Bridge Conceptual Design Guidelines

Version 1.0

Bridge Engineering Section
Technical Standards Branch
Alberta Transportation

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Foreword

These guidelines cover all aspects of bridge conceptual design, including bridge location, sizing, geometrics, and protection works. These guidelines apply to all Alberta Transportation projects involving bridge size structures, including all tasks identified as Bridge Planning as per the current version of the Engineering Consultant Guidelines, Vol. 1.

Although this document is intended to be thorough, certain cases may arise where specific guidance is not provided or not applicable. Consultants working for Alberta Transportation must exercise good engineering judgment in the application of these guidelines. Although the bridge conceptual design process is based on optimization, the design exception process may be triggered on some projects (section 4.2). Any project specific questions relating to these guidelines should be directed to the project administrator.

Any feedback or technical clarification requests relating to this document should be directed to the Director, Bridge Engineering Section, Technical Standards Branch, Alberta Transportation.

Approved:

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1 <u>Introduction</u>

1.1 Process Description

The purpose of the bridge (including bridge size culverts) conceptual design phase of a bridge design (sometimes referred to as Bridge Planning, section 10.10 of the Engineering Consultant Guidelines, Vol. 1) is to find the most suitable solution for a roadway to cross a stream or road (or other facility). In order to develop an optimal solution, all relevant issues must be considered and a range of feasible alternatives may need to be examined. The optimal solution will provide the most value of all alternatives considered, balancing cost, functionality, and other constraints.

This document provides certain criteria that must be met with all feasible options, as well as a description of the process and technical considerations to be employed to arrive at the optimal solution. Development of feasible alternatives will require balancing competing constraints from several distinct technical areas. Selection of the optimal solution will require some judgement, and a solid defense of the recommended proposal should be prepared. In some cases, budgetary restrictions may require selection of a feasible, but non-optimal, option.

An example of a feasible, but sub-optimal solution may be replacement of a bridge on the same alignment and gradeline, using staged construction. The existing alignment and profiles may have limitations relative to the road designation that cannot be addressed without a new alignment. However, the existing functionality can still provide safe and economical use with the current operating speed. This option could be significantly less expensive due to the reduction in road construction work.

The major constraints in bridge conceptual design are hydrotechnical (stream crossings) and bridge/highway geometrics. Additional significant constraints include structural options, geotechnical parameters, environmental and regulatory issues. Due to the often competing constraints, it is important to question all stated constraints and identify potential mitigation techniques, where appropriate.

Experience has shown that this process works best when the lead bridge conceptual designer is well versed in the hydrotechnical and bridge/highway geometrics technical areas, and understands how the other disciplines impact bridges.

On grade separation projects and bridges that are part of the design of a new highway, highway design details are likely to be developed by others. As such, the optimal bridge conceptual design will require significant communication

between the bridge conceptual design team and the roadway planning/design team. For stream crossings, impacts on highway alignments and profiles due to hydrotechnical parameters and river engineering issues (section 2) are generally understood. However, integration of bridge specific geometrics issues (section 3) into the overall highway design must also be done to ensure the optimal overall solution.

For stream crossings, the bridge conceptual design team is typically also responsible for the detailed design of any river protection works involved with the bridge project.

1.2   Role in Alberta Transportation Projects

The end product of the bridge conceptual design phase will be sufficient definition of the geometrics of the bridge to facilitate structural design of that option, starting with the "choose design" process. Detailed design requirements for bridge projects are detailed in the 'Bridge Structures Design Criteria' and 'Design Guidelines for Bridge Size Culverts' documents. The conceptual design phase for Alberta Transportation bridges is not subject to the CAN/CSA-S6-06 Canadian Highway Bridge Design Code (CHBDC).

Ideally, the conceptual design and detailed design processes should not overlap, to avoid the risk of expensive do-overs of detailed design tasks. Additionally, many cost savings can be realized during the conceptual design phase in comparison to later design phases.

The bridge conceptual design team may still need to participate in the detailed design and construction phases of a project. This may involve:

- clarifying elements of the conceptual design
- commenting on impacts of proposed changes to the concept
- reviewing pier configurations for river functionality (drift, scour)
- comment on detour structure sizing and location, if applicable
- recommend modifications to river protection works, if applicable

In addition to involvement in the design and construction of a new bridge, bridge conceptual design expertise is also employed in several other areas of bridge and highway management, including:

- *Functional Planning Studies (ECG, Vol. 1, Section 6)*
- *Bridge Location Studies*
- *Bridge Assessments (Bridge BPG No. 5)*
- *Bridge Rehabilitation (river engineering)*

The main difference between a bridge design project and involvement in these tasks is the level of detail in data collection, analysis, and reporting undertaken. In most cases, these other projects will involve limited site survey and geotechnical data. Sufficient work is required to identify a solution that will resemble the optimal solution, based on available data and assumptions provided. Additional refinement of the bridge conceptual design will occur during the subsequent bridge design process. All relevant site constraints identified should be clearly documented as part of these planning assignments to assist any future design task.

1.3   Constraints

Hydrotechnical (stream crossings):
- Design parameters for stream at crossing site – flow depth (Y), mean flow velocity (V), design flow (Q)
- Structure impact on hydraulics – constriction (increase in V, Y), drift/ice handling, adjacent structure impact
- River issues – bank stability, flow alignment, extent of protection works, river processes (degradation/aggradation)

Bridge/Highway Geometric:
- Highway alignment – curve radius, super-elevation, spiral run-out, skew, land impact, travel length, safety, adjacent access location, land severance
- Gradeline – rate of gradient change (K), length of curve (L), bridge height, freeboard, grade on bridge, cut/fill balance, pavement transition, vertical clearance
- Bridge geometrics - width, cross slope, sight distance, shy distance, clear zone requirements

Structural:
- Span arrangements, maximum span length, pier locations
- Structure depths
- Culvert vs. bridge, major bridge vs. standard bridge

Geotechnical:
- Slides, headslope ratios, remediation works
- Pile depths, settlements
- Retaining walls

Environmental:
- Fish passage need, habitat compensation requirements
- Wildlife passage need
- Highway drainage, stormwater management

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- Navigated use of waterway

Construction:
- Traffic accommodation (detour/staging)
- Berm flow constriction

Stakeholder:
- Upstream flooding impacts, bank erosion in vicinity
- Impact on route length, safety, access relocation
- R/w concerns and purchase
- Utilities

Future Plans:
- Life cycle bridge rehabilitation, culvert lining, traffic accommodation
- Highway widening, twinning, minimize throw-away costs

1.4   Process Overview

The main steps in the bridge conceptual design process are:

Data Collection:
- Assemble all pertinent data for crossing site, stream, existing structure etc.
- Refer to section 2.1 and 3.1 for list of pertinent data

Site Inspection:
- Visit site, record measurements and observations
- Refer to section 3.1 for items to consider
- Include upstream and downstream crossings (streams)

Arrange for Technical Input:
- Order site survey, communicate extents – see section 3.1
- Arrange for geotechnical investigation (test holes) and analysis
- Order environmental/regulatory inputs – confirm requirements, coverage, refer to TRANS' Environmental Management System

Hydrotechnical Assessment:
- Determine hydrotechnical design parameters (Y, Q, V) – section 2.2.2
- Identify hydrotechnical issues, including river protection works (section 2.4, 2.5)

Review Technical Inputs:
- Check survey for completeness and accuracy, update as necessary
- Note geotechnical recommendations for slide mitigation, headslope ratios and pile penetration (obtain mitigation options, if necessary)

- Note fisheries constraints (restricted activity periods, habitat compensation, fish passage need…)
- Other constraints (utility relocation, land purchase…)

Develop Feasible Options:
- Co-ordinate with concurrent highway geometric design, if it is being done by others
- Combine alignment, profile, bridge opening, and protection options (section 3.2)
- Undertake hydraulic analysis (stream crossings, section 2.3)
- Sketch key components of all options to confirm feasibility
- Quantify costs and identify pros and cons for all options

Evaluate Options:
- List and evaluate all feasible options, with costs and list of pros and cons
- Obtain structural review and comment as necessary
- Identify optimal solution

Prepare Initial Report:
- Summarize work done, parameters established, options considered
- Provide recommended optimal solution, with supporting sketches (section 4.3)
- Obtain independent review of completeness, accuracy, and consistency by experienced bridge conceptual designer (not an independent design)

Obtain Department Guidance
- Meet/communicate with project sponsor, as appropriate
- Obtain approval/guidance on option to finalize, required modifications

Finalize Documentation:
- Update initial report with changes, as appropriate
- Finalize sketches/drawings as necessary to fully document plan

Follow Up:
- Participate in detailed design and construction phases, as necessary

2 <u>Hydrotechnical Design</u>

2.1 *Hydrotechnical Data*

2.1.1 Channel Parameters

Typical channel parameters are key inputs to the calculation process for hydrotechnical design parameters. The typical channel is a trapezoidal

representation of the entire stream reach that the crossing is located over. The key parameters required to describe the typical channel are:

Bedwidth (B) – width of the base of the trapezoid
Topwidth (T) – width of the top of the trapezoid (at bankheight)
Bankheight (h) – height of the trapezoid
Slope (S) – hydraulic slope of channel (m/m)

Sources of information for 'B' and 'T' include georeferenced airphotos, digital elevation models (DEM), and site measurements.   As these parameters are for the typical channel, many cross sections should be sampled. It should be noted that in many cases channels in proximity to an existing crossing may have been modified during construction, and may not be representative of the natural channel.

Surveyed cross sections are usually too limited in number to enable estimation of the typical values. Assessment of airphotos and DEM data using GIS tools is more effective and efficient in determining the median channel width values. 'B' values can generally be estimated from surface water width on airphotos at low water levels with adjustments based on observations and survey data as appropriate. Some localized hydrographic survey is required in the vicinity of the crossing to facilitate design (section 3.1).

The bankheight is the height above streambed at which the channel transitions to the floodplain, and there should be a sudden change (decrease) in the slope of the terrain perpendicular to the channel. Sources of information for 'h' include surveyed cross sections, high resolution DEMs (e.g. LiDAR), photos with scalable objects, and site measurements. Calculations can be very sensitive to this parameter, and a sensitivity analysis may be required during the channel capacity estimation process.

Many other bankheight definitions have been published. A common one is based on the line of permanent vegetation. This will typically be below the geometric definition of bankheight used in bridge hydraulic analysis, and should not be used for this purpose.

The channel slope 'S' is typically a small number (<<1) so a significant length of channel profile (many kms in some cases) is required in order to estimate this parameter effectively. Therefore, channel slopes are estimated from DEM data. Channel profiles have been extracted from DEM data for most streams in the province and published in the Hydrotechnical Information System (HIS). HIS includes a tool that facilitates estimation of 'S', and this should be sufficient for most sites. Calculations are generally not very sensitive to small changes in 'S'. In some cases, the crossing may be located close to a sudden break in the channel profile, and additional investigation may be required to confirm which slope would apply to the crossing site. This may require the use of high

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resolution DEMs, extended channel profile survey, or association with the nature of the channel at the crossing with either the upstream or downstream reaches.

In many cases, the variations in bed levels measured in a streambed survey exceed the total drop observed over the length of the survey. Natural channels are a difficult environment for surveying, with difficulties in locating the thalweg, limited line of sight, and hazards from flowing water, soft silt deposits, and slippery banks. Therefore, extending the length of surveys to assess the channel slope is generally not a practical solution.

