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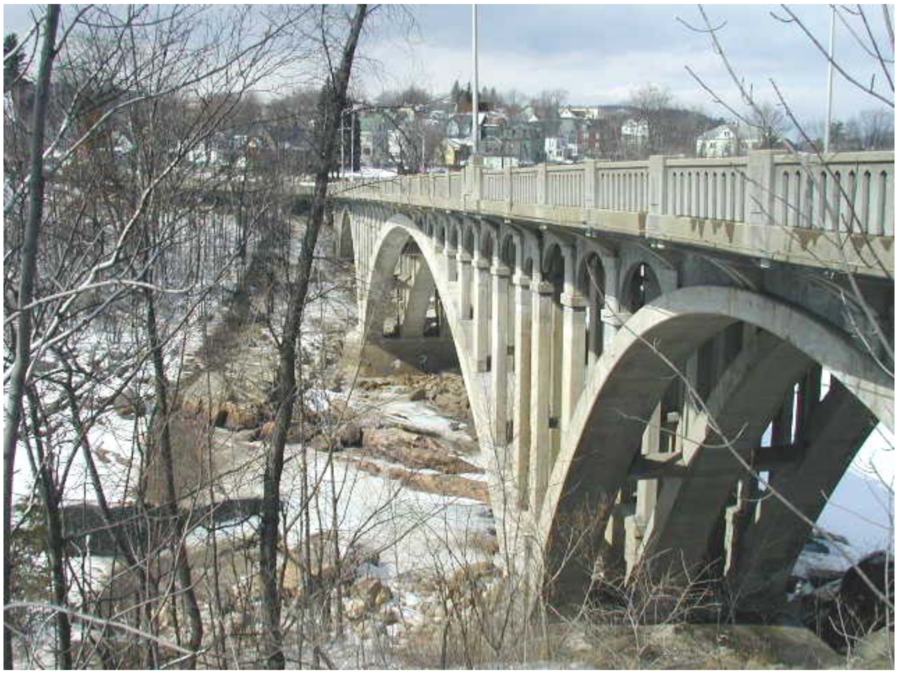
Bridge Design Guide, Chapter 2 : Preliminary Design, 2003

Maine Department of Transportation

Chapter 2

PRELIMINARY DESIGN



Chisholm Park Bridge, Rumford

Sandy Stream Bridge, Moose River Plantation

2 PRELIMINARY DESIGN

2.1 Preliminary Design Report

The Preliminary Design Report (PDR) documents the justification for decisions made in the conceptual design process. Forms are available electronically that assist in completing the PDR. At the end of the preliminary design phase, all those invested in the project have reviewed the scope of work, and this scope is considered final. The PDR is then used as the starting point to proceed to final design.

For those projects with spans of 50 feet or less, consideration should be given to a reduced preliminary design effort, as discussed in Section 1.5 Small Bridge Initiative.

The PDR is organized into the following sections. The depth of study and extent of investigation of options will depend upon the complexity of the project. Samples of completed forms are found in Appendix B PDR Forms. A description of each section follows the listed sections.

1. Title Page
2. Table of Contents
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17. Miscellaneous Information
18. Traffic and Accident Data
19. Estimates

2.1.1 Title Page

The Title Page contains the following:

PRELIMINARY DESIGN REPORT
BRIDGE NAME and NUMBER
OVER
RIVER NAME
TOWN, MAINE
FEDERAL PROJECT NUMBER
PIN NUMBER

2.1.2 Table of Contents

This should be a properly identified index of pages.

2.1.3 Background Information

This page provides a quick reference for background information on the project. Much of this information is found either in MaineDOT's ProjEx, the Planning Report, or Bridge Management's SI&A sheet, all of which will be provided by the Project Team. The following sections are completed as shown below:

Program Scope: Copy verbatim the scope from the Biennial Transportation Improvement Program (BTIP).

Program Reads: Copy verbatim the contents of the project description in the BTIP.

Project Background: Provide a brief written description of the project's background, including site review by the 6-Year Plan team, any previous studies and recommendations, requests by Towns, and any other pertinent information.

Structurally Deficient: A structure is structurally deficient if the condition rating for the deck, superstructure, substructure, or the culvert and retaining wall is 4 or less. A structure may also be structurally deficient if the appraisal rating for the structural condition or waterway is 2 or less.

Functionally Obsolete: A structure is functionally obsolete if the appraisal rating for the deck geometry, under clearances, or approach roadway alignment is 3 or less. A structure may also be functionally obsolete if the appraisal rating for the structural condition or waterway is 3. Any bridge classified as structurally deficient is excluded from the functionally obsolete category.

2.1.4 Location Map

This should be from the Highway Atlas, U.S.G.S., or another map showing the project location. Do not use copyrighted material such as a DeLorme's Maine Atlas and Gazetteer.

2.1.5 Bridge Recommendation Form

All portions of the Recommendation Form should be completed as shown below. A complete description of each component should be included under that component. There are several variations to this form depending on the project scope. If there are parts that are not applicable to the structure type, they need not be included.

Review by - Signature of Engineer of Design is obtained here prior to proceeding with any further work.

Project - State the type of project. Examples:

> "Bridge replacement with 300 ft of approaches, including transitions"
> "Bridge rehabilitation project with no approach work"
> "Bridge replacement as part of Arterial Program project"
> "Bridge replacement with approaches by Arterial Program"

Alignment Description - Give a description of the horizontal and vertical alignments at the structure location and the relationship to the existing alignment. Example:

> "1200' horizontal curve located approximately 30' upstream of existing bridge and a 500' sag (crest) vertical curve with a finish grade 3.5' higher than existing bridge."

