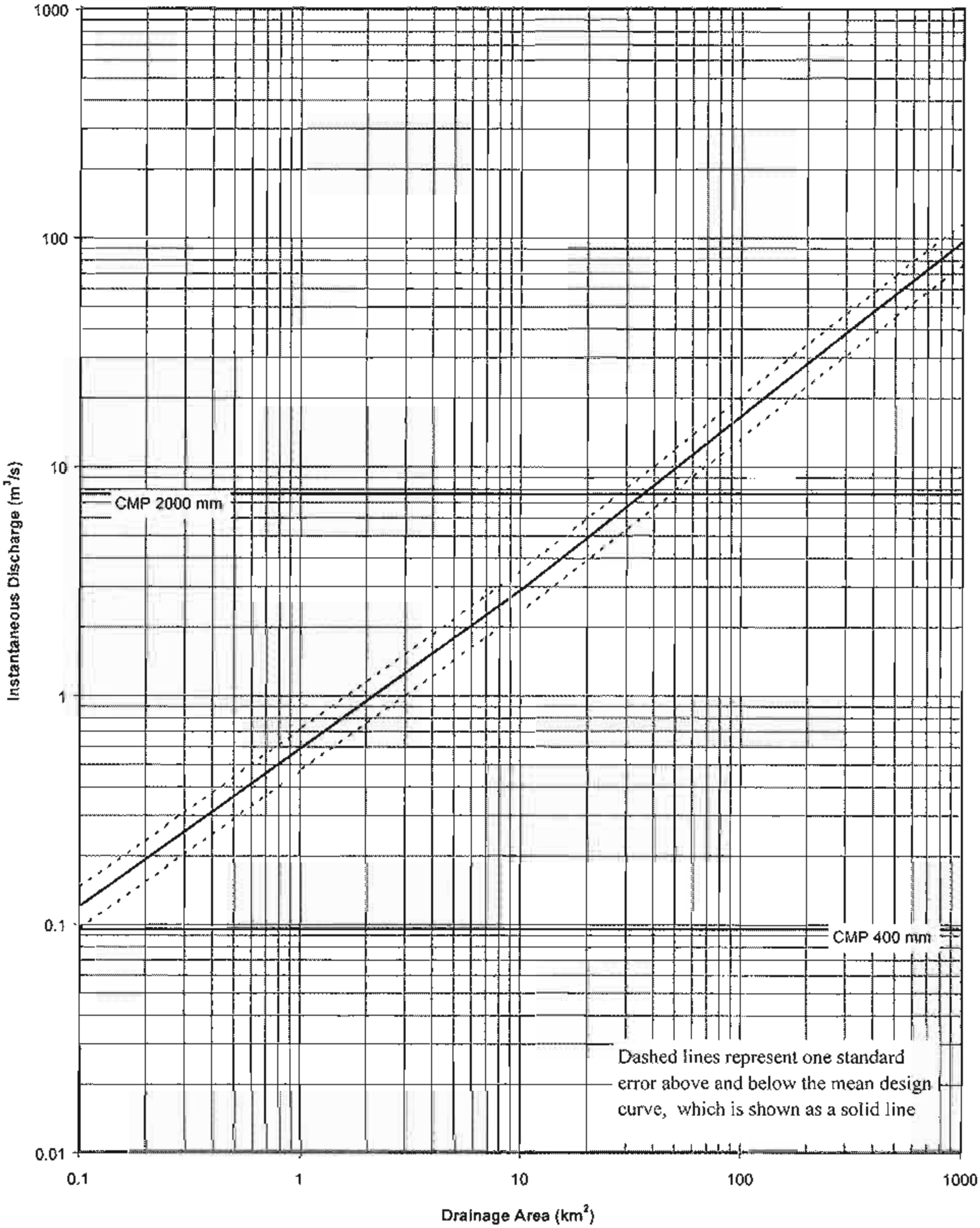


Figure 2.7. Recommended design curves for the 100-year return period annual maximum instantaneous discharge – Zone 3: Eastern Okanagan Highland and Okanagan Range.

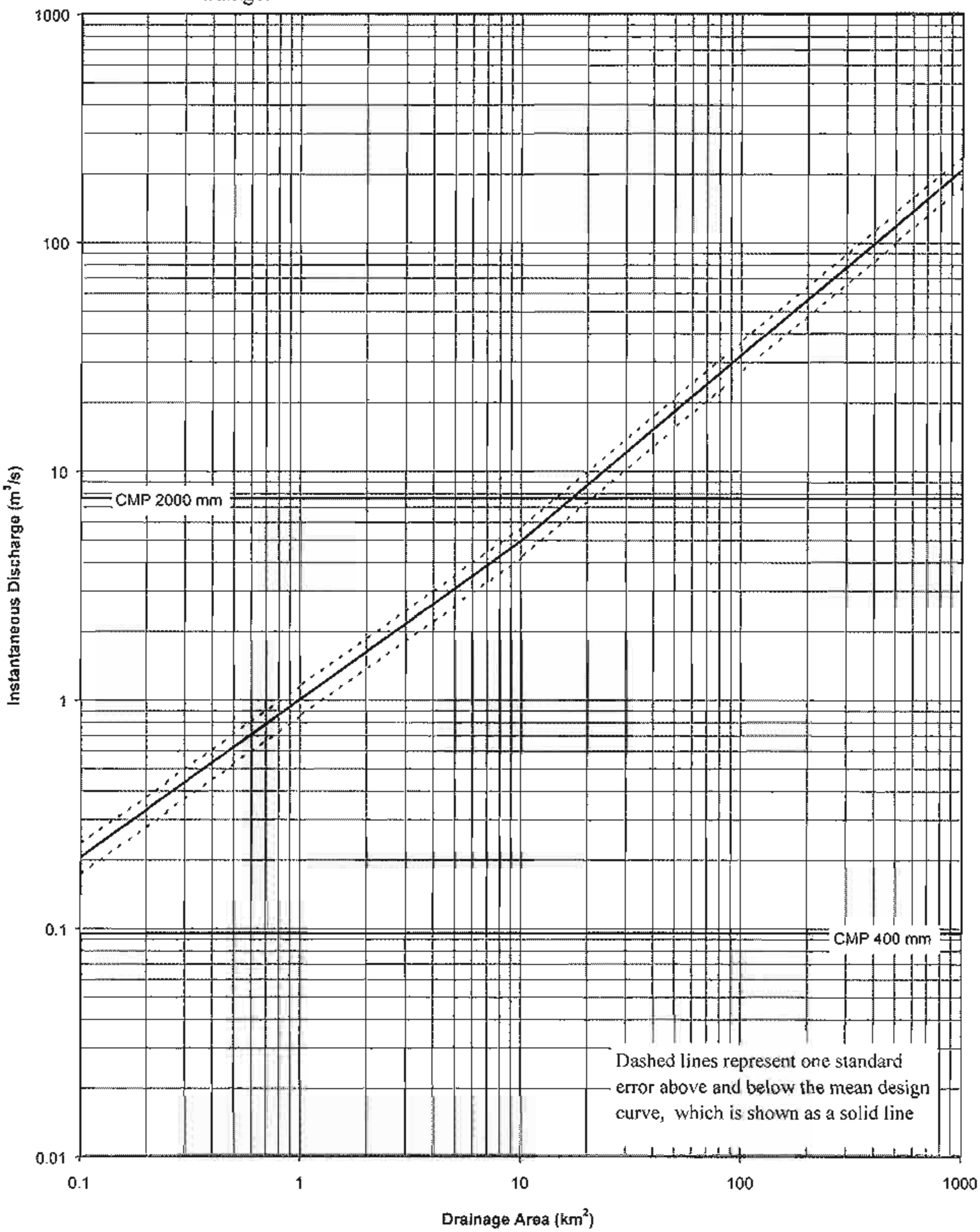
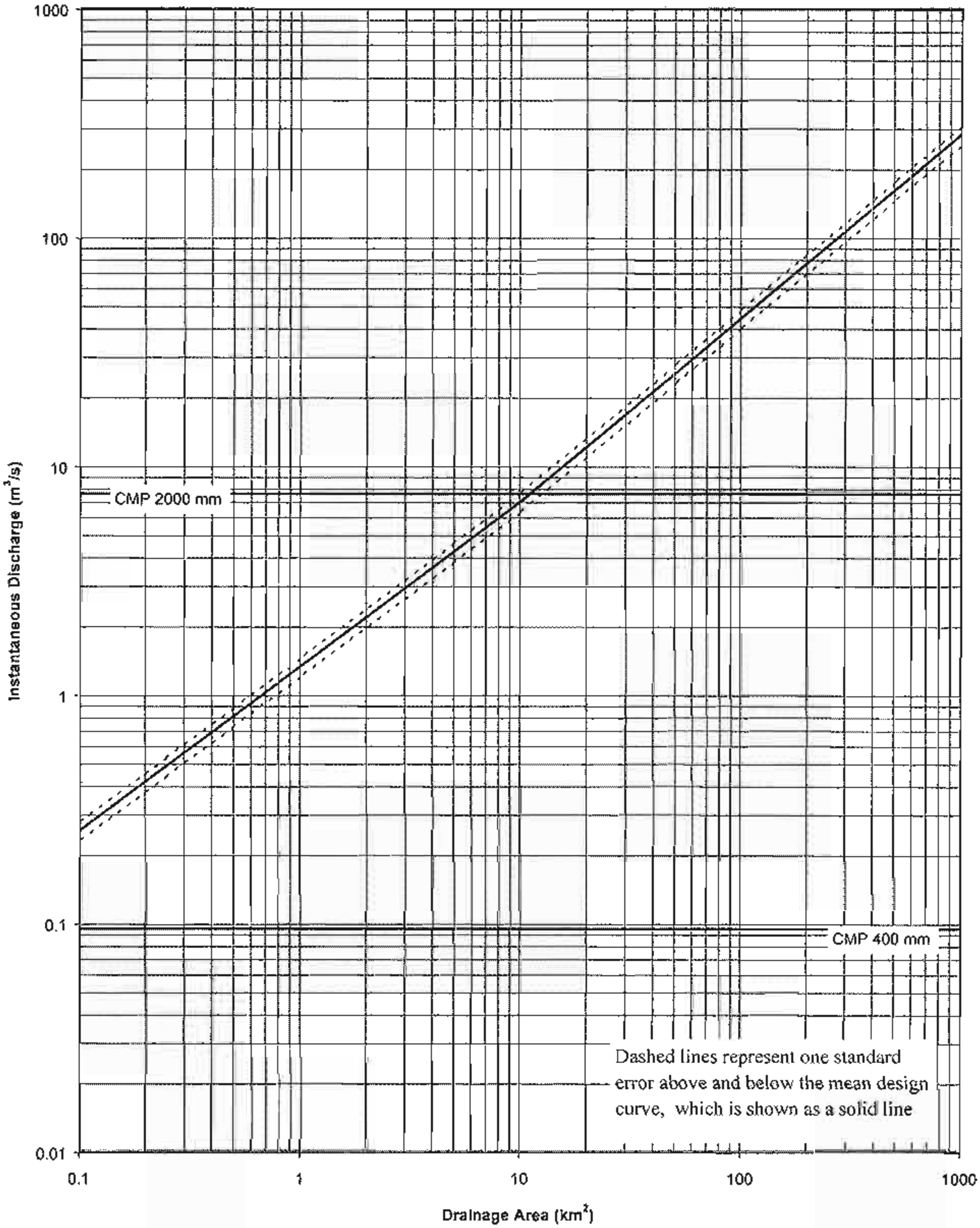


Figure 2.8. Recommended design curves for the 100-year return period annual maximum instantaneous discharge – Zone 4: Upper Kettle River.



3.0 CULVERT HYDRAULIC ANALYSIS

3.1 OVERVIEW

The procedures used to determine the culvert sizes required to pass design peak flows are outlined in this section. Section 3.2 addresses flow through circular Corrugated Metal Pipes (CMP) and Pipe Arch culverts. The equations for flow through a single, double and partially buried CMP installation are provided, as is an equation for flow through a single pipe arch {equations (3), (4), (5) and (6)}. The assumptions made during the derivation of these equations are listed. Section 3.3 addresses flow through simple rectangular and trapezoidal structures with natural bottoms. The equation from which the rectangular structure dimensions are calculated is provided {equation (7)}.

3.2 CIRCULAR CMP AND PIPE ARCH CULVERTS

The flow equations for the design of circular CMP and pipe arch installations have been derived using simplifying assumptions that apply to most forest road culvert installations, as listed below:

1. The culvert protrudes from the road fill on both the upstream and downstream sides of the road.
2. The culvert flows under inlet control (i.e. the downstream end is free flowing).
3. The water depth at the culvert inlet during the design flow does not exceed the top of the culvert[5].
4. The upstream end of the culvert is square to the flow, and has no sidewalls.

Under these conditions, the CMP culvert size, D [mm] required to pass a given design flow, Q [m^3/s] is:

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[5] The *Forest Practices Code of B.C Act* states that stream culverts in community watersheds must be designed to pass the design discharge without the depth of water at the culvert inlet exceeding the top of the culvert (Section 9(3), Part 2 of the Forest Road Regulation (March 1, 2000). *CULVERT* (Ver. 6.1) meets this requirement for all culvert installations.

$$D = 1000(Q/1.141)^{0.367} \tag{3}$$

If two identical CMP culverts are installed side-by-side at the location of interest, the size of each culvert required for both culverts together to pass the design flow is given as:

$$D = 1000(Q/2.282)^{0.367} \tag{4}$$

If a CMP is installed partially full of sediment at the location of interest, the size of the entire CMP required to pass the design flow is given as:

$$D = 1000(Q/1.141)^{0.367}(1 - F/D)^{-0.462} \tag{5}$$

where F is the depth to which the culvert is to be filled and D is the culvert diameter (e.g. if the culvert were one quarter full of sediment, F/D = 0.25). The user specifies the value of F/D during the operation of the program *CULVERT* (Ver. 6.1).

The equation to size a pipe arch considers both the span (S) and the rise (R) of the pipe arch. A characteristic length[6] (L) is used in the equation.

$$L = (SR)^{1/2} \qquad L = 1000(Q/0.973)^{0.390}$$

(6)

Using the above equations, the program *CULVERT* (Ver. 6.1) is able to compute required culvert sizes for single, double and partially buried circular culvert installations, as well as single pipe arch installations.

Note that especially in non-mountainous terrain, there may be locations at which the assumption of inlet control is invalid. In these cases, culvert sizing should be referred to a Professional Engineer (P. Eng.).

The *CULVERT* (Ver. 6.1) program provides CMP culvert sizes for design flows to the nearest millimetre. For pipe arch installations, the program rounds the computed size up to the next largest standard pipe arch size. Standard pipe arch sizes are provided in Table 3.1.

Table 3.1. Standard pipe arch sizes.

Span (mm)		Rise (mm)	L (mm)
7620	x	4240	5684
7040	x	4060	5346
6250	x	3910	4943
5890	x	3710	4675
5490	x	3530	4402
5050	x	3330	4101
4720	x	3070	3807
4370	x	2870	3541
3890	x	2690	3235
3730	x	2290	2923
3400	x	2010	2614
3100	x	1980	2477
2690	x	2080	2365
2590	x	1880	2207

Span (mm)		Rise (mm)	L (mm)
2440	x	1750	2066
2240	x	1630	1911
2060	x	1520	1770
2130	x	1400	1727
1880	x	1260	1539
1630	x	1120	1351
1390	x	970	1161
1150	x	820	971
1030	x	740	873
910	x	660	775
800	x	580	681
680	x	500	583
560	x	420	485
450	x	340	391

The *CULVERT* (Ver. 6.1) program also calculates the culvert sizes corresponding to a one standard error confidence interval using a flow equal to one standard error above and one standard error below the mean design flow. For new installations, the recommended culvert size is that calculated for the upper limit of this confidence interval. Use can be made of the one standard error confidence interval for evaluating the suitability of existing installations. Note that, since the program rounds pipe arch culvert sizes to the next highest standard size, there may be instances where the mean design culvert size is the same as the size at either the mean plus one standard error or the mean minus one standard error.

Equations (3), (4), (5) and (6) provide openings big enough to pass design flows, but not to pass debris. A final selection of culvert size should only be made after a site visit to assess local channel geometry and the potential for sediment and debris accumulation. A checklist identifying some of the issues to be considered in the field has been developed by Gary

[6] The characteristic length (L) is equivalent to the diameter of a CMP of equal cross-sectional area.