2.1.2 Historic Highwater (HW) Observations

Historic observations of highwater are another key component in the calculation process for hydrotechnical design parameters. These observations may govern the design parameters in some cases, and confirm the typical channel parameters in many others. In many cases, available highwater data will include measurements, photos, and observations. Some of the key sources of historic highwater observations include:

Hydrotechnical Information System (HIS):
- Tool developed by Alberta Transportation that includes close to 4000 highwater records collected by the department at many stream crossing sites across the province. Some records from Alberta Environment are also included.
- All crossings of a stream are grouped together to facilitate use of observations at nearby crossings.

Water Survey of Canada (WSC):
- Section of the Federal Government that measures, estimates, and publishes flow data at many sites across Canada.
- For the last 50 years or so, there have been close to 400 gauges in operation. Although flows are reported at these gauges, stage (flow depth above an established datum) is actually measured continuously, with occasional flow measurements used to convert these stages to flows (using a rating curve).
- Alberta Transportation has collected actual flow measurement and peak stage data from the files of WSC and published it within the PeakFlow tool This data can be used to assess the accuracy of published flow values.
- Note that even actual flow gauging measurements under flood conditions can be inaccurate due to the unsafe and difficult conditions present.

Bridge Correspondence Files:
- Highwater records in HIS do not necessarily cover all records for some sites. Bridge correspondence files may contain additional observations of value. Note that the files at the Twin Atria and at the Regional offices do not necessarily contain the same information.

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- In addition to measurements and observations, work orders for repair to flood induced damage and other channel related work may also be found.

Bridge Design Drawings:
- In many cases, design drawings will note estimated highwater levels corresponding to specific flood events, in addition to the design highwater levels used. These values should be confirmed by checking the original data used by the designer (review available design notes for estimated depths and datum corrections).

Bridge Inspection Reports:
- Some highwater observations may be found on Bridge Inspection and Maintenance (BIM) system inspection forms. Sometimes, these observations carry over from one inspection to the next, so it can be difficult to associate them with a specific event, unless one is noted.

Site Inspection Observations:
- In some cases, highwater marks from a recent event may still be present during a site inspection. Common marks include deposits of silt and drift, grass and weeds on fences, and abrasion marks on piers.

Local Sources:
- Information on past highwater levels and impacts may be available from nearby landowners, municipal officials, newspaper records, and maintenance contract inspectors.

Airphotos:
- Airphotos flown close to the peak of the flood event will show the horizontal extent of flooding, enabling estimation of associated water levels. Peaks occurring at night time and cloud cover can impact the effectiveness of this source.

2.1.3 Other Hydrotechnical Data

Additional data that is of interest in hydrotechnical design includes:

- Drainage Area (DA) – the total surface area potentially contributing flows to a site. This is a parameter used in the Basin Runoff Potential check (section 2.2.5). DA can be determined using DEM data and GIS tools. DA data has been published for most stream crossings in HIS.
- Airphotos – review of historic airphotos can show changes in flow alignment and bank location over time, facilitating assessment of lateral mobility of the stream. Bank erosion can be tracked using georeferenced airphotos and GIS tools. Banktracking analysis summaries are published for many sites with extensive river protection works in HIS.

- Scour surveys – scour surveys can show vertical changes in streambed topography over time, including local and general scour and bedform movement. Scour surveys are published for many scour vulnerable crossing sites in HIS.
- AESRD Flood Risk Mapping System – this online tool will show if flood risk mapping has been done that affects a given site. Crossings in these areas may be subject to additional constraints.
- AESRD Code of Practice maps – these maps classify streams in terms of their importance in fisheries management, and note restricted activity period dates.
- Local site hydraulic influences – such as other structures (weir, bridge, culvert, dam), sudden channel changes (slope, width), and confluences with other channels.
- Site channel observations – channel features (bedforms, islands…), flow concentration and alignment, active bank erosion, and ice scars on trees.

2.2 Design Parameters

2.2.1 Overview

Design of stream crossings requires estimation of design flow depth (Y), mean channel velocity (V), and resultant flow (Q). Key principles in determining these parameters are that they are:

- Representative of the physical capacity of the channel to deliver flow to the site
- Consistent with the highest historic highwater observations.

Approaches such as flood frequency analysis and rainfall runoff modeling have proven to be incapable of meeting these principles at most sites, as documented in "Context of Extreme Floods in Alberta". This document shows that the highest flood stage values tend towards a certain level at all long-term gauge locations in Alberta. This suggests that there is a design worthy stage for stream crossing infrastructure, rather than a continuum of increasing stages with smaller probabilities of exceedance. This observation is consistent with the flow routing effect that occurs when channels spill their banks, and the fact that these channels have typically been formed by the runoff regime of the basin over many years (centuries).

If a hydrotechnical summary has been published for the site in question (HIS), the published summary should be confirmed and updated as necessary, based on available information. If one is not available, the hydrotechnical design parameter process, as described below, should be applied to available data. Any of the published summaries can be used as a template for following this process.

Hydrotechnical summaries are documents that record the application of the hydrotechnical design parameter process. These summaries have been established and published for about 1500 crossing sites within HIS.

2.2.2 Hydrotechnical Design Parameter Process

There are three main components to the process to determine hydrotechnical design parameters for stream crossings. These are:

- **Channel Capacity (CC)** – basic technique to estimate capacity of the stream to deliver flow to the crossing under flood conditions, governs for most sites

- **Historic Highwater (HW) Observations** – ensures design parameters are representative of the largest observed historic events, governs for some large crossing sites, confirms CC values at many others.

- **Basin Runoff Potential (BRP) Check** – technique to check if the basin can supply enough water to fill the channel, seldom governs but necessary for those sites where it does govern.

The process to apply these components, as documented in the published hydrotechnical summaries, is as follows:

1. Estimate Typical Channel Parameters:
- Estimate B, T, h, S (see section 2.1.1)

2. Apply Channel Capacity (CC) Technique:
- Set $Y = Y_{CC}$, calculate V, Q (see section 2.2.3)

3. Assemble Historic Highwater (HW) Data:
- Identify/confirm HW data from all sources (see section 2.1.2)

4. Check if HW exceeds CC:
- If $Y_{HW} > Y_{CC}$, set $Y = Y_{HW}$ (re-calculate V,Q) (see section 2.2.4)

5. Calculate Basin Runoff Potential (BRP):
- If drainage area $< 100km^2$, look up 'd', calculate Q_{BRP} (see section 2.2.5)

6. Check if BRP governs:
- If $Q_{BRP} < Q$, set $Y = Y_{BRP}$ (re-calculate V)

2.2.3 Channel Capacity:

This technique estimates the capacity of the channel to deliver flows to the crossing site at a defined depth (Y_{CC}) above the bankheight (h). This depth assignment is based on flood level observations across the system. As the

floodplain will have been activated at this level, the resulting parameters are representative of a flood, and significant flow routing will be in effect. The depth assignment is as follows:

h	Y_{cc}
< 1.0	h + 0.5
1.0 – 2.0	1.5 * h
> 2.0	h + 1.0

These values are based on analysis of the largest historic highwater observations from WSC and HIS data. Analysis shows the highest Y values trending with 'h', with most values being within 1m at higher 'h' values and 0.5m at lower 'h' values. Values that exceed the channel capacity limit will be accounted for in the historic highwater analysis (section 2.2.4). Most of the sites that exceed channel capacity levels have been accounted for in published hydrotechnical summaries within HIS.

Hydraulic calculation of V is as follows:

B	V_{cc}	n
>= 10	$14R^{0.6}S^{0.4}$	NA
7 – 9	$R^{0.67}S^{0.5}/n$	0.040
4 – 6	$R^{0.67}S^{0.5}/n$	0.045
<= 3	$R^{0.67}S^{0.5}/n$	0.050

Where:

R = hydraulic radius = A/P
A = typical cross section area of flow at Y_{CC} (m^2)
P = wetted perimeter of typical cross section at Y_{CC}
n = Manning roughness coefficient, with adjustment as follows:

S	'n' adjust
< 0.0005 (B > 8)	n = n – 0.005
0.005 – 0.015	n = n + 0.005
> 0.015	n = n + 0.01

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The equation used for B>=10m is based on analysis of WSC measurement data for all large, alluvial streams in the province. This equation provides a better fit to available data than the Manning equation, with no need for a roughness parameter. This analysis is documented in "Evaluation of Open Channel Flow Equations".

The Manning equation is still employed for smaller channels due to the lack of data for channels in this range in the WSC data-set, and the increased influence of bank roughness in this range of channel widths. The 'n' values shown have been found to be representative of hydraulic conditions at most sites studied, as well as being in range with values published in hydraulic reference material. Application of these values on Alberta Transportation projects will result in increased consistency. The slope adjustment factor accounts for the different exponent used in the Manning Equation.

The Alberta Transportation 'Channel Capacity Calculator' has all of these calculations built into it.

The resulting flow value is calculated as Q = VA. In most cases, the flow at Y_{CC} will be significantly higher than the flow at bankheight, due to the additional flow area and higher mean velocity. The flow estimate generated by this technique is for the main channel only, and will not include flows adjacent to the channel on the floodplain (section 2.3.1).

2.2.4 Historic Highwater Observations

When considering historic highwater observations from various sources, the following points should be considered:

- As a hydrotechnical design parameter, the flow depth should be for the channel without the influence of any structures that will no longer be in place with the new design. Therefore, any increase in water levels caused by an existing structure, such as a constricted opening, superstructure in the water, or blockage due to drift, should be accounted for.
- Highwater data for structures located on the same stream either upstream or downstream should be considered to maximize the amount of available data. Judgment should be applied when considering data from sites with substantially different drainage areas or channel parameters.
- Some data may conflict with other data, or seem infeasible compared to physical parameters for the site or the known impact to the structure. The source and evidence for highwater data should be considered when establishing the validity of this data.

An existing bridge may have influenced water levels under flood conditions, due to headloss from a constriction, or blockage of the opening. Measurements taken downstream of a bridge are more likely to be representative of the natural channel response under flood conditions. Measurements taken upstream and

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downstream will enable accurate assessment of the operation of the bridge under the flood conditions. If measurements are only available upstream, potential hydraulic impacts of the bridge should be considered.

Collecting accurate data under flood conditions can be very difficult, often resulting in conflicting information. Timing of observations relative to the flood peak should also be considered, as some may have been before the peak (no higher marks visible), and some may have been after levels have receded (highwater marks should be visible).

2.2.5 Basin Runoff Potential Analysis

In some cases, there may not be enough supply of water from the basin to fill the channel to channel capacity levels. Although this is uncommon, it can occur when the drainage area (DA) is relatively small (< 100km^2) relative to the capacity of the channel.

An extreme case for illustration purposes would be a small drainage basin (e.g. 5km^2) that drains into a ravine that cuts through the valley wall of a larger river. The ravine may be steep and have high banks, with a very high capacity, but the basin will not supply enough water to fill it.

To estimate this upper bound on rate of supply of water from the basin, a Basin Runoff Potential Map (see below) has been developed that has assigned the largest observed unit discharges ('q', cms/km^2) to various hydrologic regions across the province. Once 'q' is known, the basin runoff potential upper bound check can be calculated as $Q_{BRP} = q * DA$. One known exception to the map is the Cypress Hills area, south of Medicine Hat, where higher unit discharges have been recorded due to the higher basin gradient, and q = 0.4cms/km^2 is recommended.

The Runoff Potential Map is a variant of the Runoff Depth Map. This map was created by analyzing all major runoff events in the WSC database, and assigning peak values to areas of similar hydrologic characteristics. Details of this process can be found in 'Development of Runoff Depth Map for Alberta'.