Approach Section - Give a description of the typical approach section at the bridge, including the type of guardrail. Example:

> "Two 11' paved lanes with 3' shoulders (30' rail-to-rail) with standard sideslopes. 21" aggregate subbase course gravel with 3" pavement thickness. Type 3 guardrail."

Spans - Give the span lengths along the centerline of construction on straight tangents, and along working lines or chord lines for structures on a curve. If on a curve, indicate span lengths as "along long chord" or other descriptive indication. This section is not required for culvert-type structures.

Skew - Give the skew angle of the substructure units, or the centerline of a culvert-type structure, relative to the longitudinal working line of the structure. The skew angle should always be given as "Ahead on Left" or "Back on Left".

Loading - Indicate the appropriate design vehicle loading.

> For a typical superstructure:
> “HL – 93 Modified”
>
> For a culvert-type structure:
> “HS 25”

Superstructure - Give the design description and governing parameters of the superstructure. For culvert-type structures, this section is simply called Structure. Examples:

> For a typical superstructure:
> “Five rolled beams of A709/A709 M, Grade 50W steel with a composite structural concrete slab, elastomeric bearings, one compression seal expansion joint, and a 3” bituminous wearing surface with ¼” (nominal) membrane waterproofing. 36’ curb-to-curb with standard 2-bar steel rail. 2% normal crown."
>
> For a culvert-type structure:
> "16’-4” span by 8’-2” rise aluminum structural plate pipe arch. Flow line of 1% with Elevation 100.00 at the centerline of construction."

Abutments - State the type of abutment and anticipated support system. Also give any specific features required. This section is not required for culvert-type structures. Example:

> "Stub concrete abutments with return wings on steel H-piles, 1.75:1 (plain or heavy) riprap slopes in front" or "Deep concrete abutments with approach slabs on spread footings with sandblasted architectural facing".

Piers - State the type of piers and anticipated support system. This section is not required for culvert-type structures. Example:

> "Mass concrete pier with distribution slab and concrete seal supported by steel H-beam piles."

Opening and Clearance - For water crossings, give the total area of bridge opening and the area of bridge opening at a common elevation for both the existing and the recommended structures. The areas should be normal to the direction of flow. Also, give the minimum clearance depth at Q50 for both the existing and the recommended structures.

For overpass structures, give the minimum vertical and horizontal clearances for both the existing and the recommended structures.

For culvert-type structures, give the total opening for both the existing and the recommended structures.

Disposition of Existing Bridge - Give a brief statement of what is to be done with the existing bridge. Examples:

"To be removed to streambed, property of Contractor."
"Superstructure and abutments to be removed below slope line."
“Steel beams to be retained by the Department."
“Existing wearing surface, rail, and curbs to be removed.”

Available Soils Information - State what soils information was available during study or was obtained from existing plans. Also indicate if scour analysis should be made in the final design of the foundation.

Additional Design Features - Describe any design features that are not described in any other part of the Recommendation Form (e.g. something that is unusual or experimental), but which are necessary to complete the project description.

Maintenance of Traffic - State how and where traffic is to be maintained during construction of the project, whether one lane or two lanes will be required, and whether signals or flaggers will be required. Also state if maintenance of pedestrian traffic is required. If a road closure is proposed, give the detour length from abutment to abutment.

Construction Schedule - Include any restrictions and/or commitments. Example:

“One construction season with landscaping the following spring. Bridge must be reopened to traffic by Labor Day.”

Dates - For projects funded through construction, enter advertise, construction begin, and construction complete dates. For PCE-P projects funded through design, give the “Plans to R/W” date. For PCE-C projects funded through public meeting give environmental document date.

Program Funding Level – Enter either “Construction” or PCE level

Approximate Cost - Enter the programmed, approved, and the estimated project costs under the appropriate headings.

> *Commentary: The estimated cost of the project is located in 4 places within the PDR: the program funding table, summary of preliminary design, preliminary plan, and the cost estimate.*

Project Fiscally Approved – Signature of Assistant Program Manager is obtained here prior to proceeding with any further work.

Utilities - List the known utilities in the project limits. The utility list may be obtained from the Utility Coordinator or the utility data base.

Additional Soils Information and Additional Field Survey - Indicate whether or not the information is required.

Exception to Standards - List any exceptions to Federal or State Standards that either requires approval from FHWA (for NHS projects only), the Engineer of Design, or the Bridge Program management team via the Coachpoint process. Examples of exceptions to standards are reduced bridge widths, omitting of the leveling slab on butted precast superstructures, and reduced hydraulic clearances.

Comments - This is for comments by the Engineer of Design.

2.1.6 Summary of Expected Impacts

This form provides a summary of the expected impacts and the required permitting for the recommended project. These impacts may be right-of-way, historical, archeological, environmental, etc. The required permitting may include Coast Guard, FAA, and the various environmental permits. Filling in the required information for this form will be a project team effort.

2.1.7 Summary of Preliminary Design

This is a summary of the Preliminary Design performed to determine the project recommendations. It should describe, in an orderly fashion, the alternatives considered, with a summary of the assumptions and comparisons that are pertinent to the justification of the recommendation. It should include a discussion of bridge width, alignment, and maintenance of traffic, with the reasoning used to arrive at the recommendation. It may include a discussion of geotechnical, environmental, or utility issues, if these are pertinent to the project.

The Summary should discuss the pros and cons of the alternatives considered and the reasons for the selection of the recommended alternative. Only the engineering that is pertinent should be discussed. The Summary should be short and to the point and should avoid superfluous and lengthy discussions.

For a water-crossing structure, reference should be made to the Hydrology/Hydraulic/Scour Report with the conclusions repeated as to the feasible structure alternatives and ultimate recommendation.