McClelland of the Kamloops Forest Region and is presented in Appendix B. If necessary, a larger size than specified by *CULVERT* (Ver. 6.1) should be selected.

The partially buried culvert is a design focussed on improving fish passage through the culvert to upstream areas. As such, the culvert size required to pass the 50-year or 100-year design discharge is only one consideration in selecting an appropriate culvert size. In addition, one must identify the species requiring passage (including information on the life stage and swimming ability) and the time of year during which passage is required. Typically, the installation is designed to maintain an average velocity low enough to allow fish passage for the species (and ages) present during high flows in the spring, and to maintain a water depth sufficient to allow fish passage for the species (and ages) present during low flows. The design of this type of culvert must also consider the hydraulics and sediment transport dynamics within the culvert over a range of flows. Specifically, partially buried culverts should be designed with consideration of:

1. velocities in the culvert barrel and at the culvert outlet,
2. degree of turbulence within the culvert and at the culvert outlet,
3. stability of the bed within the culvert at the design discharge, and
4. porosity of the bed within the culvert (to prevent subsurface flow during low flows).

During the design of a partially buried culvert installation, the culvert size calculated by *CULVERT* (Ver. 6.1) should be adjusted to account for these additional design considerations. Information on designing culverts to provide fish passage is provided in the *Stream Crossing Guidebook for Fish Streams* (Poulin and Argent, 1998), and in *Fish Passage Design at Road Culverts* (Washington Department of Fish and Wildlife, 1999).

3.3 RECTANGULAR STRUCTURES AND BRIDGES OVER TRAPEZOIDAL CHANNELS

The procedure for estimating required sizes for rectangular structures and bridges over trapezoidal channels (Figure 3.1) is based on a solution of the Manning equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2} \quad (7)$$

where:

- Q = design discharge [m^3/s]
- A = cross-sectional area of the structure [m^2]
- R = hydraulic radius of the structure [m]
- S = slope of the channel bed [m/m]
- n = Manning's roughness coefficient [$s\ m^{-1/3}$]

When solving for the rectangular structure dimensions, a user must specify the slope of the channel (S), the channel bottom material (from which Manning's n is derived), and the required span[7] (width) for the structure. Given these parameters *CULVERT* (Ver. 6.1) solves a re-arranged form of equation (7) using the calculated design discharge to obtain the structure height necessary to pass the design flow. Because the solution is valid for rectangular structures, it is useful for sizing log culverts and bridges for rectangular cross-sections. A user-specified freeboard allowance (with a default value of 0.6 meters) is added to the design height to account for debris and sediment accumulation at the upstream side of the structure.

A bridge over a natural channel (Figure 3.1) is different from that of a rectangular structure in that the channel has natural banks (assumed trapezoidal), and crossing construction can take place outside the wetted perimeter. When solving for the design dimensions, a user must specify the top width, bottom width and channel depth of the existing natural channel, as well as the channel slope and channel bottom material. The program iteratively solves for the design height (including a user-specified freeboard allowance) to pass the design discharge, based on the information provided. When the design height is greater than the channel depth (Case 1, Figure 3.1), the program also calculates the height of the abutments required to

[7] According to the Forest Road Engineering Guidebook (Sept. 1995), log culverts should have an opening with a minimum width of 1.5 m and a minimum height of 0.5 m.

elevate the structure to the design height. When the calculated design height is less than the channel height (Case 2, Figure 3.1), *CULVERT* (Ver. 6.1) calculates the span necessary to cross the trapezoid at the design height.

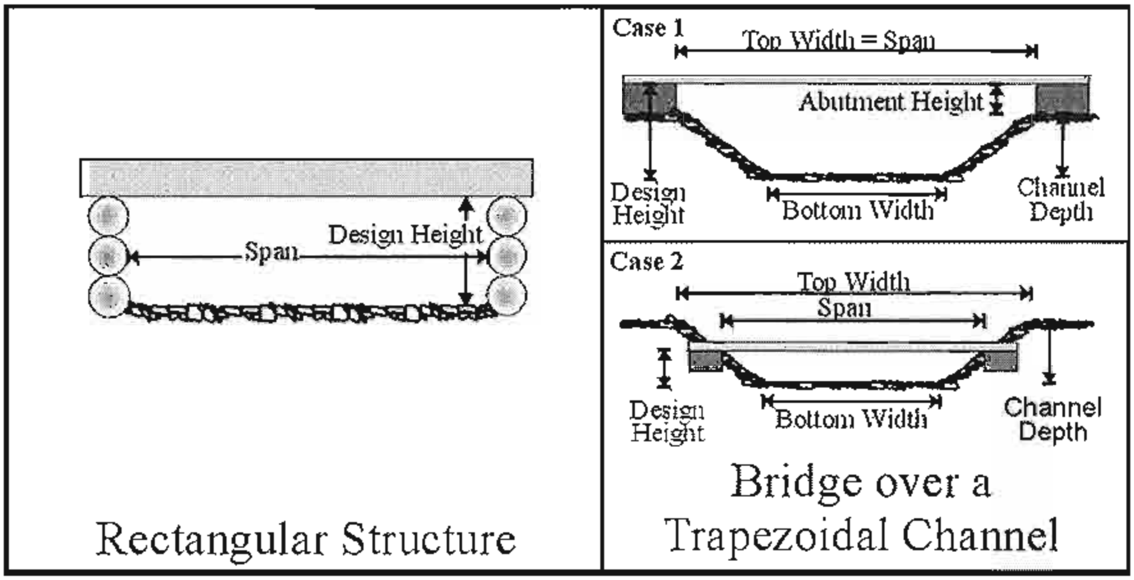

Figure 3.1. Schematic diagram of rectangular and trapezoidal structures.

Unlike for circular CMP and pipe arch culverts, estimates of the required dimensions for rectangular or trapezoidal installations cannot be made without measurements of the proposed installation locations. The user-specified freeboard allowance should reflect local knowledge of sediment and debris transport.

4.0 *CULVERT* COMPUTER PROGRAM

A brief discussion of the operation of *CULVERT* (Ver. 6.1) is provided here. A more complete discussion is provided in the *CULVERT* User's Manual (Summit, 2000). The program has been developed using the Microsoft® Visual Basic™ programming system for the Windows 95™ operating system.

In order for the *CULVERT* (Ver. 6.1) program to calculate the design discharge for a proposed crossing, the user must select the appropriate return period (either the 50-year or 100-year return period), the appropriate peak flow zone (Map 1), and specify the drainage area of the watershed upstream of the proposed crossing. The program then calculates the mean design flow estimate, plus a range in design flows corresponding to the upper and lower limits of a one standard error (68%) confidence interval about the mean design flow estimate.

The user then selects the desired type of installation; either:

- single, double or partially buried CMP culvert,
- single pipe arch culvert,
- rectangular structure, or
- bridge over trapezoidal channel.

The program computes a design installation size plus a range in sizes, based on the range in the design flow estimates. A recommended size for new installations is also provided, based on the upper limit of the one standard error confidence interval.

When the analysis for a given site is complete, the user has the option of saving the data, returning to the start, or exiting the program. At each stage of the process, the user has the option of returning to the previous screen or to the start of the program. It is possible to report and print results from any of the forms within the program, except the form requiring the user to specify the site code and drainage area. Two output options are possible: printing a hard copy table, and exporting the data to a Microsoft Excel™ spreadsheet file.

5.0 LIMITATIONS OF PEAK FLOW MODEL AND CULVERT (VER. 6.1) COMPUTER PROGRAM

Important considerations related to the use of the peak flow model and computer program are described here:

1. The peak flow model and *CULVERT* (Ver. 6.1) are valid only within the Penticton Forest District.
2. *CULVERT* (Ver. 6.1) can only be used by parties holding a valid licence agreement for this software.
3. The program is designed to estimate 50-year and 100-year return period peak instantaneous discharges in locations which are not affected by significant upstream flow regulation and which are not situated downstream of significant lake, wetland or reservoir storage. For crossings downstream of natural lakes or wetlands, we suggest that a design flow of 85% of the value provided by *CULVERT* (Ver. 6.1) is appropriate.
4. The peak flow model and *CULVERT* (Ver. 6.1) have been developed using data from drainage basins that are less than 5,000 km^2 in size. The peak flow model and computer program should not be used to calculate design discharges for drainage areas larger than 5,000 km^2.
5. The design flow methodology is intended to predict the 50-year and 100-year return period design flows for ungauged locations. However, there may be instances where design peak flow estimation based on specific gauged records is more appropriate.
6. To account for uncertainty, the program uses the upper 68% confidence limit on the mean design flow as the recommended design discharge.
7. Common hydraulic assumptions which are generally valid for forest roads have been made in order to simplify the calculation of required circular CMP culvert sizes: inlet control, ponded depth at design flow does not exceed the level of the culvert top, the culvert entrance is square to the flow and has no sidewalls, and the culvert protrudes from the fill at both ends.

8. The program is intended to provide consistent estimates of opening sizes required to hydraulically pass design flows. Sizes may be modified by particular field conditions such as locations where sediment and/or debris loading is expected. Site inspections are recommended for all proposed culvert installations before a final culvert size is chosen.

Finally, it should be noted that the *Forest Practices Code of British Columbia Act* stipulates several provisions related to crossing design in part 2 of the Forest Road Regulation. Some of the key provisions are listed here:

1. All major[8] culverts must be designed by a Professional Engineer (P.Eng.). During operation of *CULVERT* (Ver. 6.1), the user is cautioned whenever a major culvert is encountered. Although the estimated design discharge provided by the program will be valid, the design elements of such installations should be referred to a Professional Engineer. In these cases, culvert recommendations provided by the program must be viewed with caution. The program is not intended to replace a proper design by a Professional Engineer (P.Eng.).
2. The minimum design peak flow for stream culverts and for permanent bridges is the 100-year return period annual maximum instantaneous discharge. The minimum design peak flow for temporary bridges is the 50-year return period annual maximum instantaneous discharge.
3. With some exceptions[9], all bridges must be designed by a Professional Engineer (P.Eng.).

Users of the program are advised to become familiar with these and other relevant provisions of the Code.

[8] A major culvert is defined under Part 1 of the Forest Road Regulation (March 1, 2000) to be "a stream culvert having a pipe diameter of 2,000 mm or greater, or a maximum design discharge of 6 m^3/s or greater."

[9] See Section 10, Part 1 of the Forest Road Regulation under the *Forest Practices Code of British Columbia Act* (British Columbia Provincial Government, March 1, 2000).

6.0 REFERENCES

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Appendix A

DESIGN PEAK FLOW ANALYSIS:
PENTICTON FOREST DISTRICT

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A.1 INTRODUCTION

A peak flow hydrology study was conducted for the Penticton Forest District (Figure A.1). The purpose of the study was to develop a reliable and consistent methodology for estimating design flows within the Penticton Forest District. The study was a component of a larger scale project to develop a user-friendly method to derive consistent estimates of required culvert sizes within the Penticton Forest District. The results of the hydrology study have been incorporated into the computer program *CULVERT* (Ver. 6.1) which is a Windows 95^{TM} compatible computer program that has been customised specifically for the Penticton Forest District. The program *CULVERT* (Ver. 6.1) performs the required calculations – its operation is summarised in a separate report (Summit, 2000).

The peak flow hydrology study included two major components:

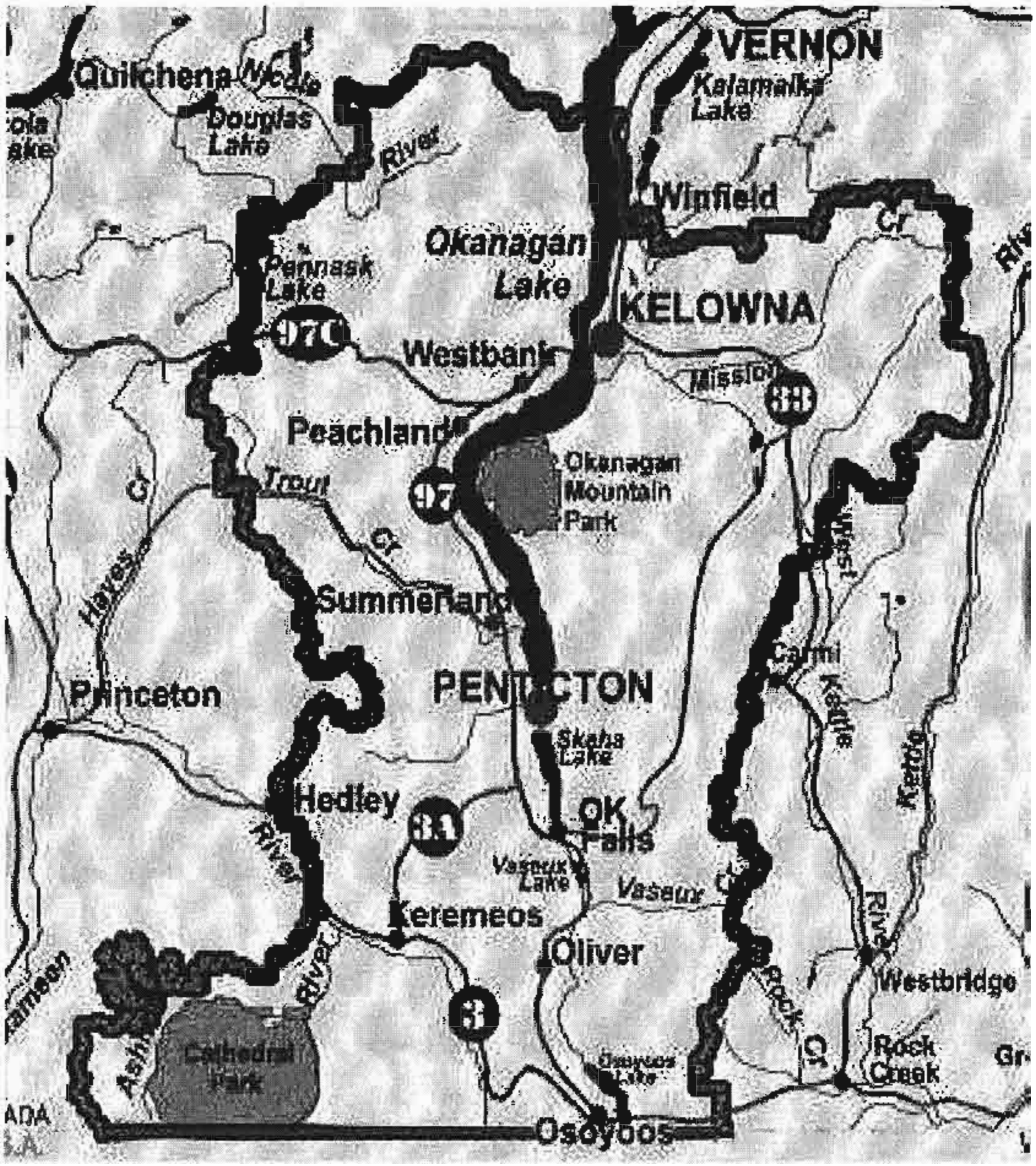
1. review of existing data, information, reports, and analyses relevant to the hydrology of the Penticton Forest District (Figure A.1), and
2. development of an appropriate methodology for estimating the required design peak flows, which comprises both the Index Flood Method and the Rational Formula Method.

This Appendix describes the methods employed during the study, and the results obtained.

A.2 PREVIOUS INFORMATION

Relevant information includes discharge data collected at many gauging stations – operated by Environment Canada and by the United States Geological Survey (USGS) – during the past 100 years in the interior of British Columbia and in Washington State. Data for the present study were extracted from the Hydat CD-ROM published by Environment Canada (Environment Canada, 1996) and downloaded from the USGS web page (http://waterdata.usgs.gov/nwis-w/us). These gauging stations have operated over periods ranging from 4 to 54 years. Intensity-duration-frequency curves for precipitation records,

Figure A.1. Penticton Forest District study area.



Penticton District Boundary

collected by the Atmospheric Environment Service were also used (Environment Canada 1991). Another useful data source was an unpublished report presenting a methodology for applying the Rational Formula to discharge estimation (Reksten and Davies, unpub.).