Some potential adjustments to Q_{BRP} include:

- If a large on-stream storage facility is located upstream of the site, basin runoff potential estimates for the downstream contributing drainage area should be added to estimated peak outflows from the lake to account for the storage routing effect.
- If there are significant amounts of poorly drained areas near the periphery of the basin, these areas should be excluded from the value for DA used in the calculation.

- If there are significant amounts of storage and poorly drained areas throughout the basin, consider reducing the value for 'q' based on judgment.
- If the basin covers multiple hydrologic regions, an approximate weighted average for 'q' can be applied.

q (cms/km²)

Value
2.00
1.40
1.25
1.10
1.00
0.85
0.70
0.55
0.25

Basin Runoff Potential Map

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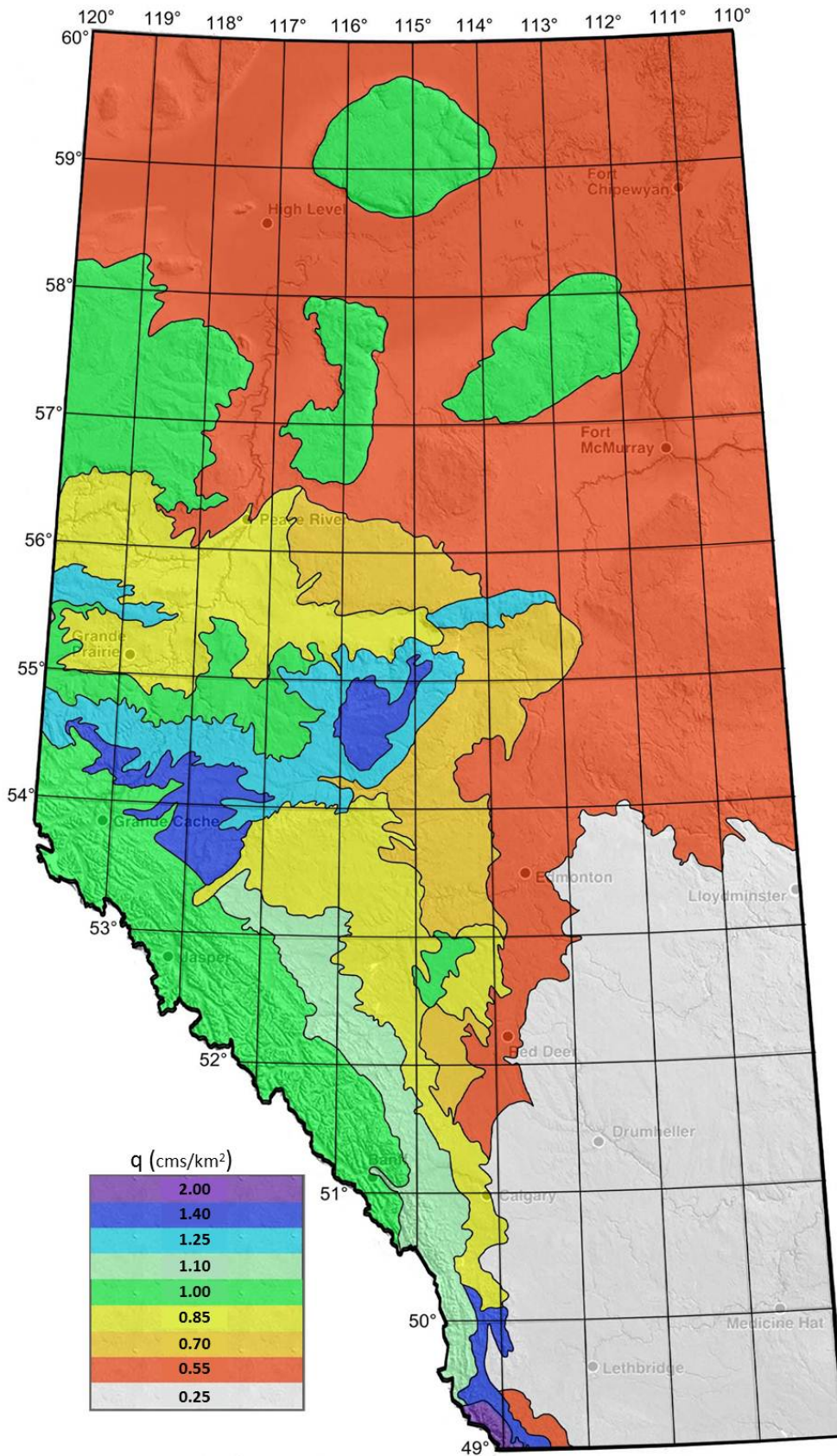
2.2.6 NWPA Reference Level

Current Transport Canada (TC) practice for assessing navigation impact of a crossing structure is to use a reference level based on the mean annual flood (also referred to as the Q_2 or 1 in 2 year flood). Clearance provided by a bridge is measured from this reference water level to the underside of the bridge girders. Currently, this applies to all streams declared navigable by TC under the Navigable Waters Protection Act. As of April 1, 2014, crossings on only 5 rivers in Alberta will be subject to TC approval under the new Navigated Waters Act. These are: Bow River, South Saskatchewan River, North Saskatchewan River, Athabasca River, and Peace River. All other stream crossings will be assessed following the department's practice to ensure due diligence in protecting the common law right of navigation.

When TC approval is necessary, the Q_2 reference water level will need to be established. For crossings with nearby WSC gauges with fairly long records (e.g. >20 years), the Q_2 can be calculated as the average of the reported annual maximum mean daily flows. The reference water level can then be established using the calculations described for channel capacity calculations in section 2.2.3. However, for sites without this data, the following method is proposed to estimate the equivalent to the Q_2 water level.

1. Calculate design flow (Q) as per section 2.2

2. Calculate $Q_2 = Q/(4 + 600S)$

3. Calculate Y_2 using the calculation procedure from section 2.2.3

4. Add Y_2 to streambed elevation to get the Q_2 reference water level

This method of estimating Q2 is based on analysis of design flows determined at long-term WSC gauging sites and correlating with calculated Q2 values. This analysis is documented in 'Estimation of Navigation Clearance Box Reference Water Level'.

2.2.7 Fish Passage Design Flow

A fish passage design flow Q_{FPD} is required for assessment of fish passage at culverts on fish bearing streams (see section 2.3.4).   This is calculated as follows:

1. Calculate $Y_{FPD} = 0.8 - 34.3S$; minimum $Y_{FPD} = 0.2$

2. Calculate Q_{FPD} at Y_{FPD} as per section 2.2.3

This method of estimating Q_{FPD} is based on analysis of observed flow values at all WSC gauges with significant records on streams in the range of suitability for

culvert crossings. Y_{FPD} is set at an envelope curve of all observed values correlating to flow values that are exceeded only 5% of days in a year. This approach should ensure that fish passage is evaluated at a relatively high flow, while providing more consistent results than statistical estimates such as the 1 in 10 year 3 day delay flow.

2.3   Hydraulic Calculations

2.3.1 Overview

Ultimately, hydrotechnical influences on bridge design involve sizing the opening and sizing the protection rock. The Y, V, Q values determined in section 2.2.2 will form the boundary conditions for hydraulic impact calculations. For culverts and constrictive bridges, calculations involving gradually varied flow (GVF, such as backwater curves) and rapidly varied flow (RVF, rapid flow adjustments with abrupt energy losses over a short distance) are necessary to determine hydraulic impacts of the structure. These calculations can be done by relatively simple models using prismatic channels and one dimensional (section averaged) techniques, such as those facilitated by the 'Flow Profile' tool.

The 'Flow Profile' tool is a one-dimensional, steady flow model based on a prismatic channel. It facilitates analysis of bridges and culverts as described in its associated documentation.

Advanced techniques, such as multi-section (e.g. HEC-RAS), two-dimensional, and unsteady flow calculations are not necessary and offer little value in bridge design. Some of the reasons to avoid using more complex models include:

- *Boundary conditions are one dimensional anyway*
- *Natural rivers have mobile boundaries (scour, bedforms, lateral erosion)*
- *Many natural factors cannot be modeled accurately – drift, ice, sediment transport*
- *Data-sets don't exist to support true calibration of complex models*
- *Complicated outputs are difficult to interpret and assess*
- *These models are expensive and require significant resources*
- *Most of the output, accurate or not, is not needed to design a bridge*

As noted in section 2.2.3, channel capacity calculations do not account for flow adjacent to the channel in the floodplain. Hydraulic calculations, confirmed with observations during floods, suggest that the down-slope component of flow on the floodplain is a small portion of the channel flow (typically < 10%).

Reasons for this include:

- *Relatively shallow Y and low V (high relative roughness)*

- *Lack of a defined and continuous channel in the floodplain*
- *Presence of many natural (trees, topography variation) and man-made obstructions (roads, development)*
- *Water is trapped by backwater from the channel as it meanders across the floodplain*
- *Most flow is a lateral interaction with channel as levels change*

Even with the relatively simple models used in bridge design, it is important not to confuse accuracy with precision. With limitations in describing channel geometry (present and future), assumptions in hydraulic parameters such as roughness and loss coefficients, and naturally occurring features such as drift, ice and sediment, calculated precision should not be inferred as accuracy. In general, if confidence in Y is +/- 10% and V is +/- 20%, the parameters are acceptable for bridge design purposes. Some level of sensitivity analysis should always be incorporated into the hydrotechnical design of stream crossings. For reporting purposes, Y and V should be rounded to 10% (min. 0.1m for Y, min. 0.1m/s for V).

2.3.2 Bridge Hydraulics

Sizing a bridge opening involves placing the bridge fills and setting the roadway gradeline to provide desired clearance to highwater (freeboard). For locating bridge fills, a common starting point is to place the fills in line with the channel banks at the crossing location. From this point, a range of narrower (more constrictive) and wider options can be considered as part of the optimization process.

As the degree of constriction increases, V will increase through the opening and headlosses will increase with V^2. Headlosses will be due to the expansion of flow as it leaves the bridge opening, higher V through the bridge opening, and the contraction of flow as it enters the bridge opening. Potential impacts of a constricted bridge opening include:

- Need for more and larger rock protection
- Potential impact on adjacent bank erosion
- Potential increased flooding impacts on adjacent upstream development
- Reduced freeboard, possibly requiring a gradeline raise

The magnitude of these impacts can be assessed with hydraulic modeling as per section 2.3.1. Hydraulic impacts are generally not sensitive to small changes in fill location, especially for low V crossings. Additional criteria for hydraulic modeling for bridges are as follows:

- Hydraulic modeling is typically only required for constricted options (less flow area than the typical channel), or high V in the channel (>3m/s).

- Boundary conditions are Y, V, Q as per section 2.2.2.
- The impact of lost flow area to protection works and piers should be considered for smaller crossings (B < 30m).
- Head losses at the contraction and expansion of flow should be based on the differential velocity head, with common coefficients (K) of 0.3 for contraction and 0.5 for expansion.

Bridge openings wider than the typical channel can provide advantages in less extent of protection works, or even no protection works if sufficient buffer is provided between the toe of the headslope and the top of existing bank. In some cases, these benefits may counteract the expense of additional bridge length. However, excavating banks to provide a hydraulic opening larger than the typical channel can result in adverse impacts, such as sediment deposition and local flow alignment issues that can lead to increased bank erosion.

In a similar manner, the freeboard provided at a bridge should be determined through optimization. The starting point for freeboard assessment should be 1.0m between highwater (or high ice) and the lowest point of the bottom of the girders above the water. Lower values can be considered if the following conditions are met:

- Lowering the gradeline by reducing the freeboard could result in a significant reduction in cost
- There is a high degree of confidence in the design highwater level
- There is limited potential and/or history for drift or ice accumulation at the site
- The bridge is on a significant grade, with most of the bridge having more clearance than the minimum.
- A single span bridge (no piers) is proposed, with less risk of blockage

Higher levels of freeboard would seldom be justified hydraulically, but may result from optimization of the bridge concept.