In some instances, especially on large and expensive projects, there may be several alternatives developed for public or internal review and selection. These alternatives should be summarized here, with the back-up data and calculations bound and filed elsewhere in the project file.

2.1.8 Existing Bridge Synopsis Form

This form provides a description of the physical characteristics, history, and condition of the existing structure and should be filled in as completely as possible from information in Bridge Maintenance files and project records.

2.1.9 Hydrology/Hydraulic/Scour Report

This is a summary of the hydrologic analysis that determines the design and check discharges, the hydraulic analysis that determines the structure opening and/or structure alternatives, and the scour analysis that determines the foundation requirements. Normally, this report combines the Hydrology and Hydraulics, but it can be separated into two reports if warranted. The MaineDOT Environmental Office Hydrology Unit provides a spreadsheet with the results of the U.S.G.S. full regression equation. Flows based on other methods should be computed and documented by the Designer. These flows are summarized in this section. Example:

Drainage Area	110 sq mi
Design Discharge (Q50)	1240 cfs
Check Discharge (Q100)	1410 cfs
Scour Check Discharge (Q500)	1660 cfs
Ordinary High Water (Q1.1)	380 cfs
Flood of Record (Q---)	1820 cfs @ Elevation 64.3

If HEC-RAS runs will be necessary for the hydraulic study, stream slopes should be determined. If the structure is in a tidal zone, the following elevation data should also be summarized:

Mean Lower Low Water (MLLW)	-8.5 ft
Mean Low Water (MLW)	-8.2 ft
Mean Tide Level (MTL)	-0.3 ft
Mean High Water (MHW)	7.5 ft
Mean Higher Water (MHHW)	9.4 ft
2003 Predicted High Tide	10.7 ft

The hydraulic analysis is then discussed. Structural openings should be analyzed for flow capacity, outlet velocities, and backwater heights, using Bureau of Public Roads (BPR) charts and graphs, backwater runs, or other applicable methods. Culvert-type structures should be checked for fish passage at low flow conditions.

If no single structure alternative is obvious, the Hydrology/Hydraulic/Scour Report should describe those alternatives that are hydraulically feasible, and the final recommended alternative should be discussed in the Summary of Preliminary Design of the Bridge Recommendation Form.

A summary gives the final conclusions and hydraulic parameters. Also, for comparative purposes, the Summary should give the hydraulic parameters of the existing bridge. Example:

	Existing Bridge 60 ft clear span	**Recommended 88 ft clear span**
Headwater El. @ Q50	104 ft	101 ft
Headwater El. @ Q100	107 ft	102 ft
Discharge Velocity @ Q50	9.1 fps	5.2 fps
Discharge Velocity @ Q100	12.6 fps	6.5 fps
Ordinary High Water (Q1.1)	98.1 ft	98.1 ft
Discharge Velocity @ Q1.1	3.5 fps	2.0 fps
Clearance @ Q50	1.3 ft	4.2 ft

2.1.10 Preliminary Plan

A half-size copy of the Preliminary Plan will be added to the PDR after its preparation and it should be included in the Table of Contents. Typical sections of existing and proposed bridges should be shown on the Preliminary Plan, as well as proposed construction and other pertinent data.

2.1.11 Photographs

A good selection of color photographs of the bridge, roadway, and stream should be taken during a field inspection visit or from photographs taken by others. Photographs may also be copied from the Bridge Maintenance files or obtained from local residents taken during a flood or during the construction of the existing bridge. When possible, the date the photographs were taken should be noted.

2.1.12 Summary of Existing Upstream and Downstream Bridges

Information about the upstream and downstream bridges may be useful for the hydraulic analysis. If so, they are listed here along with the size of the hydraulic opening and pertinent ice, flooding, and debris concerns.

2.1.13 Site Inspection Report

All field trips to the project site should be documented, describing all pertinent findings, conclusions, and points of interest.

2.1.14 Information Reports

Reports from Bridge Maintenance Supervisors, local residents, or Town Officials pertaining to structural condition or hydraulics should be documented. A copy of the most recent inspection report should also be included here.

2.1.15 Survey Plans of Existing Bridges

Archived survey or general plans of the existing bridge should be printed and included here. Plans of nearby bridges may also be included if they have pertinent information related to flood history, soils, or topography which could be used in the preliminary design. Pertinent structural plans may also be included for complex rehabilitation projects when deemed beneficial.

2.1.16 Hydrology/Hydraulic/Scour Data

This section provides the back-up data to the Hydrology/Hydraulic/Scour Report, such as the flow data tabulation, aerial photographs, analysis of existing bridges, FEMA data, BPR hydraulic graphs and charts, HY-8 results, HEC-RAS results, scour computations, and other relevant information. If the project has extensive computer reports from the hydraulic analysis, include the most pertinent information in the PDR. Additional hydrology/hydraulic/scour data should be compiled in a separate document, placed in the project file, and referenced in the PDR.

2.1.17 Miscellaneous Information

Any other pertinent information that is developed or obtained can be included here.

2.1.18 Traffic and Accident Data

The traffic data information obtained from the Bureau of Planning is included here. Include accident data if pertinent to the project.

2.1.19 Estimates

Preliminary Cost Estimate forms are available electronically to assist in estimate preparations. They should be included here for all developed alternates. Supporting spreadsheets that estimate costs using detailed pay items should not be included in the PDR; however, they can be placed in the project file. As a check on the accuracy of the estimate, the square foot cost obtained should be compared to historical square foot cost data found in the Bridge Program's Bridge Unit Cost database. All project costs should be rounded as shown in Table 2-1.