A.3 DESIGN PEAK FLOW METHODOLOGY

The methodology chosen to estimate design peak flows for the Penticton Forest District (the study area) is the Index Flood Method for drainage areas greater than 10 km^2, and a modified version of the Rational Formula for drainage areas less than 10 km^2 (after Reksten and Davies, unpub.). The methodology is consistent with the estimation procedure recommended by Reksten (1987) for the estimation of peak flows at ungauged locations in B.C.

The intent of the hydrologic analysis is to provide a reliable and consistent estimate of the 50- and 100-year return period peak flows for ungauged locations within the study area. Given statistical uncertainty in the design peak flow estimate, a one standard error (68%) confidence interval in the design peak flow estimate is computed, the upper limit of which is adopted as the recommended design peak flow for new installations.

A.3.1 INDEX FLOOD METHOD

A.3.1.1 Overview of the Method

The Index Flood Method is commonly used for estimation of peak discharges at ungauged locations in Canada (National Research Council of Canada, 1989) and is entirely consistent with the methodology presented in the Community Watershed Guidebook (p. 70-72, MoF/MELP, 1996). Relevant data from gauged locations in the vicinity of the locations where estimates are required is used to derive an appropriate model for calculating the annual maximum daily discharge (the index flood). Ratios of higher return period (e.g. 50-year and

100-year) annual maximum daily discharges to the index flood are then determined[1]. Finally, an average ratio of the peak instantaneous to the peak daily discharge is determined[2].

The maximum instantaneous discharge {calculated using equations (2) and (3), Section A.3.1.5} is used as the basis for crossing design. In the present application of the Index Flood Method, a one standard error (68%) confidence interval is provided for the design discharge estimate. The confidence interval is based upon the combined standard errors of the index flood, the instantaneous-to-daily peak discharge ratio, and the ratios of the 50-year and 100-year return period flows to the index flood.

A.3.1.2 Regional Model for Index Flood

Historic streamflow data from hydrometric stations in the interior of British Columbia and northern Washington State were examined for relevance to the study area. Data from 62 gauging stations were selected for use in developing the appropriate models for the index flood. This data set does not include stations on streams that are regulated or that are downstream of major lakes or wetlands, nor does it include stations with less than 4 years of peak flow record. All of the gauging stations have drainage areas less than 5,000 km^2. Stations larger than this typically integrate the hydrologic characteristics of several zones, and therefore cannot be used.

The data for each station was used to calculate the mean annual peak daily discharge for the period of record. The estimate was then adjusted to a long term mean based on the flow records for several stations with records extending back to near the beginning of the 20th century. This reduces the effect of an individual station's particular period of record. The adjusted mean annual peak daily discharges for the 62 stations are the "index floods" for those stations.

[1] These ratios are used to scale the index flood upwards to the desired return period.
[2] This ratio is used to scale peak daily discharges to peak instantaneous discharges.

These values were then transformed to a scale-independent estimate of the discharge per square kilometre[3], and plotted on a 1:250,000 scale base map. Spatial relationships between the streamflow gauging stations were then examined, and four zones homogeneous with respect to peak flows were delineated within the study area. Both the peak flow zones and the gauge locations are shown on Map 1 of the main report. The zones are:

1. Zone 1: Southern Okanagan Basin,
2. Zone 2: Northern Okanagan Basin,
3. Zone 3: Eastern Okanagan Highland and Okanagan Range, and
4. Zone 4: Upper Kettle River.

The index flood values (annual maximum daily discharge) for hydrometric stations relevant to each of the four homogeneous peak flow zones are plotted against drainage area in Figure A.2. Lines drawn on Figure A.2 represent least-squares regression lines through the data; dashed lines represent a 90% confidence interval about the mean. Table A.1 presents the equations represented by the least-squares lines, as well as regression diagnostic parameters. As data presented on Figure A.2 and the sample calculation presented in Table A.1 demonstrates, the zones are distinct from one another. Estimates of the index flood made for a drainage area of 50 km^2 for each of the zones are different from each other at a confidence level of 90%. Summaries of relevant data for the stations used within each peak flow zone are provided in Table A.2.

[3] Peak discharge per unit area (Q_{p2}/A) is known to be a function of drainage area (A). This effect must be removed before basins of different sizes can be compared. For the purpose of zone delineation, we have done this by calculating and mapping $k = Q_{p2}/A^{0.75}$, where Q_{p2} is the index flood. The exponent 0.75 is approximately correct throughout B.C.

Table A.1. Least-squares regression equations and diagnostics for the index flood vs. drainage area.

Zone	Regression Equation[1, 2]	R^2	Standard error of estimate [in log units]	Sample Calculation ($A = 50\ km^2$)		
				Estimate $[m^3/s]$	90% Confidence Interval[3]	
					Upper Limit $[m^3/s]$	Lower Limit $[m^3/s]$
1	$LOG(Q_{p2}) = 0.779LOG(A) - 1.230$	0.830	0.152	1.24	2.11	0.642
2	$LOG(Q_{p2}) = 0.760LOG(A) - 0.756$	0.962	0.088	3.43	4.79	2.45
3	$LOG(Q_{p2}) = 0.811LOG(A) - 0.573$	0.992	0.063	6.37	8.08	5.01
4	$LOG(Q_{p2}) = 0.802LOG(A) - 0.293$	0.995	0.037	11.7	13.3	10.3

1: Q_{p2} is the mean annual maximum daily discharge $[m^3/s]$

2: A is the drainage area $[km^2]$

3: equal to ± 1.65 standard errors from the mean.

A.3.1.3 Return Period Ratios

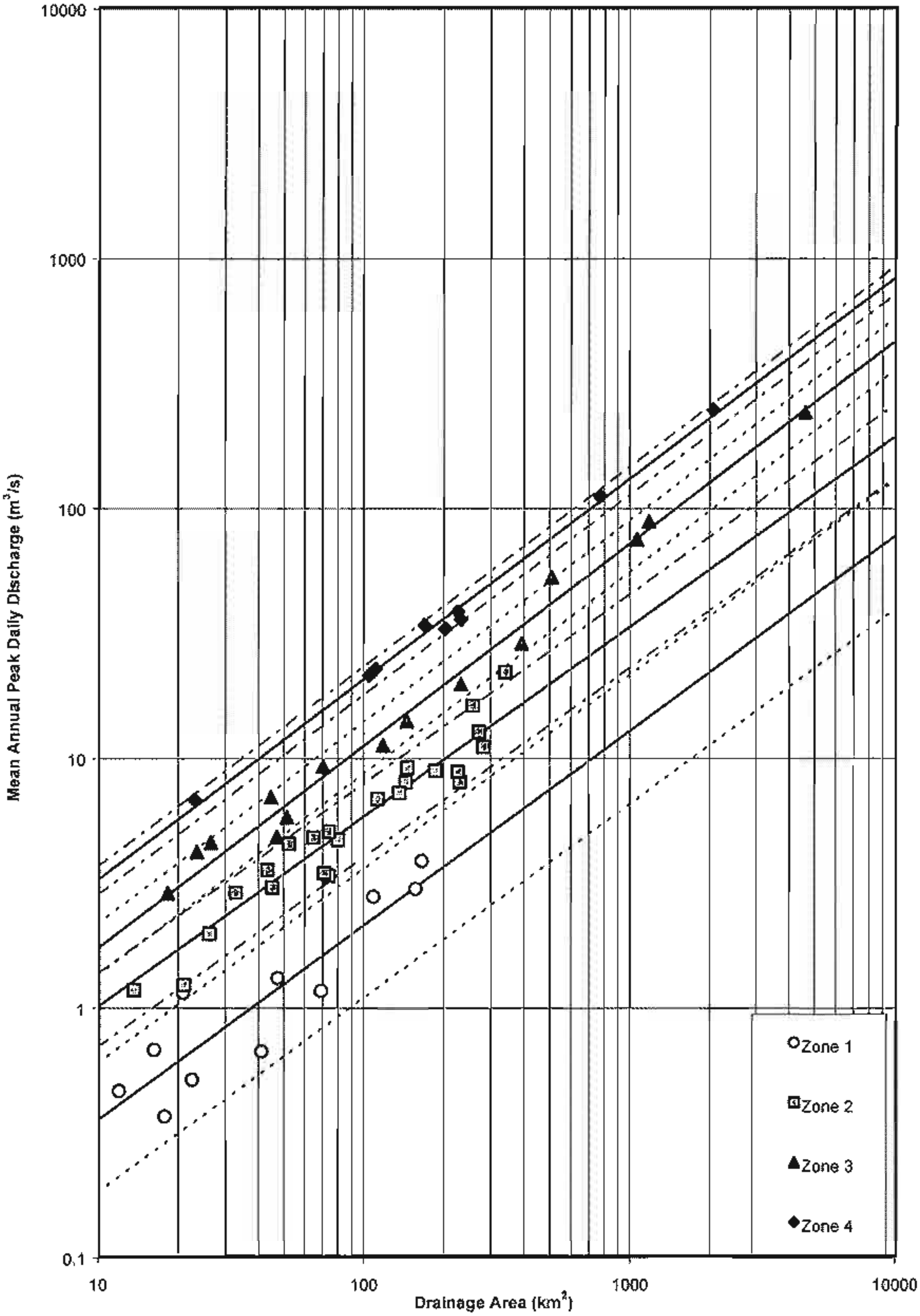
For stations with appropriately long records[4], return period analysis has been performed and the 50- and 100-year return period annual peak daily discharges have been calculated. Four different distribution types were fit to the data: Pearson Type III, Log Pearson Type III, Log Normal and Gumbel distributions. Each distribution type was qualitatively assessed for goodness of fit, and where the fit was questionable, the Kolmogorov-Smirnov test was applied. Additionally, the expected value of the skew of the distribution was assessed for the Log Normal and the Gumbel distribution[5]. For a Log Normal distribution, the expected skew of the log of discharge is 0.0 {i.e. $Log(Q_p)$ is normally distributed}. The expected skew for the Gumbel distribution is given by:

$$\gamma = 1.14(1 + 8.5/n) \tag{1}$$

[4] For estimating the 50-year return period a minimum record length of 14 years was selected. For the 100-year return period, a minimum record length of 20 years was selected. These are based on guidelines for flood frequency analysis in B.C., presented in Reksten (1987).

[5] The Pearson Type III and Log Pearson Type III include skew as a parameter of the distribution, and therefore skew has no fixed expected value.

Figure A.2. Mean annual peak daily discharge vs. drainage area.



where γ is the expected skew, and n is the number of points to which the Gumbel distribution has been fit. Based on these tests, several of the distributions were rejected for some of the stations used in the analysis (Table A.3). The estimate of the 50-year and 100-year return period annual peak daily discharge is given by the average of all valid distributions. Using these averages, a representative ratio of the 50-year return period annual peak daily discharge to the index flood (Q_{p50}/Q_{p2}) and of the 100-year return period annual peak daily discharge to the index flood (Q_{p100}/Q_{p2}) were calculated for the four homogeneous zones. Additional data from an adjacent and physiographically similar peak flow zone (Zone 5, not addressed in this report) have been included in the return period ratio analysis for Zone 4. The flood generating mechanisms producing the Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} ratios are similar in Zones 4 and 5. The data from Zones 2 and 3 have been combined, because taken separately, there is no statistical difference between the ratio estimates. These two zones are hydrologically and physiographically similar, which justifies combining the data sets. By increasing the number of data points used to estimate the Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} ratios for Zones 4 (by using data from outside Zone 4) and for Zones 2 and 3 (by combining the data from Zones 2 and 3) the precision of the estimates is increased.

Preliminary analysis of the flood frequency data indicated that gauges with larger stations had smaller Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} ratios than gauges with smaller drainage areas. Additionally, the larger stations tended to integrate the effects of more than one peak flow zone. To address this potential problem, only stations with drainage areas less than 1,000 km^2 were used to determine the Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} ratios.

A.3.1.4 Instantaneous to Daily Peak Flow Ratio

Several methods can be employed to estimate a representative ratio of peak instantaneous to peak daily discharge. We have examined all paired observations[6] of instantaneous and daily peak flows for all the stations listed in Table A.3. The instantaneous to daily ratios (I/D) for a given station were found to be unrelated to the discharge return period (i.e. the I/D ratio was the same for the mean annual flood as for a flood with a much higher return period). Also, it was observed that the I/D ratios were higher and more variable for drainage basins smaller than 300 km^2 than for basins larger than 300 km^2. Based on these findings, we have calculated the average I/D ratio using all paired observations of instantaneous and daily peak flows from only the stations with drainage areas less than 300 km^2. There are 14 stations and 188 total observations suitable for this calculation. The data and the mean I/D ratios are presented in Table A.4. We have applied I/D ratios of:

- 1.16 to Zone 1, based on 58 observations from four stations in Zone 1,
- 1.16 to Zone 4, based on 35 observations from three stations in Zones 4 and 5, and
- 1.15 to Zones 2 and 3, based on 95 observations from seven stations in Zones 2 and 3.

[6] A paired observation is comprised of a daily peak discharge and an instantaneous peak discharge from the same flood event (i.e. within the same 24 hour period).

Table A.2. Discharge gauging stations used in regional peak flow analysis.

Peak flow zone	Station Name	Station No.	Drainage Area (km^2)	Period of Record	Record Length (years)	Adjusted[1] mean annual maximum daily discharge	
						m^3/s	$L/s/km^2$
1	BULL CREEK NEAR CRUMP	08NM133	46.9	1965-1986	21	1.30	27.7
1	GREATA CREEK NEAR THE MOUTH	08NM173	40.7	1971-1998	26	0.660	16.2
1	HAYNES CREEK NEAR OSOYOOS	08NM126	17.6	1959-1964	4	0.362	20.6
1	INKANEEP CREEK NEAR OLIVER	08NM012	164	1920-1950	18	3.82	23.3
1	KEREMEOS CREEK ABOVE MARSEL CREEK	08NL014	68.6	1920-1927	8	1.15	16.8
1	MCLEAN CREEK NEAR OKANAGAN FALLS	08NM005	20.7	1921-1926	6	1.14	54.9
1	SINLAHEKIN CREEK ABOVE BLUE LAKE NEAR LOOMIS [2]	1244000	108	1924-1930	7	2.748	25.4
1	SOUKUP CREEK NEAR HEDLEY	08NL035	22.3	1965-1979	14	0.509	22.8
1	SPECTACLE LAKE TRIB NEAR LOOMIS [2]	12443700	11.9	1961-1976	16	0.458	38.5
1	TONASKET CREEK AT OROVILLE [2]	12439300	156	1950-1991	26	2.957	19.0
1	TREHEARNE CREEK NEAR PRINCETON	08NL037	16.1	1965-1979	14	0.671	41.7
2	ASP CREEK NEAR PRINCTON	08NL015	51.8	1960-1969	10	4.48	86.4
2	BELVEVUE CREEK NEAR OKANAGAN	08NM035	73.3	1921-1986	28	5.00	68.3
2	BOLEAN CREEK AT FALKLAND	08LE001	228	1911-1964	15	7.92	34.7
2	BOLEAN CREEK NEAR THE MOUTH	08LE094	224	1975-1986	11	8.71	38.9
2	BRASH CREEK NEAR ENDERBY	08LC004	32.6	1916-1967	10	2.86	87.6
2	CANOE CREEK NEAR SALMON ARM	08LE005	279	1911-1962	12	10.9	39.2
2	DRY CREEK TRIB NEAR MOLSON[2]	12439200	4.35	1958-1977	20	0.515	118
2	EAST CANOE CREEK ABOVE DAM	08LE108	20.8	1983-1998	14	1.12	53.8
2	FERRY CREEK NEAR LUMBY	08LC034	145	1959-1977	18	9.06	62.5
2	FISHTRAP CREEK NEAR MCLURE	08LB024	135	1915-1998	32	6.89	51.0
2	INKANEEP CREEK NEAR OLIVER	08NM082	70.4	1942-1945	4	3.42	48.6
2	LOUIS CREEK AT BOUNDARY OF RAILWAY BELT	08LB010	269	1912-1980	6	12.6	46.8
2	MONTE CREEK ABOVE MONTE LAKE DIVERSION	08LE103	64.3	1982-1994	12	4.74	73.8
2	MOODY CREEK NEAR CHRISTINA	08NN021	13.5	1972-1984	13	1.16	86.2
2	SALMON RIVER ABOVE SALMON LAKE	08LE075	143	1966-1998	31	7.86	55.0
2	SHINGLE CREEK ABOVE KALEDEN DIVERSION	08NM038	44.8	1920-1971	14	2.99	66.7
2	SILVER CREEK NEAR SALMON ARM	08LE043	25.9	1923-1948	11	1.95	75.4
2	SIWASH CREEK TRIB NEAR TONASKET[2]	12444400	1.71	1957-1977	20	0.331	193.5
2	TOATS COULEE CREEK NEAR LOOMIS[2]	12442000	337	1920-1979	31	21.8	64.6
2	TRINITY CREEK ABOVE DIVERSION	08LC048	42.9	1981-1984	4	3.51	81.9
2	VANCE CREEK BELOW DEFIES CREEK	08LC040	73.3	1970-1998	25	3.29	44.9
2	VASEAUX CREEK ABOVE DUTTON CREEK	08NM015	255	1920-1982	27	16.0	62.7

Table A.2. Discharge gauging stations used in regional peak flow analysis (continued).