Past practices (documented in early 1990s) for freeboard at Alberta Transportation bridges was 1.0m for major bridges, and between 0.3 and 0.9m for culverts and standard bridges, depending on the roadway classification. Strict adherence to these values was often found to restrict optimization of the bridge conceptual design.

For reference, the CHBDC (section 1.9.7 as of 2006 edition), can be summarized as requiring freeboard not less than 1.0m for bridges on freeways, arterials, and collector roads, and not less than 0.3m on other roads. As noted in section 1.2, this requirement does NOT apply to Alberta Transportation projects.

2.3.3 Culvert Hydraulics

Culverts are available in a range of shapes and there is a potential for multiple barrels at a crossing. However, the most cost effective and common solution is usually a single round pipe. Due to the culvert shape not matching the shape of the channel, there will usually be a flow contraction and expansion involved. As with bridges, the amount of headloss will be dependent on the magnitude of V through the opening. Most culverts will operate under subcritical flow conditions (flow depths controlled by downstream conditions).

All main pipes at a crossing are to be embedded (buried) below the streambed, as per the 'Design Guidelines for Bridge Size Culverts' document.   As such, most culverts will operate under subcritical flow conditions (flow depths controlled by downstream conditions). Therefore, a useful starting point for sizing a single round culvert is the flow depth (as per 2.2.2) plus the burial depth plus a rough estimate of the headloss. This would result in a culvert opening that would have close to no clearance (freeboard) between upstream water levels and the crown of the culvert.

From there, the culvert opening can be optimized for the site. Hydraulic calculations are typically more complicated for culverts than for bridges, due to factors such as the different shape of opening, the burial depth, and the potential for full flow. Various combinations of rapidly varying flow, gradually varied flow, normal flow, and full flow are possible. Supercritical flow may result in some cases, with the potential for hydraulic jumps. As such, hydraulic modeling based on the principles of section 2.3.1, is recommended to assess each option being considered.

All of these hydraulic conditions can be modeled by the "Flow Profile" tool.

Manning roughness parameters for culverts are available from various sources, although results are often not that sensitive to this parameter. Commonly, 'n' ~ 0.03 is used for corrugated metal pipes, n ~ 0.013 for concrete pipes, an n ~ 0.016 for smooth wall steel pipes. As with bridges, head losses at the contraction and expansion of flow should be based on the differential velocity head, with common coefficients (K) of 0.3 for contraction and 0.5 for expansion.

A sensitivity analysis, using the next bigger or smaller pipe diameter should be done. If a single round pipe is problematic, multiple pipes or elliptical shapes can be considered and evaluated. In addition to evaluation of fish passage (section 2.3.4), drift blockage (section 2.5.2) and icing (section 2.5.1), the following issues should be considered in evaluating potential culvert sizing options:

- Upstream flooding impacts - consider impact on flood sensitive upstream developments. The magnitude of any impact of a stream crossing on upstream developments is largely dependent on the resulting headloss

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(increase in water level due to constricted opening) and drift blockage potential

- Protection works – high velocities at the culvert ends may require larger rocks and a greater extent of protection works. High velocity flows directed at unprotected banks downstream may result in increased erosion. Insufficient protection works at the downstream end may result in scour holes, which can impact stability of the structure and adjacent banks.
- Uplift failure - culvert ends should be checked against potential hydrostatic uplift pressure if design water levels upstream and downstream are higher than the crown of the culvert. Additional weight on top of the culvert ends, or installation of a cutoff wall may be required.
- Embankment stability – excessive headloss due to a constricted opening can result in a large differential head across the culvert embankment, resulting in potential for piping failure. This can be mitigated with extension of clay seals or installation of an impermeable membrane.
- Road overtopping – excessive headloss can also result in the road being overtopped. In addition to interruptions to traffic under highwater conditions, erosion of the downstream embankment may result.
- Future rehabilitation - for high fill and high traffic volume crossings, consideration should be given to increasing the culvert size to allow for future lining with minimal traffic interruption. An increase in diameter of one standard size will often allow for a future culvert barrel to be installed inside with the space between the barrels grouted.

2.3.4 Fish Passage Hydraulics

For culverts on fish bearing streams, the potential for fish passage needs to be assessed and designed for. The flow value (Q_{FPD}) used for this assessment is as defined in section 2.2.6. The main principle for assessment of fish passage is that the mean velocity throughout the culvert should be less than or equal to the mean velocity in the channel at Q_{FPD}. Burial depth will create a backwater curve within the culvert, starting at the outlet, which will result in lower mean velocities. This backwater impact reduces with distance from the outlet, and normal flow conditions can be reached if the culvert is sufficiently long or steep. The hydraulic analysis procedures to undertake this mean velocity comparison are as per section 2.3.3. When comparing mean velocities, the precision of the mean velocities should be extended to 0.01m/s due to the relatively low magnitude.

The reasoning behind the velocity comparison principle is that if the fish can handle certain velocities in the stream to get to the culvert, the culvert should not be a velocity barrier to them. Velocity has traditionally been the main criteria used in evaluating fish passage at culverts. It has been suggested that point velocities will be higher in culverts due to the uniform section. However, the 'Velocity Distributions Impacts on Fish Passage at Culverts' document shows that there is still significant variance in point velocities within culverts and there

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are typically significant areas of low velocities within culverts. It has also been suggested that natural channels provide more opportunities for rest than culverts. However, many of the channels crossed by culverts have reaches with relatively uniform cross sections over the typical length of culverts. Therefore, the mean velocity comparison approach still appears to be the most practical method for evaluating fish passage.

This approach does not involve the use of fish swimming performance curves. These curves have often resulted in mean velocities that are a small fraction of the mean velocity in the channel and that cannot be physically met with a culvert or bridge crossing.

If the fish passage condition is not met, slight changes to the culvert configuration can be considered. In general, increasing pipe diameter is a very cost-ineffective method of reducing mean velocities at Q_{FPD}, as most of the extra area will be above the flow depth. Increasing the burial depth is also problematic, as it can lead to construction difficulties due to increased excavation depth and will result in a more difficult upstream transition from the culvert to the channel for the fish to traverse. If a configuration solution cannot be found, installation of substrate and holders should be considered, as per the details in the 'Design Guidelines for Bridge Size Culverts' document. Substrate holders assist in retaining substrate material, increasing the effective roughness of the culvert, but without resulting in a potential obstruction to upstream migration of fish.

The hydraulic effectiveness of the substrate can be assessed by blocking off the flow area filled by the substrate and increasing the roughness. The effective Manning roughness coefficient due to the relative roughness of the flow over the rocky substrate depends on the substrate type and mean flow depth. The appropriate coefficient can be found in this table:

Y	n - Class 1M	n - Class 1
0.1	0.161	
0.2	0.079	0.141
0.3	0.064	0.095
0.4	0.057	0.079
0.5	0.053	0.071
0.6	0.050	0.065
0.7	0.048	0.062
0.8	0.047	0.059
0.9	0.046	0.057
1.0	0.045	0.055
1.1	0.044	0.054
1.2	0.044	0.053
1.3	0.043	0.052
1.4	0.043	0.051
1.5	0.042	0.050

The Flow Profile tool can block the substrate flow area and adjust the roughness parameters, if a substrate value is entered.

2.4   River Protection Works (RPW)

Fills placed adjacent to the active waterway generally require some form of protection to avoid erosion. Protection may also be required along existing banks that are vulnerable to erosion from channel flows.   The major components of river protection works systems include headslope protection, guidebanks, spurs, and extended bank protection.

Rock riprap is the preferred material for protection of bridge headslopes, culvert ends, and related river protection works, such as spurs and guidebanks. Bio-engineering solutions and engineered materials (such as concrete) are not used for these works in the proximity of a stream crossing.

Reasons for the use of rock riprap include:

- *Over 40 years of proven performance history with rock systems that can resist drift, abrasion and ice forces with the flexibility to accommodate settlement and launching*
- *Proven velocity based criteria for selection of rock protection systems, with many publications and studies (e.g. FHWA HEC 20)*
- *Relatively low cost and generally readily available sources of rock riprap*

- *Laterally mobile streams require a "hard" fixed solution to maintain alignment through the opening*
- *Consequences of protection works failure at stream crossings to public safety are significant*
- *Alternative systems have been tested but many have been found to be unreliable or uneconomical*

Bio-engineering options, such as willow staking within a rock riprap protection system, may compromise the function of the geotextile and impact hydraulic capacity of the bridge opening on smaller channels. These options are considered by Alberta Transportation for some projects that are beyond the extent of a stream crossing, such as fish mitigation works in a channel, and river protection works for locations where highways encroach on streams.

Gradations for classes of rock riprap in common use on department projects are detailed in section 10.3 (as of 2013 edition) of the "Standard Specifications for Bridge Construction" document. Class 1M riprap is effectively an oversized pit run material, and is seldom used on bridge-size structure projects. Selection of the appropriate class of rock riprap is based on V at the design flow, as follows:

Rock Class	V max (m/s)
1	2.5
2	3.2
3	4.0

The gradation of the rock riprap, with certain percentages in certain size ranges, is important to ensure interlocking of the rock. This is particularly true for sloped portions of riprap. In some high velocity cases, a modified gradation has been used for aprons, with smaller sizes excluded from the mix. The angularity of the rock (less rounded) becomes more important at high V sites as the rock is more likely to interlock when it is angular.

For sites where V exceeds 4.0m/s (not common), addition of H piles or sheet piles in the apron may be considered to enhance protection. A larger size of rock than Class 3 is typically not considered practical due to limited availability and difficulty in transportation and placement.

The typical rock protection detail (see RPW Definition Sketch below) involves:

- Lining the bank with a single thickness (denoted as 't' on sketch, equal to the maximum rock diameter) of the selected class of rock riprap
- Double thickness launching apron at the toe, half buried. The extra thickness will accommodate rock launching into an adjacent scour hole. Typical length of apron is 4-5 times the maximum thickness of the rock.

- The sloping portion of the embankment to be protected is at a maximum slope of 2:1 (Horizontal : Vertical). Trimming of the natural bank may be required for extended bank protection options (no fill placed).
- The protection generally extends up to the design highwater elevation.
- Rock is underlain by a non-woven geotextile filter fabric with appropriate key-in, to prevent the loss of fines under the rock. Specifications for the filter fabric are detailed in section 10.4 (as of 2013 edition) of the "Standard Specifications for Bridge Construction" document'. The key-in typically involves about 0.3m of the top edge of the filter fabric being trenched vertically into the fill.
- For protection placed on earthen material, extending the earthen slope vertically by 1m above the top of rock will provide a suitable working base for placing protection works.