Table 2-1 Rounding Guidelines for PDR Cost Estimates

Item	Amount	Round To Nearest:
Individual construction items such as Superstructure, Cofferdams, Approaches, Mobilization, etc.	All	$1,000
Structure Subtotal and Approaches Subtotal	All	$5,000
Total Construction Cost, PE, ROW, CE	Up to $1,000,000	$5,000
	Over $1,000,000	$10,000
Total Project Cost	Up to $500,000	$5,000
	$500,000 to $1,000,000	$10,000
	Over $1,000,000	$100,000

2.2 Economic Comparisons

2.2.1 Overview

During preliminary design, the Designer should consider different rehabilitation/replacement alternatives. A Life Cycle Cost Analysis (LCCA) is a tool used to select alternatives and to make economic decisions. Sound engineering judgment is necessary to determine input data, analyze results, and determine the relevance of the analysis.

LCCA considerations for bridges include functionality, age, condition, present costs, future costs, and present and future program funding availability. The two approaches available to evaluate LCCA are a Deterministic Analysis and Probabilistic Analysis. This section will examine both analyses.

2.2.2 Definition of LCCA

Section 303 of the National Highway System Designation Act defines LCCA as "a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment".

In short, LCCA is a method of analysis that compares the net present value of all costs related to improvements over the life of the structure. The level of detail of the analysis is determined on a project-by-project basis.

2.2.3 When to use LCCA

LCCA should be performed when comparing competing options with different life expectancies, rehabilitation costs, or maintenance costs. Common situations are listed below:

- A rehabilitation scenario for a single bridge with multiple choices such as: 1) immediate deck replacement; 2) wearing surface replacement followed in 15 years by a deck replacement; 3) deck rehabilitation and wearing surface replacement followed by a superstructure replacement in 15 years; etc. (refer to Chapter 10 Rehabilitation for a discussion of this terminology)
- Comparing a traditional bridge that has significant maintenance costs to a buried structure that has few maintenance costs
- Bridge rehabilitation compared with replacement
- Painting a bridge or waiting until the bridge is deficient and then replacing it
- Comparing steel bridge that requires painting with a concrete structure that is to be located in a harsh environment where weathering steel is not recommended
- Comparing a steel pipe to an aluminum pipe or concrete box

2.2.4 Deterministic Analysis

A deterministic analysis is the most common method, and is adequate to evaluate LCCA in most situations. This approach compares alternatives and life cycle costs based on net present value and fixed inputs. This simplified approach will provide one solution for any given set of alternatives. To vary costs or timing, inputs need to be changed and the analysis rerun. For most projects the inputs can be easily adjusted utilizing a spreadsheet. Design examples are available in Excel from the technical resource people for economic comparisons.

2.2.5 Probabilistic Analysis

The next level of LCCA is a probabilistic analysis. This approach allows for variability and uncertainty of timing and costs. The output provides a probability of which alternate will have the lowest costs over the life of the bridge. This method of analysis is recommended for projects with significant bridge replacement or rehabilitation costs, or when the deterministic approach is insufficient.

The Bridge Program utilizes a program developed by NCHRP Project 12-43. Bridge Life Cycle Cost Analysis (BLCCA) has the ability to perform both a probabilistic and a deterministic analysis. BLCCA can be installed on the Designer's PC as needed. A complete Guidance Manual and User's Manual is also available for reference that can be viewed and printed through the help menus.

2.2.6 Standard Assumptions

To ensure consistency the following assumptions are recommended:

- Use a discount rate of 4%, which approximates the FHWA discount rate. This factor accounts for the annual growth rate of an investment, and does not include inflation.

- Use current and constant dollars. For example, if the cost for a repair in year 1 is $100,000, the same repair in year 10 will also cost $100,000.

- Routine maintenance costs are assumed to be the same for all alternates and are ignored in the analysis, except when comparing different structure types such as a buried structure to a traditional bridge. These costs include such activities as minor wearing surface and concrete repairs, yearly cleaning of bearings and drains, and repair of damaged railings.

- o User costs are assumed to be the same for all alternates and are ignored in the analysis, unless one alternate has a significant impact on the public over another alternate. User costs can be requested from Planning, if they are used in the analysis.

- o Suggested rehabilitation intervals over the life of the bridge are shown in Table 2-2. These may be used as a guide in developing the future rehabilitation over the life of an existing or proposed bridge.

- o The Designer should not rely solely on LCCA. The results from LCCA always show deferring costs as the most cost effective solution. However, it is important to consider the additional costs to maintain an old bridge, the impact to the traveling public as a result of additional maintenance work, risks associated with a deteriorating structure, and availability of funding when replacement becomes absolutely necessary. The functionality of the bridge is also important. Replacing a bridge to modern standards may provide an increased bridge width, new sidewalks, or an improved alignment.

Table 2-2 Life Cycle Intervals

Capital Investment	Useful Life of Component (years)
Wearing Surface Replace/Rehab	15
Deck Rehabilitation (includes wearing surface)	30
Deck Replacement	50
Bridge Replacement	75
Painting	Refer to Section 7.2.3 Coatings
Sliplining	Depends on materials used and site conditions
Invert Lining	25+
Steel Pipe	50
Plastic Pipe	100
Aluminum Pipe	75
Concrete Pipe/Box	75-100

Notes:

1. Condition of the membrane will determine whether a wearing surface replacement will last 15 years.

2. Extreme traffic or environmental conditions will decrease the useful life of traditional bridges.

3. The substructure can at times outlast the superstructure. The useful life of the substructure should be considered before selecting a rehabilitation alternative.