Peak flow Zone	Station Name	Station No.	Drainage Area (km²)	Period of Record	Record Length (years)	Adjusted[1] mean annual maximum daily discharge	
						m^3/s	$L/s/km^2$
2	WATCHING CREEK NEAR KAMLOOPS	08LF049	79.5	1950-1974	13	4.65	58.5
2	WHIPSAW CREEK BELOW LAMONT CREEK	08NL036	185	1965-1998	32	8.71	47.1
2	WHITEMAN CREEK ABOVE BOULEAU CREEK	08NM174	112	1971-1998	26	6.87	61.3
3	ASHNOLA RIVER NEAR KEREMEOS	08NL004	1050	1915-1998	54	74.1	70.6
3	CHERRY CREEK NEAR CHERRYVILLE	08LC049	503	1982-1990	9	52.1	104
3	CORNING CREEK NEAR SQUILAX	08LE077	26.2	1979-1998	17	4.53	173
3	DENNIS CREEK NEAR 1780 METRE CONTOUR	08NM242	3.73	1985-1998	12	0.702	188
3	HEDLEY CREEK NEAR THE MOUTH	08NL050	389	1974-1998	23	28.1	72.1
3	HELLER CREK ABOVE DIVERSIONS	08LF091	46.6	1983-1994	11	4.56	97.8
3	HIUHILL CREEK ABOVE DIVERSIONS	08LD002	69.7	1917-1998	22	8.39	120
3	JAMIESON CREEK ABOVE DIVERSIONS	08LB083	230	1990-1993	4	19.5	84.6
3	KETTLE RIVER AT KETTLE VALLEY	08NN004	4560	1941-1921	8	238	52.3
3	MANN CREEK NEAR BLACKPOOL	09LB050	295	1927-2981	20	30	102
3	MARA CREEK NEAR SICAMOUS	08LC024	23.3	1946-1949	4	4.16	179
3	MIDDLE FORK TOATS COULEE CREEK NEAR LOOMIS[2]	12441700	44.3	1965-1975	10	6.86	155
3	ONYX CREEK NEAR MAGNA	08LE028	50.8	1915-1925	4	5.72	113
3	POOLEY CREEK ABOVE POOLEY DITCH	08NM210	18.1	1974-1979	5	2.84	157
3	TRAPPING CREEK NEAR THE MOUTH	08NN019	144	1966-1998	31	14.1	97.7
3	TWO FORTY-ONE CREEK NEAR PENTICTON	08NM241	4.5	1984-1998	13	0.905	201
3	VASEAUX CREEK ABOVE SOLCO CREEK	08NM171	117	1971-1998	26	10.9	93.1
3	WEST KETTLE RIVER BELOW CARMI CREEK	08NN022	1170	1974-1995	22	87.3	74.6
4	BARNES CREEK NEAR NEEDLES	08NE077	201	1951-1998	46	32.4	161
4	BURRELL CREEK ABOVE GLOUCESTER CREEK	08NN023	224	1974-1998	23	38.3	171
4	GRANBY RIVER AT GRAND FORKS	08NN002	2050	1914-1998	36	244	119
4	HARPER CREEK NEAR THE MOUTH	08LB076	168	1973-1998	23	33.3	198
4	RAFT RIVER NEAR CLEARWATER	08LB017	764	1915-1957	11	110	144
4	ROSS CREEK NEAR ANGLEMONT	08LE029	104	1915-1925	4	21.1	203
4	TRAPPING CREEK AT 1220 M CONTOUR	08NN020	22.8	1971-1981	11	6.72	295
4	WEST KETTLE RIVER NEAR MCCULLOCH	08NN015	230	1965-1998	31	35.7	155

1: These values have been adjusted to the long-term mean of one of the following stations, depending on location (Thompson River at Spences Bridge, station 08LF022 & 08LF051; Kettle River near Laurier, station 08NN012; or Columbia River at Nicholson, station 08NA002). 2: These stations are located in Washington State, U.S.A.

Table A.3. Return period ratio analysis.

Zone	Station	Drainage area [km²]	Years of data	Annual maximum daily discharge estimates [m³/s]									Average [m³/s]		Ratios	
				Index flood	Pearson III		Log Pearson III		Log Normal		Gumbel					
				(Q_{p2})	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}/Q_{p2}	Q_{p100}/Q_{p2}
5	08LE024	904	39	220	327	345	328	347	329	349	n/a	n/a	328	347	1.49	1.58
5	08ND009	655	29	143	282	308	291	328	n/a	n/a	294	323	289	320	2.02	2.24
5	08ND012	938	36	206	375	414	374	421	n/a	n/a	375	410	375	415	1.82	2.01
5	08ND014	272	25	107	200	220	200	226	n/a	n/a	204	224	201	223	1.88	2.09
5	08ND019	112	26	33.3	62.8	69.6	64.4	73.6	n/a	n/a	65.0	72.1	64.1	71.8	1.92	2.16
5	09LE027	805	44	201	307	327	297	314	298	315	320	345	306	325	1.52	1.62
4	08LB076	168	25	33.6	52.8	56.7	52.2	56.0	51.9	55.6	56.7	62.0	53.4	57.6	1.59	1.71
4	08NE077	201	48	32.4	51.4	54.7	52.2	56.0	50.1	52.9	53.9	58.1	51.9	55.4	1.60	1.71
4	08NE110	298	22	34.4	63.0	69.4	62.5	69.6	n/a	n/a	65.9	72.9	63.8	70.6	1.86	2.06
4	08NN015	230	33	35.4	55.2	58.0	56.7	59.9	58.3	62.3	n/a	n/a	56.7	60.1	1.60	1.70
4	08NN023	224	25	38.0	56.7	59.4	58.4	62.0	57.1	60.2	62.1	67.0	58.6	62.2	1.54	1.64
Average for Zones 4 & 5															**1.71**	**1.86**
Standard error for Zones 4 & 5															**0.057**	**0.074**
3	08LD002	69.7	24	9.13	21.6	24.2	24.1	28.5	22.1	25.4	23.5	26.6	22.8	26.2	2.50	2.87
3	08LE077	26.2	19	4.54	9.88	11.1	10.1	11.8	9.1	10.1	10.3	12.8	9.8	11.5	2.17	2.52
3	08NL050	389	25	28.4	65.1	70.9	65.7	70.5	n/a	n/a	75.0	84.6	68.6	75.3	2.42	2.65
3	08NM171	117	28	11.1	23.2	24.9	25.4	27.9	28.6	32.6	27.1	30.3	26.1	28.9	2.35	2.61
3	08NM241	4.5	15	0.91	1.59	1.68	1.58	1.66	1.84	2.03	1.87	2.07	1.72	1.86	1.89	-
3	08NM242	3.73	14	0.68	1.16	1.24	1.21	1.31	1.18	1.27	1.30	1.42	1.21	1.31	1.77	-
3	08NN019	144	33	13.9	21.2	22.0	21.1	21.8	23.9	25.6	24.6	26.7	22.7	24.0	1.63	1.73
2	08LB024	135	34	7.17	14.1	15.6	16.8	19.0	17.1	19.6	16.6	18.6	16.2	18.2	2.25	2.54
2	08LC040	73.3	27	3.34	6.39	6.88	6.83	7.46	7.50	8.46	7.36	8.23	7.02	7.76	2.10	2.32
2	08LE001	228	15	7.92	14.4	-	14.3	-	17.3	-	17.0	-	15.8	-	1.99	-
2	08LE075	143	31	7.89	14.6	15.7	15.4	16.8	16.5	18.5	16.5	18.4	15.7	17.3	2.00	2.20
2	08LE108	20.8	16	1.22	3.08	3.48	3.25	3.78	3.30	3.87	3.33	3.79	3.24	3.73	2.66	-
2	08NL036	185	34	8.79	22.7	25.9	21.7	24.6	22.0	25.1	23.2	26.1	22.4	25.4	2.55	2.89
2	08NM035	78.3	28	5.00	10.8	11.8	11.4	12.9	10.8	11.9	11.7	13.0	11.2	12.4	2.23	2.48
2	08NM174	112	28	6.87	13.9	15.0	14.3	15.4	n/a	n/a	16.0	17.8	14.7	16.1	2.14	2.34
Average for Zones 2 & 3															**2.17**	**2.48**
Standard error for Zones 2 & 3															**0.087**	**0.120**

Note: The flood producing mechanisms generating the ratios Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} are similar for stations in Zones 4 and 5 and for stations in Zones 2 and 3, and have been combined for the analysis. "-" indicates that record length is insufficient to make this calculation. "n/a" indicates that the distribution does not fit the data, and has been rejected.

Table A.3. Return period ratio analysis (continued).

Zone	Station	Drainage area [km²]	Years of data	Annual peak daily discharge estimates [m³/s] Index flood	Pearson III		Log Pearson III		Log Normal		Gumbel		Average [m³/s]		Ratios	
				(Q_{p2})	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}	Q_{p100}	Q_{p50}/Q_{p2}	Q_{p100}/Q_{p2}
1	08NL035	22.3	14	0.51	1.8	2.1	2.2	3.0	1.9	2.4	1.9	2.2	2.0	2.4	3.89	4.77
1	08NL037	16.1	14	0.67	2.6	3.0	n/a	n/a	n/a	n/a	2.9	3.4	2.8	3.2	4.15	4.82
1	08NM012	164	18	3.82	14.6	17.4	15.1	18.8	15.6	19.6	14.6	16.9	15.0	18.1	3.92	4.75
1	08NM133	46.9	21	1.30	4.1	4.7	4.2	4.9	4.8	5.9	4.3	4.9	4.3	5.1	3.34	3.91
1	08NM164	13.0	14	0.21	0.71	0.81	0.93	1.17	0.98	1.26	0.79	0.91	0.85	1.04	4.06	4.94
1	08NM173	40.7	28	0.66	2.4	2.7	2.9	3.5	3.5	4.6	2.5	2.9	2.8	3.4	4.27	5.19
														Average for Zone 1	**3.94**	**4.73**
														Standard error for Zone 1	**0.133**	**0.177**

Note: The flood producing mechanisms generating the ratios Q_{p50}/Q_{p2} and Q_{p100}/Q_{p2} are similar for stations in Zones 4 and 5 and for stations in Zones 2 and 3, and have been combined for the analysis. "-" indicates that record length is insufficient to make this calculation. "n/a" indicates that the distribution does not fit the data, and has been rejected.

Table A.4. Instantaneous to daily peak flow ratios.

Zone	Station	Area	Year	Instantaneous Discharge	Daily Discharge	I/D ratio
1	08NM012	164	1923	3.34	2.97	1.12
			1924	1.23	1.10	1.12
			1925	1.59	1.47	1.08
			1927	2.53	2.21	1.14
			1944	6.34	4.76	1.33
			1945	7.93	5.95	1.33
			1946	7.08	6.06	1.17
			1948	7.42	5.83	1.27
			1949	3.62	3.14	1.15
			1950	6.03	5.07	1.20
1	08NM133	46.9	1965	0.951	0.900	1.06
			1968	2.14	1.45	1.4
			1969	0.909	0.816	1.11
			1970	0.399	0.345	1.16
			1971	2.77	1.92	1.44
			1972	5.38	4.36	1.23
			1973	0.643	0.547	1.18
			1974	3.34	3.03	1.10
			1975	1.63	1.47	1.11
			1976	1.53	1.18	1.30
			1977	0.241	0.221	1.09
			1978	2.16	1.86	1.16
			1979	0.651	0.582	1.12
			1980	1.70	1.24	1.37
			1981	2.22	1.86	1.19
			1983	0.934	0.883	1.06
			1984	1.37	1.12	1.22
			1985	1.02	0.850	1.20
			1986	2.40	2.05	1.17
1	08NM164	13.0	1971	0.430	0.382	1.13
			1973	0.045	0.042	1.07
			1975	0.215	0.204	1.05
			1976	0.178	0.122	1.46
			1978	0.348	0.261	1.33
			1981	0.331	0.198	1.67
			1986	0.202	0.179	1.13
1	08NM173	40.7	1971	1.23	1.07	1.15
			1974	1.69	1.57	1.08
			1975	1.20	1.11	1.08
			1976	0.898	0.852	1.05
			1977	0.176	0.161	1.09
			1978	0.960	0.943	1.02
			1979	0.212	0.198	1.07
			1980	0.266	0.258	1.03
			1981	0.258	0.241	1.07

Note: Area refers to the drainage area (km^2), instantaneous discharge and daily discharge refer to the annual maxima (m^3/s), and I/D is the ratio of the instantaneous and daily annual maxima for data occurring within one day of each other.

Table A.4. Instantaneous to daily peak flow ratios (continued).

Zone	Station	Area	Year	Instantaneous Discharge	Daily Discharge	I/D ratio
1	08NM173	40.7	1982	0.611	0.583	1.05
			1983	0.705	0.683	1.03
			1984	1.05	1.00	1.05
			1985	0.314	0.270	1.16
			1986	0.717	0.690	1.04
			1987	0.334	0.293	1.14
			1988	0.076	0.057	1.33
			1989	0.316	0.293	1.08
			1991	0.704	0.667	1.06
			1993	0.676	0.657	1.03
			1994	0.175	0.167	1.05
			1995	0.779	0.744	1.05
			1996	1.25	1.21	1.03
Selected statistics for I/D ratio, Zone 1: average = 1.16 standard error = 0.0176 N = 58						
2	08LB024	135	1972	14.0	12.7	1.10
			1973	3.77	3.37	1.12
			1974	8.72	8.35	1.04
			1975	12.0	10.3	1.17
			1977	4.11	3.91	1.05
			1979	4.33	4.18	1.04
			1980	3.31	3.09	1.07
			1981	3.78	3.67	1.03
			1982	9.64	9.23	1.04
			1984	9.55	8.29	1.15
			1985	9.27	8.86	1.05
			1986	4.13	4.05	1.02
			1987	11.7	10.2	1.15
			1988	3.60	3.43	1.05
			1989	5.85	5.37	1.09
			1991	3.98	3.89	1.02
			1992	3.88	3.75	1.03
			1995	9.67	9.39	1.03
2	08LE075	143	1966	3.91	3.71	1.05
			1967	10.6	9.34	1.13
			1968	7.87	6.97	1.13
			1969	9.37	8.75	1.07
			1970	5.15	4.30	1.20
			1971	15.1	13.6	1.11
			1972	14.3	13.9	1.03
			1973	7.42	6.74	1.10
			1974	11.9	11.0	1.08
			1975	10.3	9.68	1.06
			1976	6.68	5.64	1.18
			1977	3.14	3.00	1.05
			1978	9.60	8.47	1.13
			1979	5.61	5.04	1.11
			1980	5.66	4.84	1.17
			1981	8.24	6.63	1.24
			1982	8.51	8.25	1.03

Note: Area refers to the drainage area (km^2), instantaneous discharge and daily discharge refer to the annual maxima (m^3/s), and I/D is the ratio of the instantaneous and daily annual maxima for data occurring within one day of each other.

Table A.4. Instantaneous to daily peak flow ratios (continued).