RPW Definition Sketch
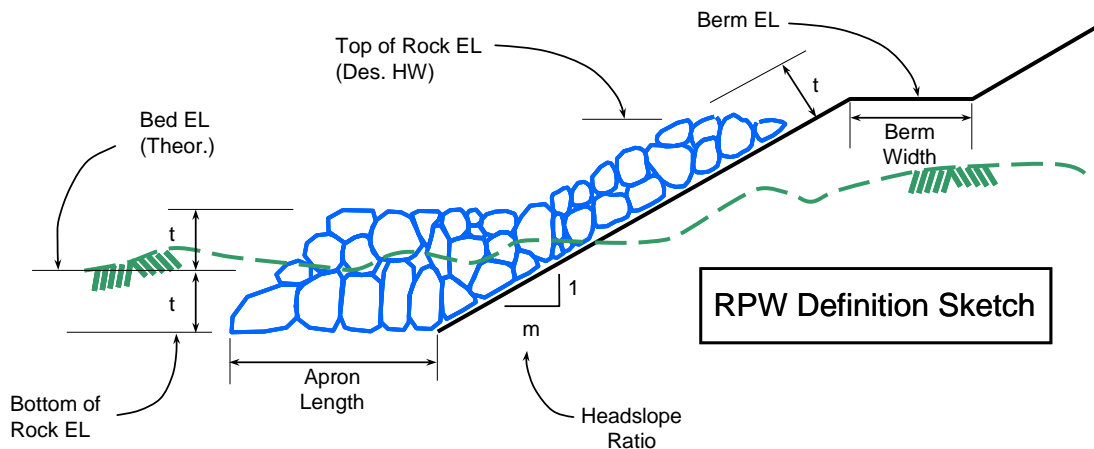
A key component of any river protection works system is that it extends well into a stable bank or naturally protected feature. This will minimize the risk of the protection system being outflanked. This is particularly important for works extending upstream. On streams with a high degree of lateral mobility, a river engineering analysis will help to determine the extent and composition of the protection works system that is required. Stable natural features, such as rock outcrops, should be utilized as appropriate.

Guidebanks are protected fills built parallel to the flow, and are commonly used to improve and maintain flow alignment through bridge openings on laterally mobile streams. Guidebanks typically extend from the headslope, and flare back towards the bank in an elliptical shape with a 2:1 ratio of distance along the stream to distance perpendicular to the stream. The transition from the natural channel into the bridge opening should be smooth.

Spurs are fills that project perpendicularly into the river, with protection works on the ends. They can be used to deflect flow away from a bank to be protected. They are typically used in groups, and can be more cost-effective that continuous protection of a bank. Spurs with significant projection into the river may cause a contraction of flow (see section 2.3.1). In some cases, a guidebank may be required to transition the flow onto a group of spurs.

Principles for use in setting the spacing of spurs within a group are as follows:

- Spacing = 4 times the projected length of spur into the flow at highwater (each spur assumed to protect the bank for 2 times the projected length in both the upstream and downstream directions).
- Spacing should typically not exceed the bankfull channel width (minimize risk of channel relocating between spurs
- For spurs with short shanks (relatively small projection into flow), spacing = 4 to 6 times the effective protected width of the spur nose
- Adjustments to spur spacing may be necessary for flow alignment changes (e.g. bends).

Guidebank and spur configurations are often best developed in plan view on top of airphotos. GIS tools can be very effective is developing these systems. A 'Spur Planning Geometry' tool is available to support this.

2.5   Other Hydrotechnical Issues

2.5.1 Ice

Ice impacts on stream crossing designs include design forces on piers, vertical clearance for ice jams, and structure blockage due to icing (aufeis). Where historic observations of ice jams, flowing ice, and icing is available, these should be considered in determining design parameters.

Ice jams form when pieces of broken ice form a partial blockage of the channel (toe of the jam). The constricted opening may result in headloss, and the accumulation of broken ice upstream of the toe may result in sustained high water levels for long distances. The sustained highwater levels are the result of the increased wetted perimeter of the floating ice combined with the high effective roughness of the broken ice projecting into the flow from above, and the submerged thickness of the ice itself. Broken pieces of ice may also project some distance above highwater.

Ice jams can form during freeze-up, as weak ice layers break and shove, and during break-up. Break-up ice jams can form due to a weakened ice cover (melting) or due to increased runoff flows physically breaking competent ice. In

general, the more competent the ice, the more severe the ice jam. In some cases, ice jam elevations can be several metres higher than highwater from summer flood events. Ice jams can form and release very quickly. The ice in ice jams is still typically floating on the water underneath.

Some principles to consider in assessing ice jam potential are:

- Check all available flood records for the site (and sites upstream and downstream) for ice jam events. Note location of the toe of jam, the maximum depth or elevation in the jam, ice pack thickness, and competency of the ice, where available.
- Severe ice jam risk is high where there is potential for upstream portions of the basin to have significant runoff in spring while there is still competent ice at downstream sections (rivers flowing from south to north are an example of this)
- The maximum height of an ice jam is generally some distance upstream of the toe (downstream edge) and is a function of the ice thickness and roughness, as well as the depth of water below.
- If a jam were to form, it would require lateral support to remain in place. Therefore, the maximum elevation is restricted to the range of bankheight plus the thickness of the ice.
- Trees along the channel banks can be inspected for ice scars, which typically are portions of the trunk facing the stream with the bark removed by abrasion from the ice.
- Ice jams tend to form at locations where there are significant changes to channel slope, width or plan-form (e.g. split by island). Confluences with large tributaries may also be more susceptible to ice jam formation.

When ice covers break-up or ice jams release, floating ice chunks will move downstream and potentially impact in-stream piers. This can result in significant structural loads applied to the piers. As defined in the Canadian Highway Bridge Design Code (Section 3.12 as of the 2006 edition), there are three components of these loads are ice strength (classified into 4 situations, a – d), elevation, and ice thickness. As with ice jams, historic records should be reviewed for information that may help quantify these three parameters. Abrasion on pier nose plates may be a useful source of information.

When sufficient historic observations do not exist to support determination of these parameters, the following parameters can be used:

Damage History	Small Stream (B < 50m)	Large Stream (B > 50m)
Minor	Sit. 'a' EL ~ 0.8 * Y t ~ 0.6m	Sit. 'b' EL ~ 0.6 * Y t ~ 0.8m
Major	Sit. 'b' EL ~ 0.6 * Y t ~ 0.8m	Sit. 'c' EL – observations. t ~ 1.0m

In this table, 'Y' is the design flow depth (section 2.2) and 't' is the design ice thickness. In some cases, two combinations of parameters may be appropriate, such as weaker load at higher elevation and stronger load at lower elevation. Subsequent structural analysis can determine which set of parameters may govern pier design.

Icing (aufeis) is when a culvert can become blocked by solid ice. The ice can form throughout winter due to repeated freezing of water supplied to the site, such as from an upstream spring. During spring runoff events, the culvert may remain blocked, restricting flows and causing flooding and possibly overtopping. The occurrence of icing should show in maintenance records, as it will likely be a frequent issue, if it occurs. Options to address icing include a larger opening than the existing pipe, or installation of a perched culvert of smaller size located above the main pipe to handle flow while the main culvert is blocked. Maintenance, involving removal of the icing before spring runoff by thermal or physical means, may be the only practical solution for some sites.

2.5.2 Drift

Large runoff events on natural streams are frequently accompanied by large amounts of drift, which can block openings and change flow alignments. Factors indicating potential drift problems at a site include:

- Significant amount of trees near the channel and its tributaries in the drainage basin
- Laterally mobile streams with active bank erosion, causing more trees to fall into the stream
- Historic highwater and maintenance records noting drift issues
- Drift accumulations at piers and on point bars (natural deposits of bed material extending from the bank into the channel)
- Presence of beaver dams

In general, culverts are more prone to drift related problems than bridges, as the surface width provided by the opening decreases at higher stages. Some drift mitigation options for culverts include:

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- Increase in opening size - may provide some benefit, but likely not cost-effective.
- Flared inlets with raised crown elevations have been used at some concrete culvert crossings to allow water to continue to flow in the event of a drift blockage
- Drift alignment piles have been used at some sites to align drift with the opening. This may require expensive modeling to correctly configure the piles, and their success is not guaranteed in the event of channel changes.
- A single round pipe is preferable to multiple smaller pipes for handling drift.
- Drift collectors, such as porous steel racks placed upstream of a crossing, have proven to be problematic. In several cases, they have failed due to outflanking during a runoff event, releasing an accumulation of drift towards the crossing.
- If a bridge is not a practical option, a culvert designed to withstand a potential blockage and requiring maintenance to remove drift during a runoff event, may be the most cost-effective option.

For bridges, mitigation options include:

- Reducing number of piers
- Locating piers outside of main flow
- Increasing span length over main flow
- Providing additional freeboard to minimize risk to superstructure
- Consider potential flow deflection against banks in configuring protection works
- Remove drift from piers at existing bridges with pier scour vulnerability

2.5.3 Scour

Most natural channels have mobile boundaries. Lateral movement of stream banks is generally referred to as erosion. Vertical changes in the streambed are generally referred to as scour. The two main types of scour relevant to bridge design are:

- constriction scour - higher velocities through the constriction may result in general lowering of the streambed throughout the opening
- pier scour – local hydraulic conditions due to the obstruction may cause scour holes to form

In general, bridges are not designed to be very constrictive (see section 2.3.2) and protection works are designed with launching aprons (section 2.4). Therefore, there is generally no need to quantify potential constriction scour.

Modern bridge construction equipment and materials has made piled in-stream foundations cost-effective. Spread footings are seldom used anymore. As long as the piled foundation penetrates > 5m into the streambed, pier scour should not be an issue for a stream crossing.

In the rare case of spread footings or short piles being proposed for a site, the bottom of the foundation should be lower than the estimated pier scour depth. Pier scour depth can be estimated as 2 times the effective pier width facing the flow alignment. If this is not practical, a layer of rock riprap, with the top surface flush with the streambed, should be placed around the pier. Rock placed above the streambed could result in increased local scour due to an increase in effective pier width.

For streams with a high potential for lateral mobility and constrained protection works extent (the ends of the river protection works were not sufficiently tied into a stable feature), consideration should be given to using scour resistant land-based piers and abutments. A scour resistant design would mean that the bridge could hold its own weight, without damage, if soil support above the streambed were to be removed. This may require rock protection to be embedded in the fill behind the pier or abutment to provide protection should the channel move laterally.

Some existing bridges with spread footing foundations and observed scour holes close to the bottom of the foundation may require rehabilitation to maintain structural safety. For bridges with significant remaining life, this should involve addition of piles to the foundation. For bridges with little remaining life, this could involve placement of rock in the scour hole. Care should be taken to minimize rock located above the streambed.

Other structures that are potentially vulnerable to scour are retaining walls and open bottom culverts. Loss of soil support for these structures could result in sudden and complete failure of the structure. If these structures are proposed within the active waterway, they should be protected from the potential impact of scour. This could be in the form of deeper foundations, a buried cutoff wall to prevent loss of soil support, and a layer of rock in front of the buried cutoff wall. Due to remaining vulnerability and the significant cost of these remedial features, these structures are not recommended for use at stream crossings.

2.5.4 Degradation

Degradation can be defined as the long term lowering of a channel elevation over a significant distance. It can occur naturally or as a result of manmade activities, such as channel straightening. Degradation can result in unstable banks and exposed substructure elements. It is important to differentiate degradation from scour, as it will require a different design configuration.

Signs that degradation may be an issue at a site include:

- comparison of historical streambed surveys, where available
- comparison of historic airphotos, showing progressive slumping and channel deepening over a significant length
- history of hydraulic structures and channel modifications on the stream
- ongoing maintenance concerns
- ravine like section approaching a confluence

If it is determined that degradation has occurred, some judgment will be required to determine if further degradation may occur. This can be based on changes in rates of progress over time, and whether the degradation was caused by man-made (reaction of one-time intervention) or natural (potentially ongoing) activities.

For bridges, predicted degradation will affect the design streambed elevation for placing protection works, the location of fills to allow for additional lowering with stable headslopes throughout the lifecycle, and the design of the foundation.