4. The U.S Army Corps of Engineers document (1997) gives a design life of 50 years for aluminum and plastic pipes. There is evidence that these materials will last much longer.

5. The life of the concrete invert lining is dependent on the longevity of the top plates.

6. The useful life of pipes can vary significantly. Considerations include the cover over the pipe, soil pH and resistively, presence of salts or other corrosive compounds, plate thickness, and flow velocity.

2.2.7 Cost Comparison for Number of Beams

The following discussion is a guide to compare the cost of reducing the number of beams on steel bridges with full cast in place decks only. Future updates to this procedure will include the use of precast deck panels and the use of precast, prestressed beams. Other issues besides cost must be considered as well when determining the optimal number of beams, such as maintenance of traffic during construction and future maintenance needs (refer to Section 7.3 Economy and Section 2.9.6 Maintainability).

For steel beam bridges with relatively wide decks, the Structural Designer may need to investigate the optimum number of beams to use. Fewer beams will result in less total steel required, but will require more deck concrete, and will have slightly higher fabrication costs per pound of steel. A discussion of the cost comparison method is found here.

Regardless of the number or size of the beams, the raw price of steel supplied from the mill can be considered a constant. For this discussion, we assume a cost of $0.50/lb. The cost of fabricating, delivering, erecting, and finishing each beam is relatively independent of the weight of the beam, though will be slightly higher for heavier beams due to issues such as additional welding lengths for deeper webs, larger beam surface area that will require more painting, and thicker plates that will require more effort to drill holes. Therefore, one can assume that this cost for the heavier beam will be approximately 10% higher. If significantly more stiffeners will be required for the heavier beam, this number might be even higher. The ratio of costs will then be the number of beams with narrower beam spacing to the adjusted ratio of the number of beams with wide beam spacing.

Wider beam spacing will also require thicker slabs. When slab thicknesses increase appreciably, the support form costs will increase because of the extra strength required to carry the extra thickness. However, the added support forms cost will be offset by a decrease in labor cost with fewer beams on which blocking must be formed, and also fewer bays in which support forms must be suspended. Therefore, the cost of forming and finishing is assumed to be equal regardless of beam spacing. The price of concrete delivered and placed can be assumed to be equal to about 35% of the unit price of deck concrete. Generally no cost adjustment is made for reinforcing steel since thicker slabs will have little change in reinforcing steel quantity

The following example illustrates this method of cost comparison.

Example 2-1 Cost Comparison of Number of Steel Beams

Assume a price comparison of four beams to five beams, with a bid price of $1.00/lb for five welded beams, and assuming equal stiffeners on all beams. Weight of steel for 5 beams is 30,000 lb.

ratio of beams = 4/5 = 0.80
ratio of diaphragms = 3/4 = 0.75
assume a cost ratio on fabricating, delivery, and erecting of 0.79, a number chosen between 0.80 and 0.75, but weighted more toward the beam ratio than the diaphragm ratio

5 beams:	mill	$0.50/lb x 30,000	= $15,000
	fab/del/erect	$0.50/lb x 30,000	= $15,000
			$30,000
4 beams:	mill	$0.50/lb x 30,000	= $15,000
	fab/del/erect	$0.50/lb x 0.79 x 1.1 x 30,000	= $13,000
			$28,000

Assume a bid price of $450/ yd^3 of deck concrete. Assume a five beam bridge will require an 8 inch slab and a four beam bridge will require a 10 inch slab, with quantities of concrete being 150 yd^3 and 200 yd^3 respectively. The slab costs would be:

8 inch deck:	forming & finishing	$290 x 150 yd^3 =	$43,500
	delivery & placing	$160 x 150 yd^3 =	$24,000
			$67,500
10 inch deck:	forming & finishing	$290 x 200 yd^3 =	$58,000
	delivery & placing	$160 x 200 yd^3 =	$32,000
			$90,000

Summary:	5 beams:	$30,000 + $67,500	= $97,500
	4 beams:	$28,000 + 90,000	= $118,000

2.3 Hydrology, Hydraulics, and Scour

2.3.1 General

Most of Maine's bridges are located over water. Bridge drainage structures will range from large culvert-type structures to multi-million dollar bridges. Although some hydrologic, hydraulic, and scour analysis is necessary for all bridge drainage structures, the extent of such studies should be commensurate with the complexity of the situation, and with the importance of the structure and of the surrounding property.

Minor spans, bridges, and extraordinary bridges are the responsibility of the Bridge Program.

2.3.2 Minor Span/Strut Determination

Designers must determine on a project-by-project basis if a drainage structure is a strut or minor span. A strut is a structure with a span equal to or greater than 5 feet and less than 10 feet. If a structure has a span equal to or greater than 10 feet, or if multiple structures have a combined opening of at least 80 square feet in area, the structure meets the minimum requirements for a minor span. For a minor span or a bridge, the drainage area is typically 2 square miles or larger with a Q50 flow of 500 cfs or larger. The following examples indicate the minimum flow for a pipe, a pipe arch, and a concrete box that meet the definition of a minor span:

- 10'-3" span by 6'-9" rise steel structural plate pipe arch (18" corner radius) that is 72' long at 0.5% slope with the end mitered to match the slope (inlet control). HW/D is 0.9 or 90% with approximately 325 cfs.

- 10' diameter steel pipe that is 72' long at 0.5% slope with the end mitered to match the slope (inlet control). HW/D is 0.9 or 90% with approximately 525 cfs.

- 10' span by 10' rise concrete box culvert that is 72' long at 0.5% slope with square edge headwall and 0° wingwalls (inlet control). HW/D is 0.9 or 90% with approximately 700 cfs.