Zone	Station	Area	Year	Instantaneous Discharge	Daily Discharge	I/D ratio
2	08LE075	143	1983	9.76	8.72	1.12
			1984	8.75	8.16	1.07
			1985	7.67	7.54	1.02
			1987	8.23	7.16	1.15
			1988	7.69	5.48	1.40
			1989	5.68	5.57	1.02
			1990	10.6	10.3	1.03
			1991	8.31	7.80	1.07
			1992	3.53	3.29	1.07
			1993	13.1	12.1	1.08
			1994	4.53	4.20	1.08
			1995	6.35	5.49	1.16
			1996	15.4	13.1	1.18
2	08LE108	20.8	1983	3.55	2.99	1.19
			1984	0.591	0.528	1.12
			1985	0.492	0.465	1.06
			1986	0.727	0.696	1.04
			1987	1.18	1.01	1.17
			1988	0.989	0.818	1.21
			1989	0.877	0.839	1.05
			1990	1.71	1.32	1.30
			1991	0.997	0.964	1.03
			1993	1.61	1.46	1.10
			1994	1.26	1.22	1.03
			1995	0.886	0.859	1.03
			1996	1.41	1.28	1.10
2	08LF049	79.5	1963	4.05	3.65	1.11
			1965	3.65	3.31	1.10
			1967	5.21	3.88	1.34
			1968	7.31	6.63	1.10
			1969	7.42	6.40	1.16
			1970	3.51	3.23	1.09
			1971	10.0	9.00	1.11
			1972	10.1	8.58	1.18
			1973	5.10	4.13	1.23
			1974	6.40	5.58	1.15
2	08LG064	85	1983	3.51	3.30	1.06
			1984	7.22	6.55	1.10
			1985	4.81	4.33	1.11
			1986	4.87	4.61	1.06
			1988	2.63	2.34	1.12
			1989	2.76	2.63	1.05
			1991	8.66	7.30	1.19
			1992	1.31	1.17	1.12
			1993	8.74	7.71	1.13
			1994	4.68	4.19	1.12
			1996	17.5	12.5	1.40
3	08LB083	230	1990	33.5	24.7	1.36
			1991	17.3	10.8	1.60
			1992	12.4	10.0	1.24
			1993	34.7	26.8	1.29

Note: Area refers to the drainage area (km^2), instantaneous discharge and daily discharge refer to the annual maxima (m^3/s), and I/D is the ratio of the instantaneous and daily annual maxima for data occurring within one day of each other.

Table A.4. Instantaneous to daily peak flow ratios (continued).

Zone	Station	Area	Year	Instantaneous Discharge	Daily Discharge	I/D ratio
3	08LE077	26.2	1982	8.15	6.25	1.30
			1983	4.82	3.28	1.47
			1986	7.45	5.35	1.39
			1987	7.18	4.88	1.47
			1988	7.69	5.75	1.34
			1989	5.54	3.78	1.47
			1991	3.81	2.88	1.32
			1992	3.07	2.29	1.34
			1994	3.05	2.39	1.28
Selected statistics for I/D ratio, Zones 2 and 3: average = 1.15 standard error = 0.0126 N = 95						
4	08LB076	168	1974	54.9	48.7	1.13
			1977	20.9	17.4	1.20
			1978	36.5	30.9	1.18
			1980	31.1	28.3	1.10
			1981	44.7	33.5	1.33
			1982	37.5	32.1	1.17
			1983	46.0	39.0	1.18
			1984	42.7	37.4	1.14
			1985	37.6	34.8	1.08
			1986	45.8	39.1	1.17
			1987	33.4	31.4	1.06
			1988	42.4	36.0	1.18
			1990	32.4	24.7	1.31
			1991	34.0	29.3	1.16
			1992	39.1	31.3	1.25
			1993	35.6	31.4	1.13
			1994	25.4	22.6	1.12
			1996	37.5	31.7	1.18
5	08LB038	280	1985	79.5	72.4	1.10
			1986	98.5	84.8	1.16
			1988	65.2	61.9	1.05
			1989	74.0	67.3	1.10
			1990	60.5	55.0	1.10
			1992	79.8	67.6	1.18
			1994	73.1	57.6	1.27
			1995	61.8	55.8	1.11
			1996	68.5	65.9	1.04
5	08LE086	253	1973	69.9	60.6	1.15
			1974	95.7	79.0	1.21
			1975	79.9	71.1	1.12
			1976	79.0	65.4	1.21
			1977	70.8	60.0	1.18
			1978	60.6	53.2	1.14
			1979	62.9	55.8	1.13
			1980	53.6	48.2	1.11
Selected statistics for I/D ratio, Zone 4: average = 1.16 standard error = 0.0112 N = 35						

Note: Area refers to the drainage area (km^2), instantaneous discharge and daily discharge refer to the annual maxima (m^3/s), and I/D is the ratio of the instantaneous and daily annual maxima for data occurring within one day of each other.

A.3.1.5 Index Flood Method Design Equations

Design peak flows using the Index Flood Method for watersheds larger than 10 km^2 are determined as follows:

$$Q_{p50i} = (I/D)(Q_{p50}/Q_{p2})(Q_{p2}) \quad (2)$$

$$Q_{p100i} = (I/D)(Q_{p100}/Q_{p2})(Q_{p2}) \quad (3)$$

where:

- Q_{p50i} = 50-year return period annual maximum instantaneous discharge to be estimated [m^3/s],
- Q_{p100i} = 100-year return period annual maximum instantaneous discharge to be estimated [m^3/s],
- I/D = representative ratio of instantaneous to daily peak discharge,
- Q_{p2} = index flood (mean annual maximum daily discharge)[7] [m^3/s],
- Q_{p50}/Q_{p2} = representative ratio of 50-year return period annual maximum daily discharge to the index flood, and

Q_{p100}/Q_{p2} = representative ratio of 100-year return period annual maximum daily discharge to the index flood.

A.3.1.6 Error Analysis

Uncertainty in the estimated design peak flow derives from possible errors in the raw peak flow hydrometric data (associated primarily with gauge locations and rating curves), differences in the value of data from each station due to differences in the length and the period of record, and possible errors due to the nature of the gauging station (manual or recording). Furthermore, while we have attempted to account for major differences in location, elevation and aspect in defining homogeneous peak flow zones, differences in these

[7] The index flood is calculated based on the drainage area upstream of the proposed stream crossing.

and other factors (such as forested area, degree of land-use development, soils and geology) persist within each zone.

To account for this source of uncertainty, a one standard error (68%) confidence interval is provided around the mean design peak flow estimate. The true value of the design peak flow is estimated to fall within the upper and lower confidence limits 68% of the time. Standard errors in the design peak flow estimates are based on the combined standard errors in the annual maximum daily discharge, the instantaneous to daily peak flow ratio, and the ratios of the 50- and 100-year return period to the mean annual peak daily discharge. The standard errors of the 50- and 100-year return period design discharge are estimated as:

$$SE(Q_{p50i}) = \mathrm{Q}_{p50i}\left(\left(\frac{SE_{I/D}}{I/D}\right)^2 + \left(\frac{SE_{Q_{p50}/Q_{p2}}}{Q_{p50}/Q_{p2}}\right)^2 + \left(\frac{SE_{Q_{p2}}}{Q_{p2}}\right)^2\right)^{1/2} \tag{4}$$

$$SE(Q_{p100i}) = \mathrm{Q}_{p100i}\left(\left(\frac{SE_{I/D}}{I/D}\right)^2 + \left(\frac{SE_{Q_{p100}/Q_{p2}}}{Q_{p100}/Q_{p2}}\right)^2 + \left(\frac{SE_{Q_{p2}}}{Q_{p2}}\right)^2\right)^{1/2} \tag{5}$$

where $SE(Q_{p50i})$ and $SE(Q_{p100i})$ are the estimated standard errors in the 50- and 100-year return period maximum instantaneous discharge; $SE_{Q_{p50}/Q_{p2}}$ and $SE_{Q_{p100}/Q_{p2}}$ are the estimated standard errors in the mean ratio of the 50- and 100-year return period peak daily discharge to the mean annual peak daily discharge; $SE_{I/D}$ is the standard error in the mean instantaneous to daily ratio; and $SE_{Q_{p2}}$ is the standard error in the mean annual peak daily discharge. The standard error in the mean annual peak daily discharge is assumed to be a constant percentage of the mean annual peak daily discharge. Since the relation of mean annual peak daily discharge to drainage area is log-transformed, standard errors in mean annual peak daily discharge must be transformed back into linear form before equations (4) and (5) are applied. A summary of mean and standard error values used in equations (4) and (5) is provided in Table A.5.

The one standard error (68%) confidence interval about the mean design peak flow estimate is equivalent to the mean discharge estimate plus or minus the standard error in the design peak flow estimate.

Table A.5. Summary of mean and standard error statistics used in equations (2) to (5).

A. Zone 1: Southern Okanagan Basin.

Parameter	Mean	Standard error [%]	n
I/D	1.16	1.52	58
Q_{p50}/Q_{p2}	3.94	3.38	6
Q_{p100}/Q_{p2}	4.73	3.74	6

B. Zone 2: Northern Okanagan Basin and Zone 3: Eastern Okanagan Highland and Okanagan Range.

Parameter	Mean	Standard error [%]	n
I/D	1.15	1.10	95
Q_{p50}/Q_{p2}	2.17	4.01	15
Q_{p100}/Q_{p2}	2.48	4.84	11

C. Zone 4: Upper Kettle River.

Parameter	Mean	Standard error [%]	n
I/D	1.16	0.966	35
Q_{p50}/Q_{p2}	1.71	3.33	11
Q_{p100}/Q_{p2}	1.86	3.98	11

D. Index flood (Q_{p2}) for all zones.

Zone	Mean [m^3/s] [1]	Standard error above the mean [m^3/s] / [%] [2]	Standard error below the mean [m^3/s] / [%] [2]	n
1	1.06	0.450 / *42.5*	0.310 / *29.2*	11
2	4.33	0.970 / *22.4*	0.800 / *18.5*	25
3	10.8	1.70 / *15.7*	1.42 / *13.1*	17
4	36.9	3.00 / *8.13*	2.80 / *7.58*	9

1: This value is based on the data presented in Table A.2, representing different ranges in drainage areas for the different zones. These values are therefore not directly comparable, however the standard errors reported in percent are directly comparable.

2: Because the index flood is estimated based on a logarithmic relation, the standard error of the estimate is given in Log units which, when back-transformed to arithmetic units, produces asymmetrical standard errors about the mean.

A.3.2 RATIONAL FORMULA METHOD

Although the data set includes four stations with drainage areas less than 10 km^2, the regional methodology is generally considered suspect[8] for watersheds smaller than about 10 km^2.

[8] There are two reasons for this: first, most gauges are on streams larger than 10 km^2, and second, small basins behave differently, hydrologically, than larger ones (this a function of the typical size and/or intensity of flood-generating precipitation events).

Therefore, the Rational Formula Method of estimating flows (as presented in Reksten and Davies, unpublished) was applied to each of the peak flow zones for watersheds less than 10 km^2 in area. This involved analysis of precipitation records from climate stations at Oliver, Vernon and Revelstoke (Environment Canada, 1991). The precipitation records were used in conjunction with the Rational Formula, which is commonly used to estimate design peak flows for small watersheds in B.C. The equation used to calculate the mean annual maximum instantaneous discharge is:

$$Q_{p2i} = \frac{0.28CPA}{T_c} \tag{6}$$

Where:

Q_{p2i}	= mean annual maximum instantaneous discharge [m^3/s]
C	= runoff coefficient [dimensionless]
A	= drainage area [km^2]
T_c	= time-of-concentration[9] [hrs]
P	= total precipitation occurring within the time-of-concentration [mm] during the mean annual maximum precipitation event.

We have made calculations of Q_{p2i} for a number of drainage basin sizes ranging from 0.1 km^2 to 10 km^2. The values of C were taken from Table 1 in Reksten and Davies (unpublished) which expresses C as a function of the surface cover (impermeable, forested, agricultural, rural or urban) and physiography (mountain, steep slope, moderate slope, rolling terrain or flat). The table also suggests that the contribution of melting snow to the runoff event be accounted for by adding a constant (0.10) to the runoff coefficient. Similarly, values of T_c were taken from Figure 1 in Reksten and Davies; in which curves relating T_c to A are presented for different physiographic classifications. The values of P were taken from intensity-duration-frequency (IDF) curves for three climate stations that are representative of

[9] This is the time required for surface runoff generated at the most distant point in the drainage basin to reach the point-of-interest (i.e. the potential crossing location)

the study area: Vernon, Oliver and Revelstoke. The parameters used and the estimated discharges are presented in Table A.6.

Table A.6. Rational Formula method parameters and estimates.

	Runoff Coefficient (C)			Time-of-concentration (T_c) [hrs]			Total Precipitation (P) [mm][4]			Mean Annual Max. Inst. Discharge (Q_{p2i}) [m^3/s]		
Zone #	1[1]	2&3[2]	4[3]	1[1]	2&3[2]	4[3]	1[5]	2&3[6]	4[7]	1	2&3	4
10 km²	0.90	0.75	0.90	3.50	7.20	3.50	11.2	14.7	14.9	8.04	4.30	10.71
7 km²	0.90	0.75	0.90	2.80	6.00	2.80	10.5	13.9	13.7	6.60	3.40	8.62
5 km²	0.90	0.75	0.90	2.40	5.00	2.40	10.0	13.1	12.9	5.27	2.74	6.78
3 km²	0.90	0.75	0.90	1.80	4.00	1.80	9.2	12.1	11.6	3.88	1.91	4.86
1 km²	0.90	0.75	0.90	1.15	2.35	1.15	8.3	10.2	9.79	1.78	0.908	2.14
0.7 km²	0.90	0.75	0.90	0.96	2.00	0.96	7.7	9.63	9.14	1.42	0.708	1.68
0.5 km²	0.90	0.75	0.90	0.83	1.70	0.83	7.4	9.13	8.66	1.13	0.564	1.31
0.3 km²	0.90	0.75	0.90	0.68	1.40	0.68	7.0	8.56	8.03	0.78	0.385	0.893
0.1 km²	0.90	0.75	0.90	0.44	0.88	0.44	6.2	7.34	6.82	0.35	0.175	0.391

1. assumes steep slope physiography.
2. assumes moderate slope physiography.
3. assumes steep slope physiography.
4. based on time-of-concentration.
5. using the intensity-duration-frequency curve for Oliver.
6. using the intensity-duration frequency curve for Vernon.
7. using the intensity-duration-frequency curve for Revelstoke.

Once the estimates had been made, a power equation relating Q_{p2I} and A was derived using a log-log regression of the same form as used for the index flood method (see Table A.3). The equations and R^2 for each zone are presented in Table A.7.

Table A.7. Rational Formula method equations.

Zone	Equation	R^2
1	$Q_{p2i} = 1.77A^{0.679}$	0.999
2 & 3	$Q_{p2i} = 0.891A^{0.694}$	0.999
4	$Q_{p2i} = 2.13A^{0.723}$	0.999

These equations were used in the present study only as a guide; specifically, the exponents of the Rational Formula method equations were used to extrapolate the index flood-derived best-fit lines down to drainage areas smaller than 10 km^2. Standard error estimates derived from the Index Flood Method were assumed to be similar in magnitude for drainage areas less than 10 km^2.

A.4 DESIGN PEAK FLOW CURVES

The recommended design peak flow curves for the study area, developed according to the methodology outlined in section A.3, are presented in Figures A.3 to A.10. A 68% confidence interval about the design peak flow estimate is indicated by the dashed lines in the above figures. On each curve, the discharge passable by 400 mm and 2000 mm corrugated metal pipes (CMPs) are indicated, which facilitates identification of required culvert sizes according to watershed area, as outlined in Table A.8. The equations used to generate the mean, 68% upper confidence limit, and 68% lower confidence limit design discharges are presented in Table A.9. Example design peak flow estimates are presented in Tables A.10 and A.11. When designing new culvert or bridge installations, it is recommended to base the design peak flow on the upper confidence interval curve. When evaluating the suitability of existing installations, the upper and lower confidence intervals can be used to provide reasonable bounds around the "true" design peak flow.

Table A.8 Relationship between design culvert size and watershed area for 400 mm and 2000 mm CMPs.

Zone	Return Period	Watershed area upstream of new[1] culverts of the following diameter:	
		400 mm	2000 mm
1	50	0.152 km^2 (15.2 ha)	72.2 km^2
	100	0.116 km^2 (11.6 ha)	57.1 km^2
2	50	0.087 km^2 (8.7 ha)	42.4 km^2
	100	0.072 km^2 (7.2 ha)	35.6 km^2
3	50	0.040 km^2 (4.0 ha)	19.9 km^2
	100	0.033 km^2 (3.3 ha)	16.9 km^2
4	50	0.028 km^2 (2.8 ha)	12.1 km^2
	100	0.025 km^2 (2.5 ha)	11.0 km^2

1: data based on "recommended" design peak flow (mean design peak flow plus one standard error)

Table A.9. Design peak flow equation summary.