In many cases, existing culverts will have arrested progression of the degradation further upstream resulting in a large step in streambed profile at the crossing, typically at the culvert outlet. The new outlet will have to be placed to suit the new, lower channel elevation. However, the options for setting the inlet elevation are:

- Place it relative to a future profile with degradation allowed to progress beyond the culvert. The inlet would be in the range of the culvert length times the new channel slope higher than the outlet. A pilot upstream channel would also be required. This would result in a significant excavation in the upstream channel, which could force the ultimate bank slumping due to degradation to occur to happen during construction.
- Place it relative to the existing upstream channel elevation, locking the degradation into the culvert crossing. This protects the upstream channel and banks by effectively making the culvert a drop structure. However, the culvert will be very steep with high velocities and increased need for end protection works.

Neither of these options will be conducive to fish passage.

Additional information on degradation, its impact on bridges, mitigation options, and case studies can be found in 'Degradation Concerns related to Bridge Structures in Alberta'.

2.5.5 Channel Realignments

In some cases, channel realignments in the vicinity of a crossing can result in more cost-effective and sustainable solutions. Many projects, such as twinned

highway structures, high fill culverts, and bridges on highly mobile streams require some form of channel modification to work.

The main principle in designing effective channel realignments is to mimic a stable section of natural channel in plan-form, cross section, and profile. This should result in a stable, low-maintenance, and low environmental impact solution. It is important to communicate this to regulators in the approval process.

The main benefit of a channel realignment is reduced skew at bridges and culverts, resulting in simpler designs and shorter, less expensive structures. Flow alignment can also be improved. There may also be an opportunity to enhance fish habitat.

3 **Conceptual Design**

3.1 *Data*

In addition to the hydrotechnical data (section 2.1), data on site topography and existing infrastructure is required to complete bridge conceptual design. Much of this data can be accessed from GIS data-sets and other published data sources. Geotechnical and environmental (fisheries) data may also be required by technical specialists to provide their parameters and recommendations.

Most topographic data can be extracted from digital elevation models. For conceptual design, high-precision DEM data, such as <= 1m resolution LiDAR, should meet most ground surface needs, if available. Additional data may be available from other GIS data-sets, such as land ownership information and utility location.

For conceptual designs, this data may need to be supplemented by site survey, for elements such as:

- Streambed elevations (most DEM data-sets do not have ground elevations below water). Hydrographic survey may be required for larger crossings.
- Locations of utilities not covered by GIS data
- Reference benchmark for use in detailed design and construction
- Channel alignment and bank changes since time of DEM
- Precise structure measurements if components of the existing bridge are to be retained (e.g. superstructure replacement) or may impact the construction of the new bridge.

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Sufficient topographic and survey data should be obtained to complete the necessary bridge concept sketches (section 4.3).

Highway geometric and usage data may be required to assess the functionality provided by the existing highway and bridge. The geometric data can be derived from DEM and GIS data, as well as from as-constructed drawings. Items of interest include:

- Horizontal alignment - curve radius, spiral parameter, crown/super-elevation, clearance to barriers
- Profile – maximum grades, vertical curve lengths, K values for sags and crests, grades on bridge
- Other – traffic volume, highway classification, detour length, functional plans

Any relevant geotechnical data from bridge records should be made available to the geotechnical specialist for the project. This includes:

- testholes from drawings
- pile driving records
- existing headslope ratio and performance
- observations of slides and settlements during construction and the life-cycle of the existing bridge

3.2   Option Development Process

As per section 1.4, development of feasible bridge concept options is a key component in the bridge conceptual design process. Many options may need to be considered before the optimal solution is determined.

Modern tools, such as Bridge Planning Geometry (BPG) tool combined with GIS tools (e.g. Global Mapper) greatly facilitate this process.

The typical process to develop a feasible option is as follows:

1. Select a horizontal alignment
2. Set a profile on this alignment
3. Select the optimal bridge width
4. Establish a bridge opening (locate fills) plus protection works
5. Quantify costs and identify pros and cons

There may be a need to step back in the process to refine certain elements in order to derive a feasible option. Many feasible options may result due to the combination of alignments, profiles, and bridge openings.

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The expected service life of a bridge structure is typically much longer than that of the approach roadway. As such, geometric constraints imposed by a bridge may restrict future roadway improvements for a long time, or result in significant costs to modify the bridge to meet new road geometrics. On combined road/bridge projects, the bridge geometric constraints must be integrated into the overall project design, including consideration of future highway improvements.

For stand-alone bridge replacement projects where highway geometric deficiencies are present, potential bridge impacts on future highway improvements should also be considered. In this case, however, the bridge replacement project does not necessarily require immediate improvements to the adjacent highway. Short-term tie-ins to the existing roadway may be the most cost-effective solution. Adjacent roadway alignment deficiencies, such as horizontal curves, vertical crest curves, and maximum grades can be addressed through a separate project at a later date. The ability for a given bridge option to work with a potential future roadway improvement should be documented in the conceptual design report.

A minimal option to be considered for stand-alone bridge replacement projects involves replacing the bridge at its current location with similar highway geometry. The upper bound option would be one that meets all conditions of the design roadway designation for the given service classification and traffic volume (Figure A-3.2ii, AT Highway Geometric Design Guide (GDG)). Additional options between these extremes may include improvements to geometric deficiencies through modified alignments and/or gradelines.

In general, it is desirable that a highway consistently meets a certain minimum standard and service level throughout its length. However, reductions in functionality (design speed) may be appropriate in highly constrained areas, such as the deep valleys associated with many bridge crossings. The site topography, combined with appropriate signage, should provide sufficient information to the road user that conditions are different for this section of highway.

3.2.1 Horizontal Alignment

Options for horizontal alignment of a bridge include:

- Existing alignment
- Modified alignments with adjacent bridge (avoid detour bridge/staged construction)
- New routes

When considering new routes for stream crossings, major factors to consider are:

- Crossing location – bank stability, flow alignment, crossing skew, channel width, geotechnical and fisheries issues

- Alignment topography – steep grades on valley wall
- Potential geotechnical (slide) issues along valley walls – options that run perpendicular to valley walls are often preferred in areas with slide potential (as opposed to side-hilling), although this can result in higher grades and/or grading volumes.
- Land severance impacts
- Route length

Additional factors to consider for adjacent modified alignments are:

- Flow alignment impacts with existing bridge during construction
- River protection provided by the existing bridge protection works
- Potential for improvement to the horizontal alignment geometry
- Leave sufficient offset distance to minimize interference during construction (often 5m minimum between structures)
- Location of connections to existing paths and trails

Major differences between bridges and roads that can affect horizontal alignment options are:

- the presence of bridge barriers (shy offset, sight blocking)
- potential for preferential icing on the bridge deck
- higher unit cost for bridges (length and width)
- structural complexity and cost increase significantly beyond 30 degrees skew

Due to these differences, the horizontal alignment across the bridge should result in a constant roadway cross section across the bridge. In many cases, this involves a tangent section (preferred) or a constant radius curve across the bridge. Full tangent run-out should be provided between any spiral curves and the bridge deck. For detailing simplicity, it is desirable that the tangent run-out end before the out to out of fill point (point where bridge headslope would theoretically intersect with the gradeline).

Bridge barriers can also affect stopping sight distance (figure B-3.9b, GDG). Typically, bridge barrier impact on stopping sight distance and the super-elevation associated with highway curves will require that the minimum radius curve across a bridge deck be much larger than the minimum for the approach highway and design speed. Bridge barriers can also impact sight distance to an intersection on the approach road (section D.4, GDG).

Additional horizontal sight distance issues can arise at grade separation bridges, such as on ramp/loops with retaining walls, piers and high abutments.

The process of generating a horizontal alignment using BPG and Global Mapper is as follows:

1. Draw a multi-tangent line in Global Mapper, with DEM and mapping data as a reference. Move the points that make up this line until a potential rough alignment has been created.
2. Enter the coordinates for the points that make up this line into BPG, and add basic highway geometric parameters ('R', 'e'). This will generate a series of text files including the resulting alignment, stationing, and key alignment points (e.g. Tangent to Spiral, Spiral to Curve…)
3. Import these text files into Global Mapper to visualize result. Adjust alignment points and curve parameters as necessary, and repeat the process until a potentially feasible alignment has been developed.

3.2.2 Profile

Once a horizontal alignment option has been established, a profile can be added to it. Key criteria for setting a gradeline associated with a bridge are:

- For stream crossings, set bridge elevation over the stream to allow for flow depth (section 2.2), freeboard (section 2.3), and structure depth.
- For grade separations, set bridge elevation over the roadway (or railway) below to allow for required vertical clearance (Roadside Design Guide, Fig H7.1 to H7.3), and structure depth. Note that the required vertical clearance is the minimum between any point on the roadway (or railway) below and the underside of the girder above, and the location of this point varies with the configuration of the road above and bridge above.
- Initial structure depth estimates are often in the range of expected maximum span length / 20. The structure depth accounts for all elements resulting in elevation difference between the finished roadway crown on the bridge and the underside of girders, including girder depth, haunch, deck, wearing surface and crossfall (super-elevation or crown rate times the width of bridge from centerline).
- For grade separation structures, where the provided vertical must be very precise, structural input should be sought to refine the structure depth amount in later stages of conceptual design.
- Proposed future widening of a grade separation structure needs to be considered in the structure depth allowance.
- K values should meet the stopping sight distance requirements (crest curves) and comfort control (sag curves) for the design speed for the option (Table B4.4, GDG). K values for headlight control at sag curves is desirable.
- Bridge elevation can influence the maximum grades on approach roads at valley walls. Some guidance on maximum grades for approach roadways is provided in section B4.2, GDG. In general, the optimal gradeline should be sought, which may result in even lower grades at some sites, but higher than desirable grades at others (deep valleys, high relief terrain). Grading volumes and cut/fill balance should be considered (section 3.2.5).

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- The minimum desirable gradient on a bridge deck is 1.0% for drainage purposes (avoid localized pooling of water, which can result in premature deck deterioration). This may not be practical in some cases (e.g. perched bridges, where the bridge elevation is higher than all surrounding land). In all cases, a 1% minimum option should be evaluated. Any recommended options with minimum gradients less than 0.5% should be clearly communicated to the project sponsor and identified as a 'con' for that option.
- The maximum gradient on the bridge deck should be 4% resultant (~ 3.5% longitudinal for 2% crown) for control and braking in potential preferential icing conditions. Resultant gradient is the maximum slope on a diagonal resulting from the longitudinal gradient and the cross-slope. Exceptions may be considered if a preferential icing mitigation strategy is developed, or the maximum gradient is exceeded for a very short length (<20m). Preferential icing on a bridge deck (approach road not icy at the same time) can surprise drivers and result in a loss of control if any sudden adjustments are made. These conditions are common, but not easily predictable.
- Section B4.1 (GDG) provides some guidance on minimum lengths of vertical curves. Where feasible, a length of curve (in metres) exceeding 2 times the design speed (km/hr) should be used to minimize visual effects.
- Intersection sight distance for ramps at grade separations should be considered due to the sight line blocking by bridge barriers (sections B-4.4.2 and D.4, GDG).

Steps in generating a roadway profile using BPG are as follows:

1. Enter station and elevation data for initial vertical points of intersection (VPI) and curve lengths based on review of the profile plot generated from the horizontal alignment superimposed on the DEM.
2. Review the curve parameters for overlaps and meeting required parameters as stated above. Review the updated plot for approximate balance of cuts and fills (will affect total project costs).
3. Iteratively adjust VPI and curve length data until all parameters are met and a good visual fit is achieved.