Table 2-3 can be used for guidance to determine if a structure is a strut or a minor span based upon an approximate flow.

Table 2-3 Design Flow versus Drainage Area and Wetland Percent

Drainage Area (square miles)	Wetland %	Q50 (cfs)
2	1	549
2	5	409
2	10	287
2	14	211
3	1	753
3	5	563
3	10	388
3	15	269
3	18	215

Note: Flows are based on the U.S.G.S. full regression equation. These values are provided for general guidance and should not be used for hydraulic design purposes.

2.3.3 Level of Analysis

2.3.3.1 Level 1 (Qualitative Analysis)

A Level 1 qualitative analysis involves no numerical analysis. It is used for a project when a pipe or pipe arch is being replaced by another pipe in the same location and when the project meets the following criteria:

- No signs of scour or erosion problems
- No reports of flooding problems
- Relatively stable stream (vertically and laterally)
- No history of significant ice jams or debris problems
- No buildings or homes close to the stream
- No reduction in the opening size
- Fish passage is maintained or is not an issue
- Adequate alignment (horizontal and vertical)
- No history of accidents at the bridge location

If the project team decides to use a Level 1 analysis, all the existing records should be reviewed and a site inspection conducted. The site inspection should involve the entire project team. Municipal officials, bridge maintenance, and abutting landowners should be queried for personal knowledge of flooding activities and all hydraulic and flood information should be documented in the PDR.

2.3.3.2 Level 2 (Basic Analysis)

Most bridge projects fall into the Level 2 basic analysis category. In addition to the qualitative analysis done for Level 1, a numerical analysis is performed for Level 2. Flows are computed, and hydraulics and scour are analyzed for all of the feasible alternatives.

2.3.3.3 Level 3 (Complex Analysis)

Projects that fall into the Level 3 complex analysis category typically have the following concerns:

- Difficulties determining flows (i.e. islands, divided flow, multiple streams merging)
- Uncertainty about the flow angle of attack
- Unstable streams/rivers
- Highly constricted flow with scour problems
- Tidal areas with long bridges
- Project where the opening size may be reduced drastically

Analysis for complex projects may involve a two-dimensional analysis using a program like FESWMS. If there is any uncertainty about what level of analysis applies, the Designer should contact the Bridge Program’s hydraulics technical resource people.

2.3.4 Data/Information Collection

The Designer should compile all pertinent information as described below, prior to visiting the site, and before beginning the actual hydrologic analysis for the project. The gathering of such data can simplify the hydrologic analysis and provide the background for good judgment decisions, which may be required.

- *Topographic survey* - The survey for the project site will be performed by MaineDOT’s survey crews or by consultant survey crews as determined by the Survey Coordinator. The plotted survey provides information about the stream's channel and flood plain necessary for the analysis of the structure site. The surveyor's notes and descriptions of the stream and of the existing bridge may provide valuable information on flood history and for a hydraulic analysis of the site.

- *Site inspection* - A visit to the project site and to upstream and downstream bridge sites can supply valuable information, such as high water marks on the existing structures or ice markings on trees. Photographs can be taken for reference and to help recall the site conditions. A site inspection can give team the proper perspective of the site conditions, which the survey plan or photographs cannot. If possible, the site inspection should take place after all office records have been gathered.

- *Inspection reports* - MaineDOT's bridges are inspected at least every two years. Bridges requiring underwater inspections are inspected by divers every five years. These inspection reports should be reviewed for all projects. The underwater inspection report in particular is an excellent source of information about scour problems.

- *Maintenance reports* - Maintenance reports of work performed on the bridge can provide information on debris, scour, or ice problems that may have occurred. Indications of scour or other problems requiring maintenance work could indicate an undersized structure.

- *Plans of existing bridges* - The plans of existing bridges at the subject site, as well as at upstream and downstream locations, can give valuable information on flood histories, stream information, and the necessary data for the hydraulic analysis of the structures.

- *Witnessed observations* - Narrative descriptions of past flood and normal flows may be obtained from Bridge Maintenance Supervisors, Highway Maintenance Supervisors, municipal officials, newspaper accounts, or local residents. Information pertaining to high water elevations at existing bridge sites along with the dates of the occurrences, ice or debris problems, structure adequacy, and other information obtained should be documented.

- *Aerial photographs* - Aerial photographs can be a helpful tool in evaluating the stream and its flood plain. The Photogrammetry and Control Unit maintains all aerial photograph coverage, of which prints or electronic copies can be made. They may also have aerial photograph contour plans for major highway projects that can also be useful.

- *Photographs* - Photographs of past flood occurrences can sometimes be obtained from local residents, Bridge Maintenance Supervisors, or in the Bridge Maintenance's photograph files.

- *Stream data from other agencies* - Stream flow and flood related data are sometimes available from other agencies in the State. The major sources are:

  U.S. Geological Survey: The U.S.G.S. has numerous gage stations on rivers and streams that collect hydrologic information. Through the use of formulae, this information can be transformed to other locations on the same water course. The Bridge Program's Hydraulic Library has copies of U.S.G.S. annual reports and a computer analysis summary of each gage site, which can be used to determine the existence of a gage location. If more information is required than can be obtained from these sources, the U.S.G.S. office in Augusta should be contacted.

  Natural Resources Conservation Service (NCRS): The NRCS, formerly known as the Soil Conservation Service (SCS), has studies for many flood control projects that contain information on the hydrology and hydraulics of the involved stream. The Hydraulic Library has a location map indicating completed and planned studies. The NRCS office in Bangor should be contacted for detailed information for each site for which information is desired.

  Utilities: Various utility companies have control of many dams in the State, and for most of the larger dams, they maintain flow records and capacity data. The Hydraulic Library has a listing of all known dams in the State with a brief description of the dam and the name of the dam owner.