Peak flow zone	return period	Drainage Area [km^2]	Design Equations		
			Standard error below the mean [%]	Mean value [m^3/s]	Standard error above the mean [%]
1	50-year	≥10	29.5	$Q_{p50i} = 0.269A_i^{0.779}$	42.6
		<10	29.5	$Q_{p50i} = 0.338A_i^{0.679}$	42.6
	100-year	≥10	29.5	$Q_{p100i} = 0.323A_i^{0.779}$	42.6
		<10	29.5	$Q_{p100i} = 0.406A_i^{0.679}$	42.6
2	50-year	≥10	18.9	$Q_{p50i} = 0.437A_i^{0.760}$	22.8
		<10	18.9	$Q_{p50i} = 0.509A_i^{0.693}$	22.8
	100-year	≥10	19.1	$Q_{p100i} = 0.499A_i^{0.760}$	22.9
		<10	19.1	$Q_{p100i} = 0.582A_i^{0.693}$	22.9
3	50-year	≥10	14.4	$Q_{p50i} = 0.666A_i^{0.811}$	15.4
		<10	14.4	$Q_{p50i} = 0.873A_i^{0.693}$	15.4
	100-year	≥10	14.7	$Q_{p100i} = 0.761A_i^{0.811}$	15.6
		<10	14.7	$Q_{p100i} = 0.998A_i^{0.693}$	15.6
4	50-year	≥10	8.80	$Q_{p50i} = 1.02A_i^{0.802}$	9.58
		<10	8.80	$Q_{p50i} = 1.23A_i^{0.720}$	9.58
	100-year	≥10	9.10	$Q_{p100i} = 1.10A_i^{0.802}$	9.82
		<10	9.10	$Q_{p100i} = 1.33A_i^{0.720}$	9.82

Table A.10. Example design peak flow estimates: 50-year return period.

Drainage Area (km^2)	Zone 1 [m^3/s]			Zone 2 [m^3/s]			Zone 3 [m^3/s]			Zone 4 [m^3/s]		
	lower	mean	upper	lower	mean	upper	lower	mean	upper	lower	mean	upper
1000	41.2	58.5	83.4	67.5	83.2	102	155	181	208	235	257	282
500	24.0	34.1	48.6	39.9	49.1	60.3	88.1	103	119	135	147	162
100	6.86	9.73	13.9	11.7	14.5	17.8	23.9	27.9	32.2	37.0	40.6	44.5
50.0	4.00	5.67	8.09	6.93	8.54	10.5	13.6	15.9	18.4	21.2	23.3	25.5
10.0	1.14	1.62	2.30	2.04	2.51	3.08	3.69	4.31	4.97	5.84	6.40	7.01
5.00	0.710	1.01	1.44	1.26	1.55	1.91	2.28	2.66	3.07	3.54	3.89	4.26
1.00	0.238	0.338	0.482	0.413	0.509	0.625	0.748	0.873	1.01	1.11	1.22	1.34
0.500	0.149	0.211	0.301	0.255	0.315	0.387	0.462	0.540	0.623	0.675	0.741	0.812
0.250	0.093	0.132	0.188	0.158	0.195	0.239	0.286	0.334	0.386	0.410	0.450	0.493
0.100	0.050	0.071	0.101	0.084	0.103	0.127	0.152	0.177	0.204	0.212	0.232	0.255

Drainage Area (km^2)	Zone 1 [$L/s/km^2$]			Zone 2 [$L/s/km^2$]			Zone 3 [$L/s/km^2$]			Zone 4 [$L/s/km^2$]		
	lower	mean	upper	lower	mean	upper	lower	mean	upper	lower	mean	upper
1000	41.2	58.5	83.4	67.5	83.2	102	155	181	208	235	257	282
500	48.1	68.2	97.2	79.7	98.3	121	176	206	238	269	295	323
100	68.6	97.3	139	117	145	178	239	279	322	370	406	445
50.0	79.9	113	162	139	171	210	272	318	367	424	465	510
10.0	114	162	230	204	251	308	369	431	497	584	640	701
5.00	142	202	288	252	311	381	456	533	615	709	777	852
1.00	238	338	482	413	509	625	748	873	1,010	1,110	1,220	1,340
0.500	298	422	602	511	630	773	925	1,080	1,250	1,350	1,480	1,620
0.250	372	528	753	632	779	957	1,140	1,340	1,540	1,640	1,800	1,970
0.100	499	708	1,010	837	1,030	1,270	1,520	1,770	2,040	2,120	2,320	2,550

Table A.11. Example design peak flow estimates: 100-year return period.

Drainage Area (km^2)	Zone 1 [m^3/s]			Zone 2 [m^3/s]			Zone 3 [m^3/s]			Zone 4 [m^3/s]		
	lower	mean	upper	lower	mean	upper	lower	mean	upper	lower	mean	upper
1000	49.5	70.2	100	76.9	95.1	117	176	206	239	256	281	309
500	28.8	40.9	58.4	45.4	56.2	69.0	100	118	136	146	160	176
100	8.23	11.7	16.7	13.4	16.5	20.3	27.2	31.9	36.9	40.1	44.1	48.5
50.0	4.80	6.81	9.71	7.90	9.76	12.0	15.5	18.2	21.0	23.0	25.3	27.8
10.0	1.37	1.94	2.76	2.32	2.87	3.53	4.20	4.92	5.69	6.33	6.96	7.64
5.00	0.854	1.21	1.73	1.44	1.77	2.18	2.60	3.05	3.52	3.84	4.23	4.64
1.00	0.286	0.406	0.579	0.471	0.582	0.715	0.851	0.998	1.15	1.21	1.33	1.46
0.500	0.179	0.254	0.362	0.291	0.360	0.442	0.527	0.617	0.714	0.732	0.806	0.885
0.250	0.112	0.158	0.226	0.180	0.223	0.274	0.326	0.382	0.442	0.445	0.489	0.537
0.100	0.060	0.085	0.121	0.095	0.118	0.145	0.173	0.202	0.234	0.230	0.253	0.278

Drainage Area (km^2)	Zone 1 [$L/s/km^2$]			Zone 2 [$L/s/km^2$]			Zone 3 [$L/s/km^2$]			Zone 4 [$L/s/km^2$]		
	lower	mean	upper	lower	mean	upper	lower	mean	upper	lower	mean	upper
1000	49.5	70.2	100	76.9	95.1	117	176	206	239	256	281	309
500	57.7	81.8	117	90.9	112	138	201	235	272	292	321	352
100	82.3	117	167	134	165	203	272	319	369	401	441	485
50.0	96.0	136	194	158	195	240	310	364	420	460	506	556
10.0	137	194	276	232	287	353	420	492	569	633	696	764
5.00	171	242	345	287	355	436	519	609	704	769	846	929
1.00	286	406	579	471	582	715	851	998	1,150	1,210	1,330	1,460
0.500	358	507	723	582	720	885	1,050	1,240	1,430	1,470	1,610	1,770
0.250	447	634	904	720	890	1,090	1,300	1,530	1,770	1,780	1,960	2,150
0.100	599	850	1,210	954	1,180	1,450	1,730	2,020	2,340	2,300	2,530	2,780

Figure A.3. Recommended design curves for the 50-year return period peak discharge – Zone 1: Southern Okanagan Basin.

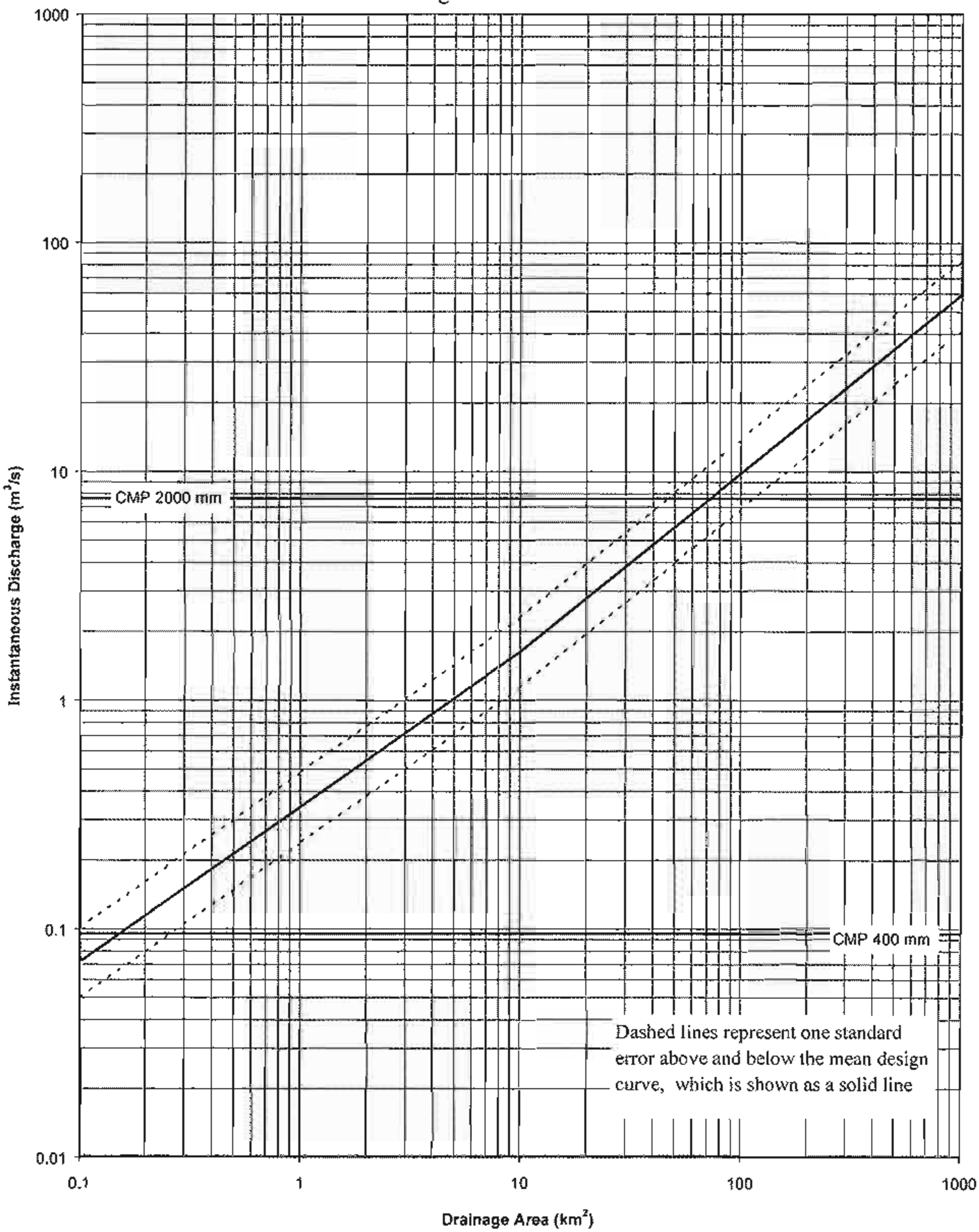


Figure A.4. Recommended design curves for the 50-year return period peak discharge – Zone 2: Northern Okanagan Basin.

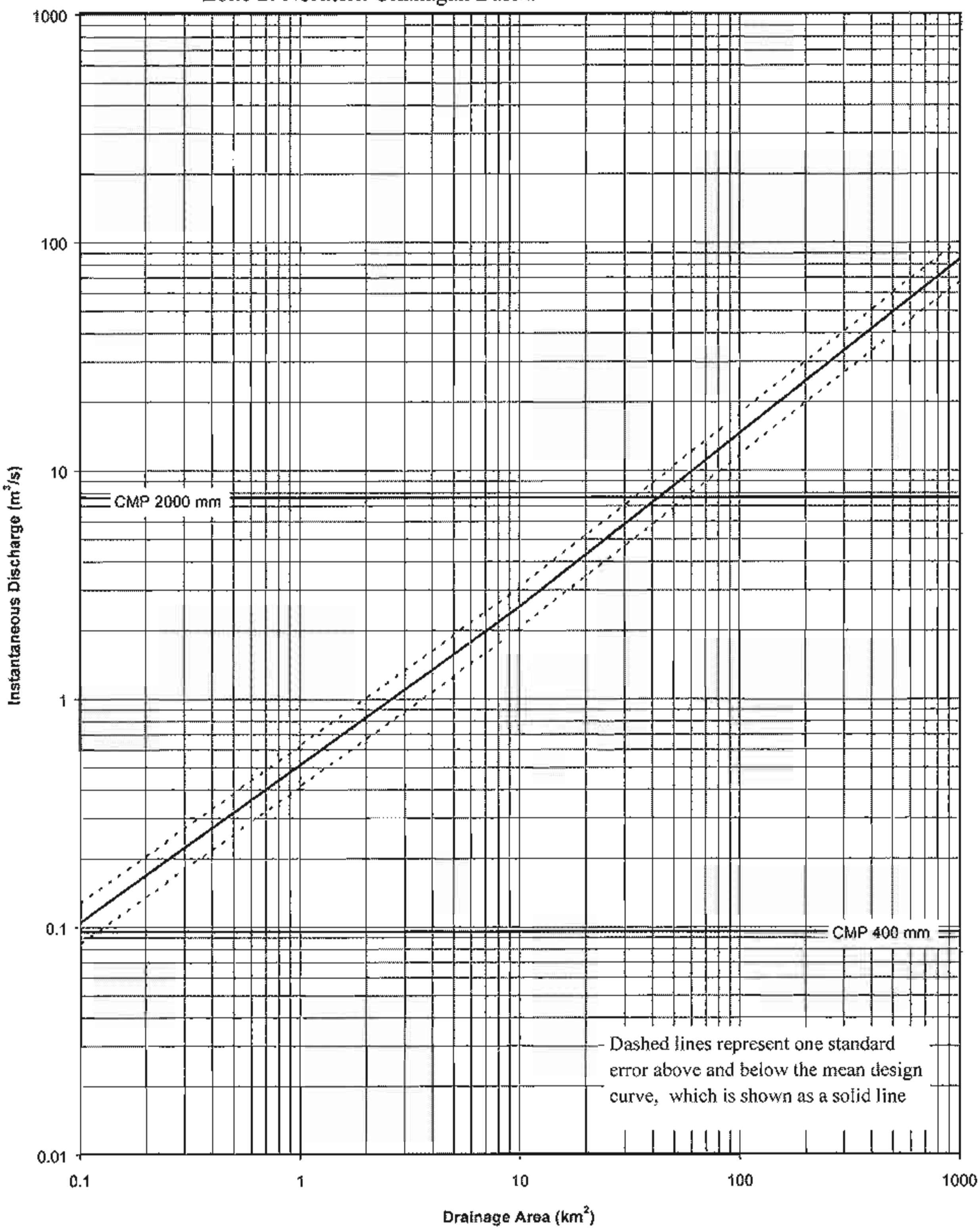


Figure A.5. Recommended design curves for the 50-year return period peak discharge – Zone 3: Eastern Okanagan Highland and Okanagan Range.