3.2.3 Bridge Width

A bridge width that matches the road width provides continuity for traffic, which is desirable. However, bridges typically cost more than roads and road widths can change over time due to the addition of overlays. Therefore, the optimal bridge width will balance functionality provided and capital construction cost. Functional aspects of bridge width include:

- Shy line offset from bridge barriers
- Storage of stalled vehicles

- Bicyclists and pedestrians
- Snow storage
- Deck drainage
- Traffic accommodation during rehabilitation

Key criteria in setting the optimal bridge width include:

- Minimum bridge width on a paved highway is 9m.
- Minimum bridge width for 2 lane rural highways with AADT > 1000vpd is 10m.
- Minimum bridge width for 2 lane rural highways with service classification (Design Bulletin 27) of 1 or 2 and AADT >4000vpd is 11m.
- For bridges on rural divided highways, bridge width should match the design clear roadway of the roadway designation (this should match the final pavement width after all future overlays). Consideration should be given to ability to maintain 2 lanes (reduced width, lower speed) during bridge rehabilitation activities (may mean increased width).
- For bridges on urban fringe highways, the bridge width should meet the shy line offset to the bridge barriers, as per Table 3.1.6.4 (TAC).
- The maximum shoulder width shall be 3.5m to reduce risk of high angle strikes with bridge barriers.
- A one lane bridge may be considered for highways with no plans for future paving and AADT < 500vpd.

As per Figure 2.2.10.5 of the TAC Geometric Design Guide for Canadian Roads (TAC), there is little incremental collision reduction benefit beyond 10m width on a 2 lane bridge.

Other factors to consider for width of bridges include:

- If structure width provided by standard precast girders (such as SL, SLW, SLC) is within 0.3m of width requirements above, the width is acceptable (no need for additional girder line).
- Stopping sight distance on curved structures (Figure B3.9-b, GDG)
- Raised medians at diamond interchange bridges
- Tapers to loop ramps on interchange bridges
- For long span, non-girder bridges on divided highways, consider placing both directions of traffic on one bridge structure with a median barrier.
- Widening a bridge to improve deck drainage for in-frequent design conditions is typically not a cost-effective strategy

If a one-lane bridge is proposed, the following conditions shall be met:

- Bridge width should be between 4.5 and 5.5m (minimize risk of 2 vehicles using bridge at the same time).

- Operating speed should be a maximum of 50km/hr (enables vehicles to safely stop and yield, minimizes severity of potential collisions). Probability of adherence to posted speed limits should be considered.
- Sufficient sight distance should be provided for approaching vehicles to see a vehicle at the far end of the bridge with enough distance to stop and yield before reaching the bridge.
- Suitable signage shall be provided and implementation of other strategies to mitigate hazards shall be considered (e.g. fill width/approach rail configuration).
- Maximum bridge length of 100m (minimize risk and impact of on-bridge conflicts requiring one vehicle to back up off the bridge).

Note that highway bridges with these low traffic volumes will likely be low priorities for replacement funding. Other strategies, such as extending life of existing bridge, restricted use, or closure, may need to be pursued. One lane bridges may be impediments for certain types of vehicles, such as seed drills.
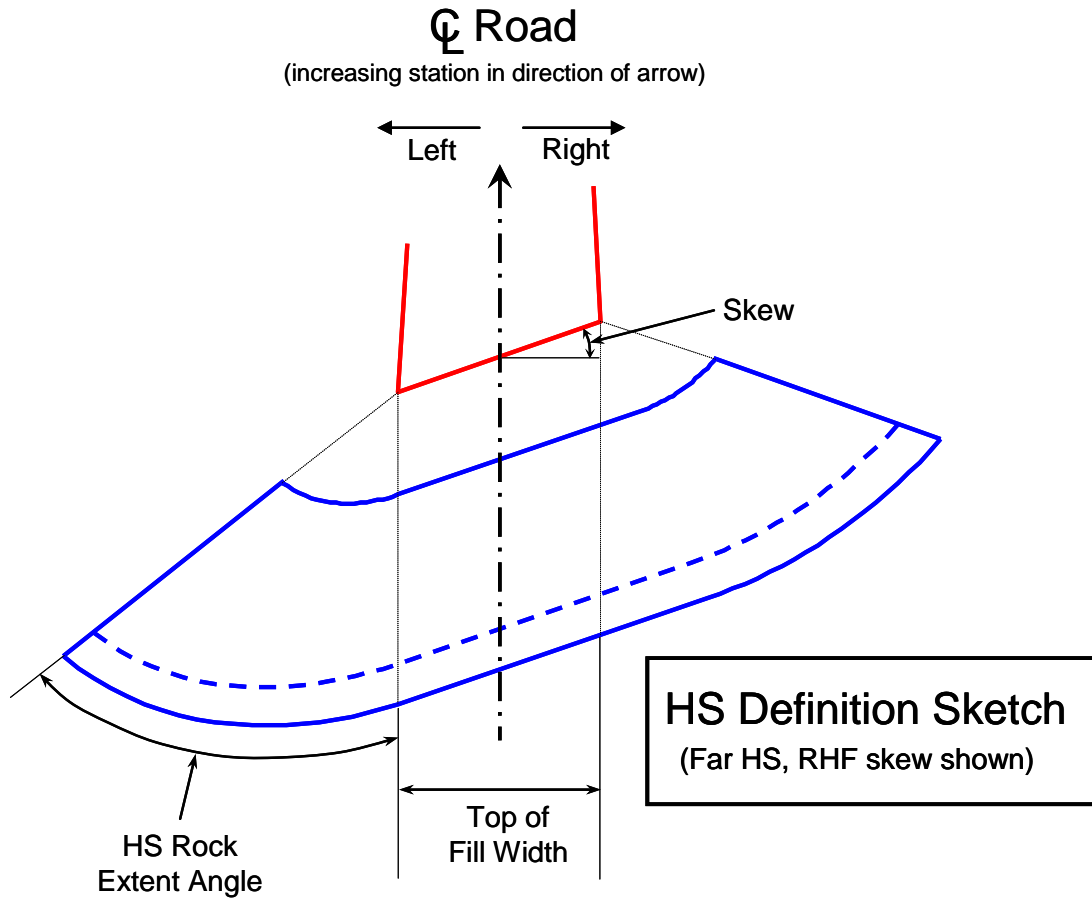

3.2.4 Bridge Opening

The bridge opening is largely defined by the bridge fills and the superstructure. Hydraulic aspects of locating bridge fills are discussed in section 2.3.2. The extent of any river protection works (section 2.4) will be linked directly to the location of the bridge headslopes and the skew angle of the crossing. Bridge fills in an active waterway should be aligned with the expected flow direction at high flows.
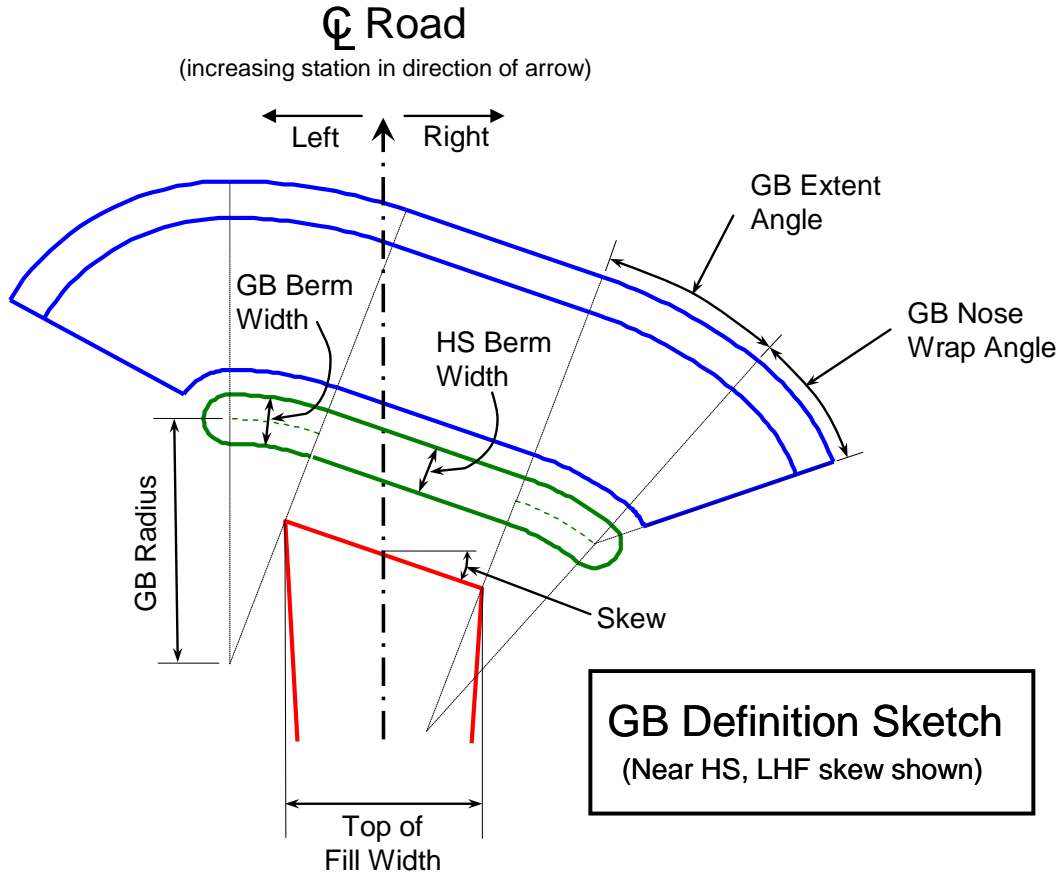
Bridge fills typically exceed the gross structure width by 2m at the top of fill (typically assume bridge clear roadway width + 3m at conceptual design stage). This provides a stable base for constructing the bridge abutments and provides lateral support for abutments and wingwalls. The maximum (and typical) headslope ratio is 2:1 (horizontal: vertical). Typical sideslopes for bridge fills are 3:1. An elliptical transition is provided at the corners between the headslope and the sideslope. The Bridge Fill Slopes Calculator tool has been published by Alberta Transportation to assist in laying out bridge fills in this manner (elliptical transitions have also been built in to the BPG tool). Flatter headslope ratios may be required due to geotechnical conditions, although mitigation works with a steeper headslope should be considered. For high fills (>10m), a horizontal bench, 4m in width, can be considered along the headslope. This will aid in improving geotechnical stability.

Fills that project into the active waterway will usually require protection works (see section 2.4). For normal bridge fills with elliptical transitions, the rock will be placed on the fill, with excavation at the ends to ensure the rock is tied in to the banks upstream and downstream (see HS Definition Sketch). An alternative is to use a guidebank, which maintains a 2:1 slope through the protection works, with the elliptical bridge fill transition located above and behind the guidebank (see

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GB Definition Sketch). If the bridge fills are outside the active waterway, and sufficient buffer distance to the active waterway bank is provided, protection works may not be required. An assessment of historic bank erosion and lateral mobility potential would be required to establish the required offset.

℄ Road

(increasing station in direction of arrow)

Left   Right

Skew

HS Definition Sketch

(Far HS, RHF skew shown)

Top of
Fill Width

HS Rock
Extent Angle

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℄ Road
(increasing station in direction of arrow)

GB Definition Sketch
(Near HS, LHF skew shown)

Although structural options are not examined in detail until later stages, potential options should be considered when developing bridge opening options. Expected span lengths will confirm assumptions made in setting the roadway profile to accommodate desired freeboard. Potential pier locations should be assessed for impact on drift and ice, in-stream berm needs during construction, and proximity to the toe of fills (too close may deflect flow onto the bank).