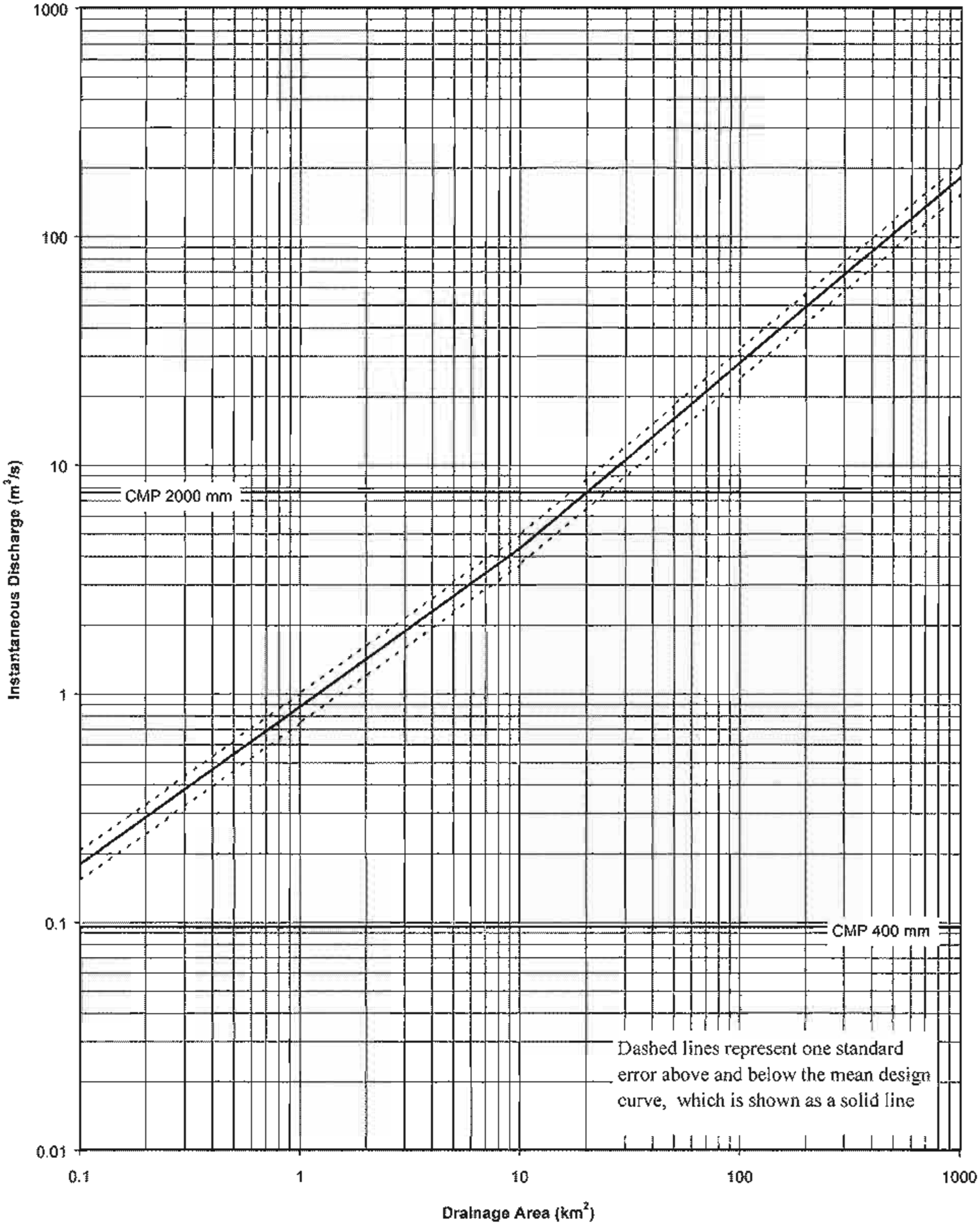


Figure A.6. Recommended design curves for the 50-year return period peak discharge – Zone 4: Upper Kettle River.

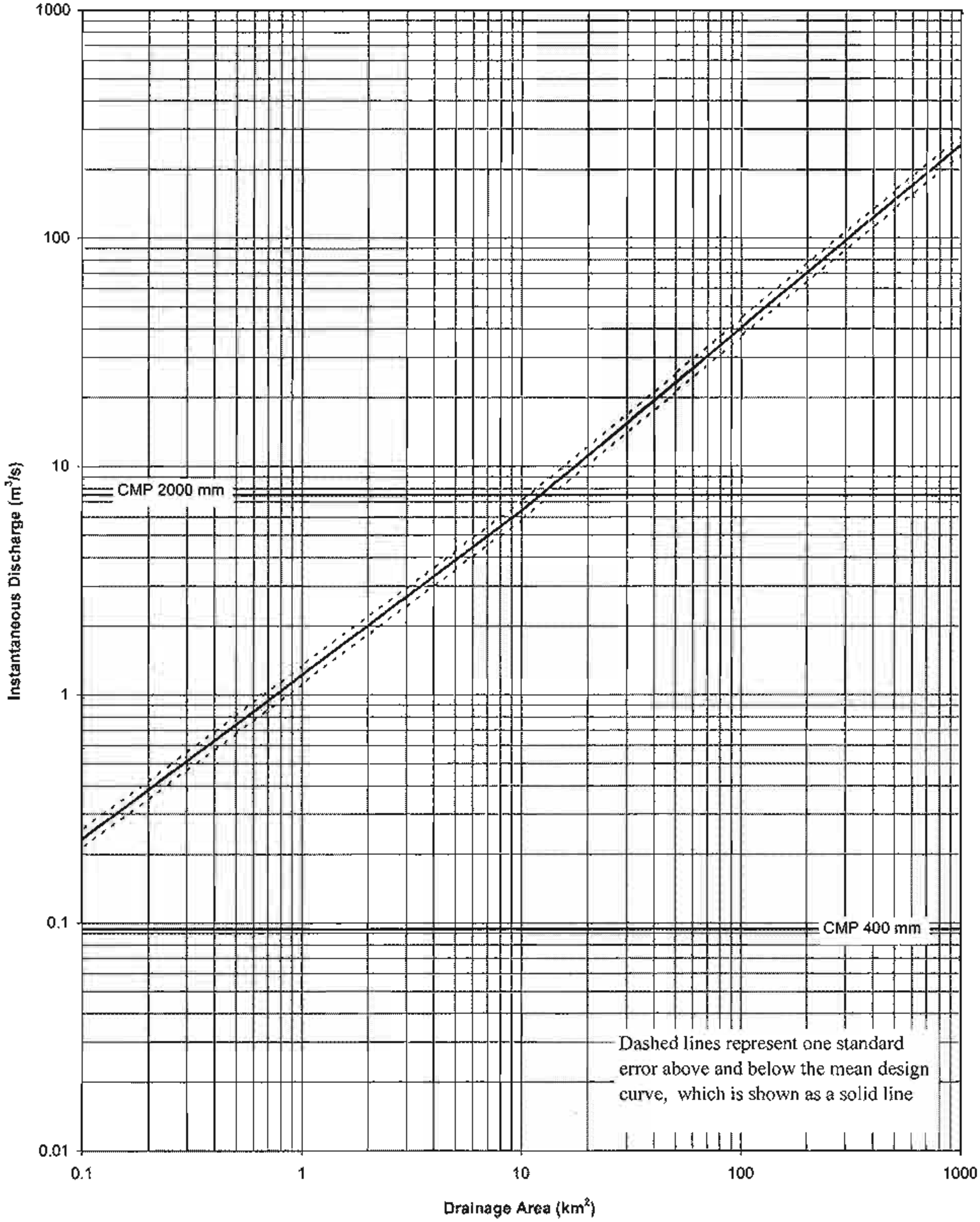


Figure A.7. Recommended design curves for the 100-year return period peak discharge – Zone 1: Southern Okanagan Basin.

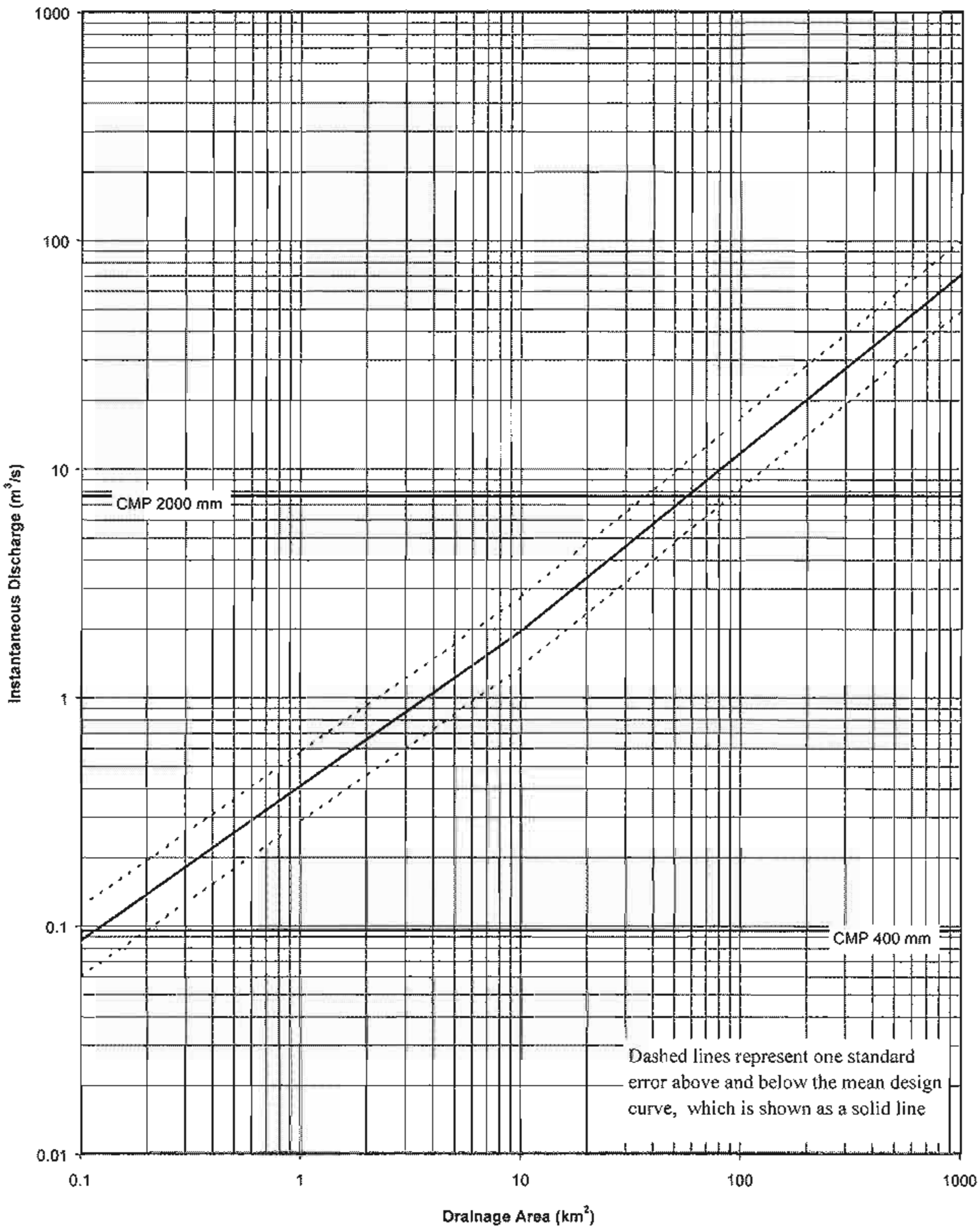


Figure A.8. Recommended design curves for the 100-year return period peak discharge – Zone 2: Northern Okanagan Basin.

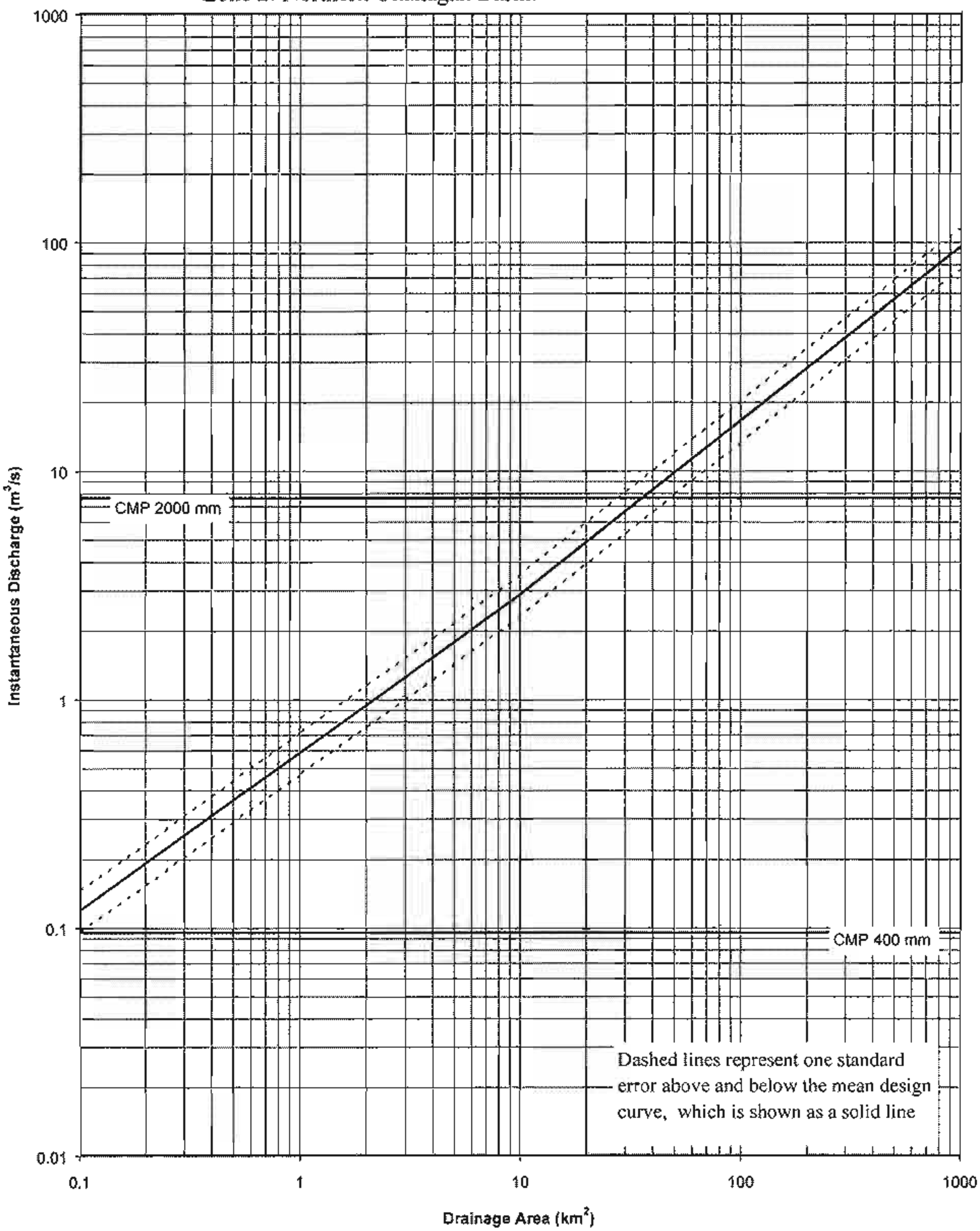


Figure A.9. Recommended design curves for the 100-year return period peak discharge – Zone 3: Eastern Okanagan Highland and Okanagan Plateau.

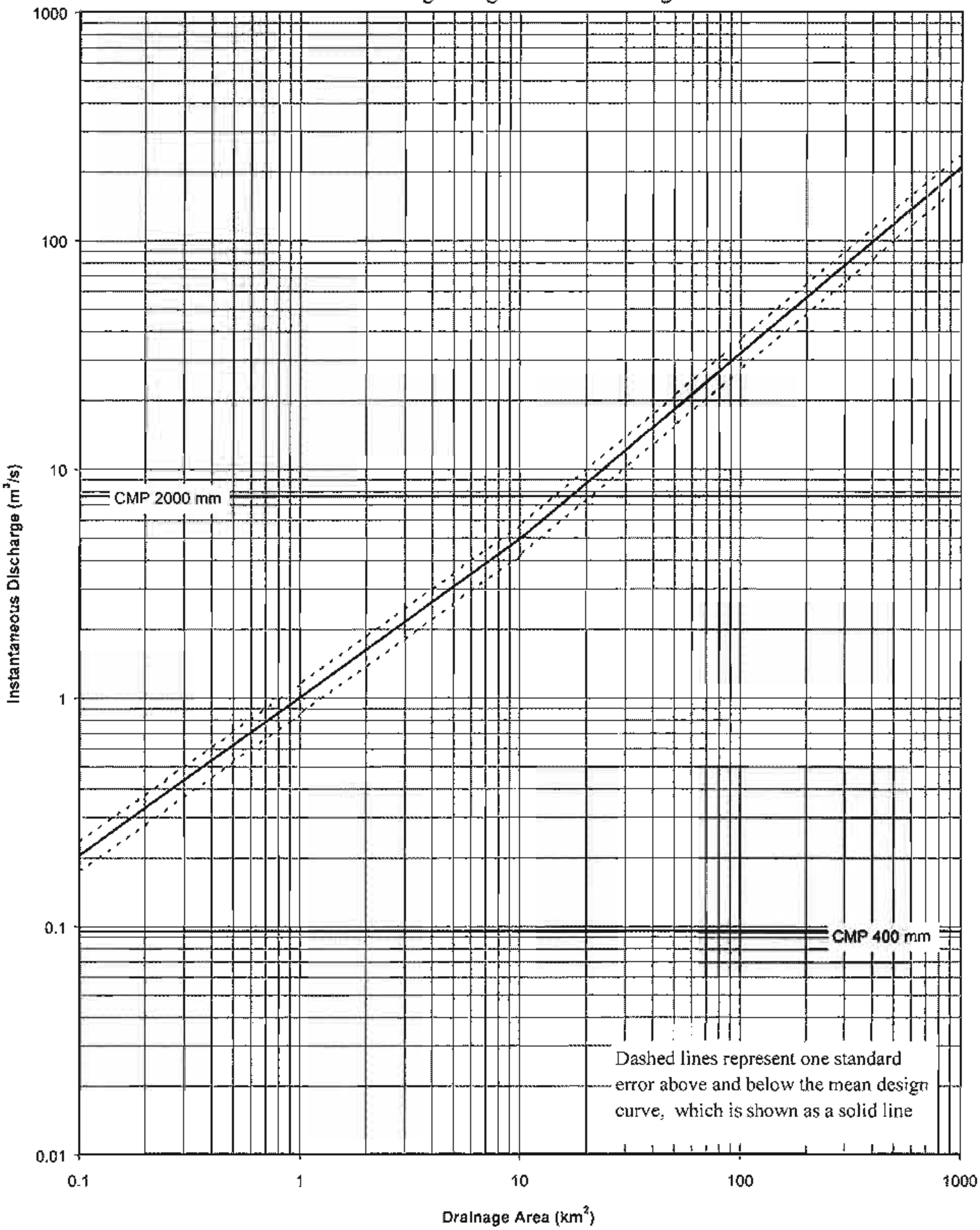
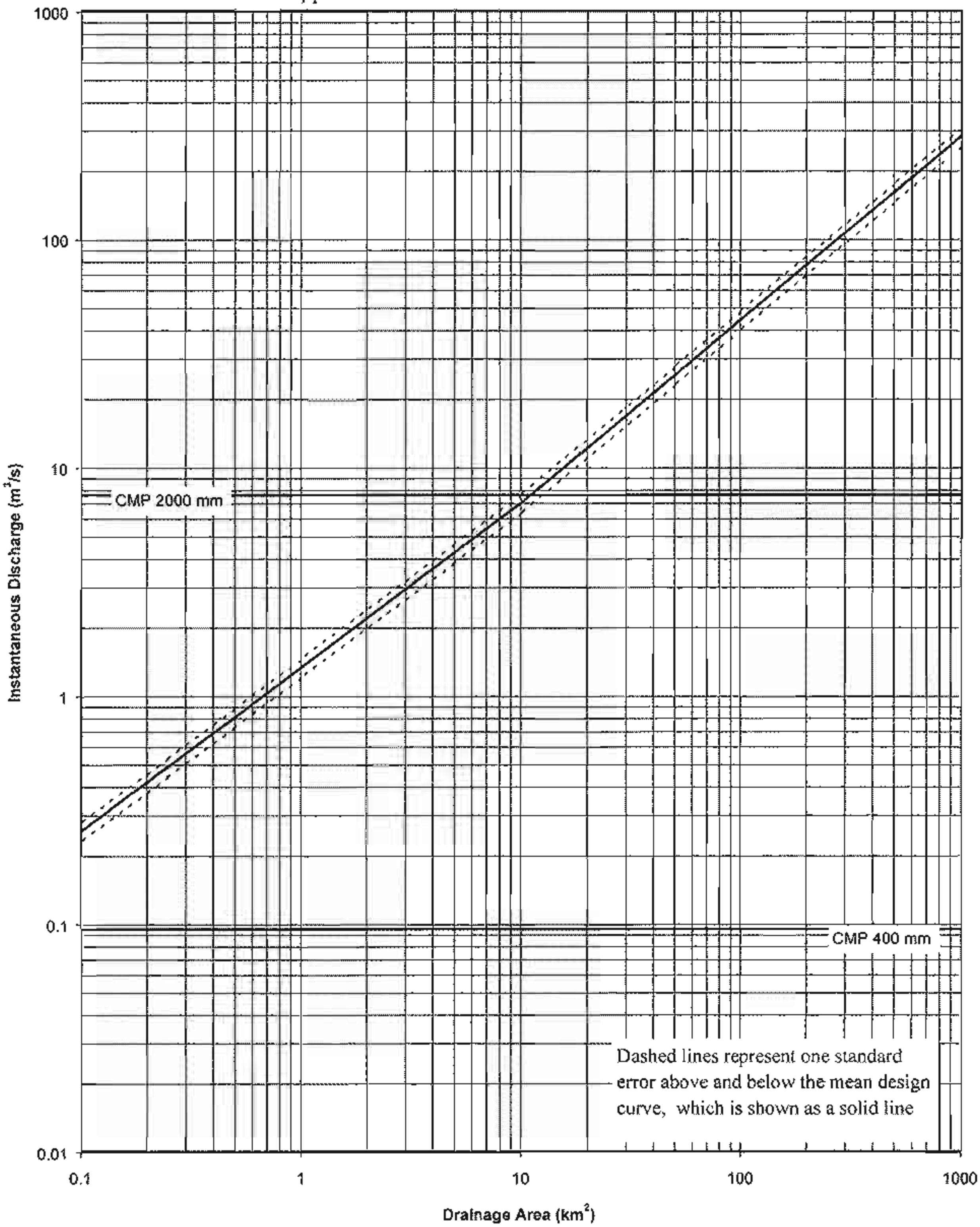


Figure A.10. Recommended design curves for the 100-year return period peak discharge – Zone 4: Upper Kettle River.



A.5 GUIDANCE FOR SELECTION OF PEAK FLOW ZONE

The boundaries of the homogeneous peak flow zones shown on Map 1 have been drawn to represent as closely as possible the hydrologic variations indicated by the data and, where data are sparse, have been inferred based on experience and professional judgement. Despite the fact that the zones are statistically different from each other, hydrologic conditions vary continuously, grading from dryer zones to wetter ones along a continuum. Conditions do not change instantaneously along precisely identifiable boundaries, as suggested by a line drawn on a map. This fundamental spatial variability of hydrologic properties must be considered when choosing which zone to use for design discharge estimation, especially at or near zone boundaries.

The key consideration is the location of the drainage basin upstream of the proposed (or existing) installation, not the location of the installation itself. For drainage basins completely within one peak flow zone and well away from a zone boundary, one can be confident that that zone represents the drainage basin, and no other zones need be considered. For basins within one peak flow zone but close to a zone boundary, it is still likely that that zone represents the basin. However, it may also be appropriate to calculate a design discharge based on the adjacent peak flow zone, and to consider the suitability of both design discharge estimates based on knowledge of the site conditions.

Drainage basins that cross zone boundaries must be addressed carefully. Since snowmelt dominates peak flow hydrographs in the Penticton Forest District, a zone covering the higher elevations is more appropriate that a zone covering the lower elevations of a basin. As a general rule basins that have more than about 40% of the area within an upstream zone should be represented by the upstream zone. However, it is appropriate to calculate the design discharge based on both zones and choose the most appropriate one, based on the site conditions. Basins that have less than 40% of their area in the upstream zone will most likely be best represented by the downstream zone, but this should be confirmed based on the site conditions. In some cases, site conditions may not help identify which zone is most representative of a drainage basin close to or crossing a peak flow zone boundary. The recommended course of action in these circumstances is to choose the larger design discharge. This is a more conservative approach, and will prevent unintentional under-sizing of an installation.

A.6 GUIDANCE FOR MODEL USE IN AREAS AFFECTED BY HYDROLOGIC STORAGE

Storage in lakes and wetlands tends to attenuate the peak of the flood hydrograph, generating lower but longer floods. Because the data set used to develop the peak flow models presented herein excludes stations significantly influenced by lake and wetland storage, the design peak flows produced by the model will overestimate the design discharge required for installations downstream of significant lake and wetland storage.

The magnitude of this effect on Q_{100} was estimated by analysing data from 13 gauged rivers (in interior B.C.) that have lakes affecting one third or more of their watershed but that are not regulated by dams. The records for these 13 stations were used to calculate the index flood, and – where sufficiently long data records were available – the 100-year flood[10]. For those stations at which only the index flood was calculated, the Q_{100}/Q_2 ratio used by *CULVERT* (Ver. 6.1) for that zone was applied to derive an estimate the 100-year design peak flow.

Table A.12 presents the ratio of the actual 100-year design peak flow (Q_{actual}) to that predicted by the model ($Q_{predicted}$) for each of the 13 stations. The mean ratio of $Q_{actual}/Q_{predicted}$ is 69%, and the ratio ranges from 90% for Barriere River below Sprague Creek to 43% for Wolfe Creek at the outlet of Issitz Lake. The standard deviation is about 15% (i.e. about two thirds of the data fall between about 55% and 85%). It is suggested that users apply a correction of 85% to the design peak flows on streams affected by lake storage, equal to the mean value plus one standard deviation, which is considered to be an appropriate

[10] Where sufficiently long records were available, the Q_{100} was estimated using the Pearson Type III frequency distribution.

level of design conservatism. For example, a crossing immediately downstream of a lake for which the model predicts a 100-year design discharge of 10.0 m^3/s should be designed to a 100-year discharge of about 8.5 m^3/s (i.e. 85% of $Q_{predicted}$). This suggestion does not apply to crossings downstream of reservoirs.

Table A.12. Ratio of actual vs. predicted 100-year design peak flows for streams affected by lakes.

Station Name	Station No.	Ratio of Q_{actual} to $Q_{predicted}$
Murtle River Above Dawson Falls	08LA004	0.86
Celista Creek Near Albas	08LE025	0.76
Scuitto Creek Near Barnhart Vale	08LE036	0.79
Spa Creek Above Cowpersmith Diversion	08LE042	0.77
Clark Creek Near Winfield	08NM146	0.72
Horn Creek Near Olala	08NM147	0.57
Camp Creek Near Thirsk	08NM134	0.52
Wolfe Creek at Outlet of Issitz Lake	08NL041	0.43
Bowron River near Wells	08KD001	0.53
Quesnel River at Likely	08KH001	0.84
Barriere River at the Mouth	08LB020	0.60
Barriere River Below Sprague Creek	08LB069	0.90
Vernon Creek below Arda Dam	08NM175	0.55
Mean		**0.69**
Standard Deviation		**0.15**

The "actual" discharges were estimated from the data, and the "predicted" discharges were produced by *CULVERT* (Ver. 6.1).

A.7 LIMITATIONS OF THE PEAK FLOW MODEL

1. It is important to note that the data set presented in Table A.2 is a subset of the data available within the broader B.C. interior region, and has been selected specifically for its relevance to discharge estimation within the study area. Thus the results of the analysis cannot be used to estimate design peak flows for streams beyond the boundaries of the Penticton Forest District (Map 1).
2. The model is designed to estimate 50-year and 100-year return period peak instantaneous discharges in locations that are not affected by significant upstream flow regulation and that are not situated downstream of significant lake, wetland or reservoir storage. For

crossings downstream of natural lakes or wetlands, we suggest that a design peak flow of 85% of the value provided by the model is appropriate.

3. The peak flow model has been developed using data from drainage basins that are less than 5,000 km^2 in size. The peak flow model and computer program should not be used to calculate design discharges for drainage areas larger than 5,000 km^2.

4. The design peak flow methodology is intended to predict the 50-year and 100-year return period design peak flows for ungauged locations. However, there may be instances where design peak flow estimation based on specific gauged records is more appropriate.

5. To account for uncertainty, the model uses the upper 68% confidence limit on the mean design peak flow as the recommended design discharge.

A.8 COMPARISON WITH PREVIOUS ESTIMATES

Previous peak flow studies in the southern interior have been made by Ministry of Environment, Lands and Parks (MELP) using similar regionalization procedures (MELP, 1988). The peak instantaneous discharge corresponding to a 100-year return period flood has been estimated for Echo Lake, McDougal Creek, Shingle Creek, and Blue Springs Creek. The instantaneous discharge corresponding to a 50-year return period flood has been estimated for Corral Creek, Echo Lake, and Texas Creek. These design discharge estimates are provided in Table A.13 along with the design estimates from the models presented in this report.

For all streams except Blue Springs Creek, the estimates presented by MELP are within the 1 standard error bounds for the models presented herein. For these 6 pairs of estimates, the design discharges produced by the present models are 11% higher, on average, than the design discharges estimated by MELP. Careful analysis of MELP's estimate for Blue Springs Creek indicates that a calculation error occurred and that the estimate is too high.

Table A.13 Design discharge comparisons with previous estimates.

Stream	Drainage area (km^2)	Return Period (yr.)	MELP's Design Q (m^3/s)	Q estimated using the present regional models			
				Zone	Design Q (m^3/s)	Lower bound[1] (m^3/s)	Upper bound[2] (m^3/s)
Corral Creek	16.0	50	5.80	3	6.30	5.43	7.33
Echo Lake	3.56	50	1.00	2	1.23	0.995	1.51
Echo Lake	3.56	100	1.10	2	1.40	1.13	1.72
McDougal Creek	38.6	100	8.28	2	8.02	6.49	9.85
Shingle Creek	308	100	32.1	2	38.9	31.4	47.8
Texas Creek	45	50	81.5 / 20.0[3]	4	21.5	19.7	23.4
Blue Springs Creek	32.5	100	16.7	2	7.03	5.69	8.65

1: the lower bound is equal to 1 standard error below the design Q (i.e 84% of the data lies above this value).
2: the upper bound is equal to 1 standard error above the design Q (i.e 84% of the data lies below this value).
3: MELP has presented two estimates for this creek, the smaller of which is more reasonable than the larger.

A.9 REFERENCES

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Appendix B

CULVERT SIZING CHECKLIST

CULVERT SIZING CHECKLIST

(by: Gary McClelland, Kamloops Forest Region)

(1) Is fish passage required? If so, is the stream slope under 6%? If so increase diameter to stream bankfull. Use *Stream Crossing Guidebook for Fish Streams**. If the stream slope is greater than 6% obtain professional advice or look at alternatives.

(2) Is the stream gradient less than 0.5%? If so, check to ensure there is not outlet control. If outlet controlled, obtain professional advice.

(3) When the culvert is running full at the inlet, what is the area of the resulting impoundment pond? If the area of the impoundment pond is greater than 100 m^2, increase pipe diameter or use different pipe configuration to lower impoundment pond elevation such that it is less than 100 m^2.

(4) Will the culvert control the water level of an existing pond and will water level fluctuations will exceed the natural fluctuations? If yes, increase culvert diameter until the resulting fluctuations are equal to the natural fluctuations. Consider professional advice.

(5) Will the consequences of failure (fill washout) affect life or property of downslope stakeholders? If so (or if there are other unacceptable consequences) obtain professional advice.

(6) Is there a safe overflow? If yes, items (7) & (8) can be ignored.

(7) Is the stream subject to floating debris? If small debris is an issue, increase diameter to bankfull stream width. If large debris is an issue obtain professional advice.

(8) Is the stream subject to debris torrents? If yes, obtain professional advice.

(9) Is the stream slope steeper than 15%? If yes, obtain professional advice.

(10) Is the value of the crossing high (i.e. >$5000) or the height of fill greater than 4m at roadway C/L? If so consider increasing culvert diameter.

(11) Is there a drainage plan for the area? If yes, check to ensure upslope activities (large cut blocks or drainage pattern alteration) will not affect stream hydrology. If upslope activities will affect hydrology, consider larger culverts or small bridge.

(12) Are there upslope alterations to the watercourse (e.g. old roads/trails diverting water into/out of the drainage)? Is there potential for upstream crossings to fail creating surges in the flow? If so consider increasing culvert diameter.

(13) Is the road or watercourse on, downslope of, or upslope of a Terrain Stability Class 4 or 5 polygon? If yes consider larger culverts or small bridge and consider obtaining professional advice.

Source: digital file provided by G. McClelland (13/04/2000)

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