In general, site specific measures are not required within the bridge opening for wildlife passage. Features such as headslope berms, fill offsets, and the natural channel may provide some wildlife passage capacity, without specific modification. If there are significant collision issues and wildlife mitigation measures (such as fences) proposed as part of a highway project, additional measures could be considered. However, any extra cost and highway safety impacts due to a longer or higher bridge must be justified economically by predicted reductions in wildlife collisions.

Drainage from highway ditches needs to be considered in the design of the bridge fills. For most rural stream crossings, no specific stormwater management facilities, such as ponds, are required. Rural highways are designed with open channel ditches, which route and filter runoff. These ditches

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are intercepted by natural streams adjacent to the bridge crossings. Ditch drainage is typically directed into low lying areas adjacent to the channel, and erosion and sedimentation guidelines apply ("Design Guidelines for Erosion and Sediment Control for Highways"). Stormwater ponds may be required at large interchanges with significant increase in impervious area and/or regulated release rates in urban areas.

Culverts generally come with fixed opening dimensions. Transition zones will be located at both ends of a culvert, as the channel shape moves from the bevel end (or inlet section) to a trapezoidal shape that can fit the natural stream. The transition zones are also used to connect the culvert invert elevations (due to burial) to the natural streambed elevations. These transition zones are typically protected due to the rapid change in cross section and relatively high velocities. In some cases, berms perpendicular to the roadway may be required to support the protection works. Channel realignments (section 2.5.5) may also be beneficial in some cases, to reduce culvert skew (and length) and improve flow alignment.

Steps in generating a bridge opening option using BPG and GIS tools are as follows:

1. Enter the station values for the tops of fills, and enter key fill parameters such as headslope ratio and fill width.
2. Review fill locations on the bridge elevation plot relative to the channel. View fill outline in plan in a GIS tool (based on exported text file), on top of georeferenced airphotos and DEM layers. Confirm fill location and alignment. Adjust as necessary (may require several iterations).
3. Enter protection works data, if necessary. Review protection works on the bridge elevation plot and in plan view on top of GIS layers (airphotos, DEM). Note tie-in points to existing banks and alignment of flow through opening. Adjust parameters as necessary (may require several iterations).
4. Roughly assess structure configuration options using built in tools and review on bridge elevation plot and in plan view on GIS layers (and 3D).
5. Note results for bridge geometry and rock volumes for evaluation of alternatives.

3.2.5 Grading Volumes

With the horizontal alignment, road profile, and bridge opening established for a given option, an estimate of road grading impact can be determined. This will facilitate further optimization of the horizontal alignment and gradeline for the current option, and allow comparison with other options. In deep valleys, the grading volumes for cuts and fills can be significant, sometimes resulting in costs in the range of the bridge costs. In general, an approximate balance of cuts and fills will be preferable, to minimize the amount of borrow/waste material (of course, total project costs will be considered in the optimization process).

The BPG tool uses the information already entered and the DEM to estimate volumes of cuts and fills between entered stations. This allows for separate calculations for either side of the bridge, as transportation of excess material across the stream may not be feasible. Cross sections can also be generated at selected stations to confirm/review impacts at areas of interest. BPG also has the ability to export text files that can be imported into GIS tools to review the extent of fills and cuts in plan and 3D.

4 Optimization

4.1 Evaluate Alternatives

Once a range of feasible alternatives has been developed, they can be assessed to identify the optimal solution (or in some cases a sub-optimal option that will fit budget constraints). In general, the comparison will account for initial capital costs and a qualitative assessment of pros and cons. At the conceptual design stage, the comparison will typically only involve replacement options, as rehabilitation and maintenance options are generally considered in assessments. As such, there is limited need for life cycle cost analysis, unless options with different expected structure lives are being compared (e.g. steel vs concrete culvert).

Options may cover a range of alignments, profiles, and bridge openings. Conceptual design options may also cover a range of potential structures including steel culverts, concrete culverts, standard bridges, and major bridges. Costs will include all road and structure costs at a type 'A' cost level (as per section 1 of the Engineering Consultant Guidelines Vol. 1). Cost components may include:

- Structure costs - unit cost times deck area
- River protection works – unit cost times rock volume
- Road Grading - unit cost for balanced grading, unit cost for borrow/waste
- Road construction – unit cost times length of road
- Geotechnical remediation – lump sum costs based on similar projects
- Land/development purchase costs – unit cost times area
- Utility costs – lump sum for major items requiring relocation
- Construction detour bridge costs/construction staging costs

Pros and Cons will address issues not accounted for by the costs, such as:

- Highway geometric functionality (performance, safety)
- Route length
- Construction issues – timing, risks, difficulty, staging

- Environmental – impacts, approvals
- Land/development impacts – sensitive areas, purchase delays

Scoring matrices are not used in selection of the optimal bridge conceptual design. These systems, where points and weighting factors are assigned to various issues, tend to be too rigid in scoring, with considerable debate on scoring charts and weighting factors. If one clearly optimal solution is not readily identified, then multiple solutions of somewhat equal value likely exist, and the selection of one specific design has less consequence.

In some cases, some preliminary structural evaluation and a more thorough level of cost estimate (type 'B') may be required on multiple options to confirm the optimal solution. Examples are culvert versus standard bridge, and 3-span bridge versus 1-span bridge (these options would likely require different gradelines due to the differing structure depths).

A recommendation on optimal bridge concept will be required with the draft of the conceptual design report. This recommendation should be supported by evaluation of these costs and factors. After discussion with the department, direction may be given to proceed with the recommended option (or variant), a different option (e.g. lower capital cost), or evaluation of an additional option.

4.2   Design Exceptions

Design exceptions (Design Bulletin No. 72) are "instances where values lower than the minimum standard or a departure from the normal practices". The intent of the design exception request process is to "determine, justify and document that good engineering judgment is being exercised".

The bridge conceptual design process (section 1.4) is an optimization process, with the normal practice being consideration of many feasible alternatives offering a range of functionality. The process also includes departmental concurrence and thorough documentation of options considered and justification of the selected option. As such, the bridge conceptual design process addresses the principles involved in the design exception process, and a design exception request should not be required in most cases. It is the responsibility of the project administrator to ensure that appropriate effort has been undertaken and sufficient documentation is available to support decisions that have been made.

For cases where roadway geometric deficiencies relative to the road design designation (as per A-3.2ii of GDG) are proposed in the bridge conceptual design, a design exception request may be required if:

- The bridge is part of a new road design project and the bridge is the functional bottleneck for the project

- The bridge is the main part of a stand-alone bridge replacement project and the concept provides a lower level of functionality than the existing section of road.

Any reductions in effective design speed due to horizontal curves, vertical curves, and stopping sight distances must be reflected in the posted speed for that section of road.

Some of the key bridge related conceptual design items that should be clearly communicated to the project sponsor and justification documented in the bridge conceptual design report include:

- Freeboard at bridges < 0.3m (section 2.3.2)
- Protection works details other than as specified in section 2.4
- Variable cross section on bridge deck (section 3.2.1)
- Gradient on bridge deck < 0.5%, resultant gradient on bridge deck > 4% (section 3.2.2)
- Bridge width less or more than values in section 3.2.3 (or one lane bridge)
- Headslope ratios other than 2:1 (section 3.2.4)
- Skew exceeding 30 degrees

4.3   Report/Sketch Requirements

Once all alternatives have been developed and assessed, a report will be prepared highlighting all acquired data, analysis results, options considered, and justification of the recommended option. All assumptions, limitations, and constraints should be clearly noted. Any additional information required to complete the detailed design phase, such as design ice parameters and survey benchmark information, should be included. The report should include sufficient sketches that illustrate the concepts and facilitate review by an independent reviewer and the department.

Following direction by the department, the report should be updated to reflect the decisions made and provide additional detail on the optimal concept. The sketches for the recommended option should meet the following objectives:

- Confirm the feasibility of the concept
- Detail and communicate the bridge conceptual design to the structural design team
- Capture any future plans considered (e.g. lane additions)

This will typically involve a minimum of a site plan, elevation view, road profile view, and streambed profile. In the case of a culvert, the elevation view will run along the culvert. Additional plan views and details may be required for new road alignments, channel realignments, and river protection works. Superimposing design elements on georeferenced airphotos and 3D views may also be of value.

Bridge Conceptual Design Guidelines

The BPG tool and GIS tools, such as Global Mapper, will facilitate generation of base versions of these views, with the need for some annotation (e.g. dimensions, notes). The department has a large library of bridge conceptual designs, that can be referred to when developing these sketches, typically contained on DD drawings. The typical contents of these sketches are as follows:

Site Plan (plan view focused on stream crossing):

- Extents – include fill transitions to roadway along road, include tie-in to bank and lateral fill extents along stream
- Centreline of road with at least two station points shown
- Top and bottom of stream banks
- Bridge fill extents
- Headslope protection works
- Utilities (with labels)
- Existing and required right-of-way extents
- Land ownership
- Existing bridge (if applicable)
- Skew angle of opening relative to roadway
- Developments or significant natural features

Bridge Elevation (cross section along road alignment):

- Extents – same as site plan along the roadway
- Elevation on vertical axis, Station along road on horizontal axis
- Gradeline of existing road centerline (if applicable)
- Gradeline of proposed road centerline
- Profile of existing natural ground along proposed roadway (if applicable)
- Profile of existing natural ground offset on both sides of proposed roadway centerline (should be clear of influence from existing fills, e.g. 20 to 30m offset)
- Bridge headslopes from fill to natural ground (note slope ratio, station and elevation for top of fill points)
- Design Highwater Level indicated within opening
- Protection works (if applicable), detailing thickness, top elevation, bottom elevation, apron length and thickness
- If protection works proposed, dimension theoretical streambed width and note elevation
- Outline of existing bridge, including deck, abutments, piers and foundation, if applicable
- Test hole location and outline
- Road gradients at estimated bridge ends

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- For culverts – show invert elevations, burial, channel transition lengths, and all elements that affect culvert length, including sideslopes, berms, road cross section and clearances.

Roadway Gradeline (extended elevation view):

- Extents – cover extent of any proposed changes, include crest curves at valley tops (if applicable)
- Elevation on vertical axis, station along road on horizontal axis
- Gradeline of existing road centerline (if applicable)
- Gradeline of proposed road centerline
- Profile of existing natural ground along proposed roadway (if applicable)
- Profile of existing natural ground offset on both sides of proposed roadway centerline (should be clear of influence from existing fills, e.g. 20 to 30m offset)
- Bridge headslopes from fill to natural ground (note slope ratio)
- Design Highwater Level indicated within opening
- All BVC/EVC (include station, elevation)
- K value and curve length for all curves
- All vertical points of intersection (include station, elevation)
- Gradients along tangent sections
- Note location of accesses and intersections

Extended Plan View (superimpose on georeferenced airphoto):

- Extents – cover extent of any proposed changes along the roadway, cover any proposed protection works or channel changes along the stream
- Existing road centerline (if applicable)
- Proposed road centerline
- Regular station points
- All curve transition points with labeled stations
- Curve data (radius, spiral parameter, length, deflection angle) for all curves
- Road cut/fill extents
- River protection works and channel alignments not shown on site plan
- Adjacent land ownership
- Existing and required right-of-way extents
- Location of accesses and intersections

Stream Profile:

- Extents – cover extent of any protection works or channel changes
- Elevation on vertical axis, Station along the channel on horizontal axis, with zero at intersection with proposed road centerline
- Thalweg profile (lowest point on each channel cross section)

- Surveyed water/ice profile (note date)
- Slope arrow and value (based on DEM data, section 2.1)
- Extents of any channel changes (include outline of culvert, if applicable)
- Note any anomalies (rock ledges, beaver dams)
- Note any confluences with other channels
- Additional stream profiles may be required when channel realignments are proposed