- *Hydraulic Library -* The Bridge Program's Hydraulic Library has copies of many different Flood Study Reports, such as Corps of Engineer Studies, HUD Flood Insurance Studies, SCS Watershed Studies, and other miscellaneous information pertaining to specific rivers and streams. The Preliminary Engineering Studies and PDRs that have been developed for MaineDOT bridge structures over the years are electronically filed in MaineDOT's TEDOCS document management system. PDRs with hydrology and hydraulic information are generally available for projects starting in about the year 1975.

- *Local newspapers -* Local newspaper files may have stories on previous floods.

- *Flood insurance studies -* River cross sections used to develop Flood Insurance Rate Maps (FIRM) can be obtained through the Maine Floodplain Management Program in the Department of Economic and

Community Development. These cross sections can be used in a hydraulic model such as HEC-RAS.

All of the above sources of information may provide valuable assistance and supplementary information that can be used advantageously; however, discrepancies sometimes are revealed when these data are compared. This indicates the need for verification and proper evaluation of the flood data, regardless of the source.

2.3.5 Vertical Datum

Since January 2000, all new projects, with a few exceptions, are referenced to the North American Vertical Datum (NAVD) of 1988.

> *Commentary: If there is any doubt about which vertical datum was used for a project, please contact the Survey Coordinator.*

Many of MaineDOT's existing plans, existing flood studies, historical flood information, and U.S.G.S. topographic maps are based on the National Geodetic Vertical Datum (NGVD) of 1929. The elevations based on this older datum must be converted to the newer NAVD of 1988. The elevations are adjusted using the following equation:

Elevation xxx.xxx (NGVD 1929) - datum shift = Elevation xxx.xxx (NAVD 1988)

The datum shift ranges between 0.591 feet and 0.722 feet. The exact datum shift for a specific location in Maine can be found at the following website:

http://www.ngs.noaa.gov/cgi-bin/VERTCON/vert_con.prl

The following data must be entered on the web page:

- North Latitude (required)
- West Longitude (required)
- Orthometric Height (optional)

Latitude and Longitude may be entered in any of the following three formats, including blank spaces:

Degrees, minutes, and decimal seconds (xxx xx xx.xxx)
Degrees and decimal minutes (xxx xx.xxx)
Decimal degrees (xxx.xxxxx)

The following example illustrates how to apply the datum shift:

Example 2-2 Datum Shift

This information comes from the Gouldville Bridge in Presque Isle.
Q100 Elevation = 431' from Flood Insurance Study based on (NGVD 1929).

Step 1: Go to website and get datum shift by entering latitude and longitude for the location you are interested in.

Latitude = 46.667^0
Longitude = 68.00^0
Datum shift = 0.627'

Step 2: Subtract datum shift (i.e. correction factor) from elevation based on NGVD 1929 to convert to NAVD 1988.

(NGVD 1929) - (correction) = (NAVD 1988)

431' - 0.627' = 430.373'

Hydrology, hydraulics, and scour reports should state which vertical datum is used. For example, the following statement can be added at the end of any report:

> Note: All elevations based on North American Vertical Datum (NAVD) 1988. Elevations based on the National Geodetic Vertical Datum (NGVD) 1929 were converted to NAVD by the appropriate shift (0.627') using the NGS Vertcon program.

2.3.6 Tidal Elevation Computations

Full daily tide predictions are limited to a small number of reference stations. Maine has only three reference stations in Eastport, Bar Harbor, and Portland. Tide predictions at other locations are referred to as "subordinate stations", can be obtained by applying specific differences to the daily tide predictions for one of the reference stations. The application of time differences and height ratios will generally provide reasonably accurate approximations at subordinate stations, however, they cannot result in predictions as accurate as those listed for the reference stations.

The National Oceanic and Atmospheric Administration, National Ocean Service (NOS) is in the process of updating the nation's tidal datums to a new National Tidal Datum Epoch (NTDE) from 1983 to 2001 to reflect changes in mean sea level along the nation's coast. The new NTDE will provide up-to-date tidal datum information. Whenever possible, data from the 1983-2001 NTDE should be used when computing elevations. The NTDE is a specific 19-year period over which tide observations are taken to determine Mean Sea Level and other tidal datums such as Mean Lower Low Water and Mean High Water. This latest update will define the 19-year period as 1983-2001. The 19-year period includes an 18.6 year astronomical cycle that accounts for all significant variations in the moon and sun that cause slowly varying changes in the range of tide. The following examples show how to determine tidal elevations at a reference station and at a subordinate station.

Example 2-3 Tidal Elevation at Reference Station

Determine the following elevations for the Eastport, Maine reference station:

Highest Observed Water Level
Mean Lower Low Water (MLLW)
Mean Low Water (MLW)
Mean Tide Level (MTL)
Mean High Water (MHW)
Mean Higher High Water (MHHW)
Lowest Observed Water Level
Predicted High Tide Elevation for 2003

Step 1: Obtain the tidal datum information from the tidal gage site using the following website for the NTDE (1983 -2001).

http://co-ops.nos.noaa.gov/bench_mark.shtml?region=me

The webpage will have a list of possible sites on the left side of the screen. Click on the Eastport location. About two thirds of the way down the web page for Eastport, you will find the tidal datums section for the particular site. For example, the tidal datums section will look like the following for 8410140 EASTPORT, PASSAMAQUODDY BAY:

TIDAL DATUMS

Tidal datums at EASTPORT, PASSAMAQUODDY BAY based on:

LENGTH OF SERIES:	19 Years
TIME PERIOD:	January 1983-December 2001
TIDAL EPOCH:	1983-2001
CONTROL TIDE STATION:	

Elevations of tidal datums refer to Mean Lower Low Water (MLLW), in METERS:

HIGHEST OBSERVED WATER LEVEL (01/10/1997)	= 7.383
MEAN HIGHER HIGH WATER (MHHW)	= 5.844
MEAN HIGH WATER (MHW)	= 5.729
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	= 3.029
MEAN SEA LEVEL (MSL)	= 2.958
MEAN TIDE LEVEL (MTL)	= 2.932
MEAN LOW WATER (MLW)	= 0.136
MEAN LOWER LOW WATER (MLLW)	= 0.000
LOWEST OBSERVED WATER LEVEL (08/09/1972)	= -1.426

Step 2: Convert the tidal datum information to the correct vertical datum. The tide information needs to be converted to the NAVD. MaineDOT has been surveying using the NAVD since about the year 2000.

Highest Observed Water Level (01/10/1997):
7.383 m – 3.029 m = 4.354 m

MHHW: 5.844 m -3.029 m = 2.815 m

MHW:	5.729 m – 3.029 m = 2.700 m
NAVD -1988:	3.029 m -3.029 m = 0.000 m
MSL:	2.958 m - 3.029 m = -0.071 m
MTL:	2.932 m - 3.029 m = -0.097 m
MLW:	0.136 m – 3.029 m = -2.893 m
MLLW:	0.000 m – 3.029 m = -3.029 m

Lowest Observed Water Level (08/09/1972):
-1.426 m - 3.029 m = -4.455 m

Step 3: Convert elevations from meters to feet. Tidal datum information based on the NTDE from 1983 -2001 is in meters.

Highest Observed Water Level (01/10/1997)
4.354 m x 3.2808 ft/m = 14.285 ft

MHHW:	2.815 m x 3.2808 ft/m = 9.236 ft
MHW:	2.700 m x 3.2808 ft/m = 8.858 ft
NAVD -1988:	0.000 m x 3.2808 ft/m = 0.000 ft
MSL:	-0.071 m x 3.2808 ft/m = -0.233 ft
MTL:	-0.097 m x 3.2808 ft/m = -0.318 ft
MLW:	-2.893 m x 3.2808 ft/m = -9.491 ft
MLLW:	-3.029 m x 3.2808 ft/m = -9.938 ft

Lowest Observed Water Level (08/09/1972):
-4.455 m x 3.2808 ft/m = - 14.616 ft

Step 4: Determine the highest predicted tide for the current year.

Go to the following web site:

http://tidesandcurrents.noaa.gov/tides03/tab2ec1a.html#7

Click on the Eastport site. Review the data for the entire year and find the date with largest height.

April 19, 2003 12:09 am 22.3 ft (datum is MLLW)

2003 predicted high tide = - 9.938 ft (MLLW) + 22.3 ft = 12.362 ft

Example 2-4 Tidal Elevation at Subordinate Station

Determine the following elevations at West Quoddy Head using Eastport as the reference station.

MLLW
MLW
MTL
MHW
MHHW
Predicted High Tide Elevation for 2003

Step 1 through **Step 4**: See Example 2-3 for the Eastport location.

Step 5: Obtain the values for the mean range, spring range, and MTL for the West Quoddy Head location (subordinate station) from the following website:

http://tidesandcurrents.noaa.gov/tides03/tab2ec1a.html#7

West Quoddy Head
Mean range = 15.7 ft
Spring range = 17.9 ft
MTL = 8.2 ft

Step 6: Compute tide levels at West Quoddy Head

MTL Eastport = MTL West Quoddy Head

MHW West Quoddy Head = MTL Eastport + Mean Range @ West Quoddy Head/2 -0.318 ft + 15.7 ft/2 = 7.5 ft

MLW West Quoddy Head = MTL Eastport - Mean Range @ West Quoddy Head/2 -0.318 ft - 15.7ft/2 = -8.2 ft

MLLW West Quoddy Head = MTL Eastport - Mean Tide Level @ West Quoddy Head -0.318 ft - 8.2ft = -8.5 ft

MHHW West Quoddy Head = MLLW @ West Quoddy Head + Spring Range @ West Quoddy Head -8.5 ft + 17.9 ft = 9.4 ft

Step 7: Determine the highest predicted tide for the current year at West Quoddy Head.

Go to the following web site:

http://tidesandcurrents.noaa.gov/tides03/tab2ec1a.html#7

Click on the Eastport site, which is the closest reference station. Review the data for the entire year and find the date with largest height.

April 19, 2003 12:09 am 22.3 ft (datum is MLLW)

Get the following reference from the Hydraulics Library:

Tide Tables 2003, High and Low Water Predictions, East Coast of North and South America including Greenland

In Table 2 of the Tide Tables book under West Quoddy Head, find the ratio of height differences at high water.

West Quoddy Head   Ratio = 0.86

0.86 x 22.3 ft = 19.17 ft (datum is MLLW)

2003 predicted high tide = -8.5 ft (MLLW) + 19.17 ft = 10.7 ft

2.3.7 Changes in Sea Level

The level of the sea along the coast of Maine is rising between 0.5 feet and 0.75 feet per 100 years. Bridges along the coast of Maine should take this rise in sea level into consideration when designing bridge projects in tidal areas. Refer to the following website for more information.

http://www.co-ops.nos.noaa.gov/sltrends/sltrends_states.shtml?region=me

2.3.8 Documentation

The PDR includes a hydrology, hydraulics, and scour report and backup information. Backup information should include, but is not limited to, the following: computer printouts (input and output), drainage area map, hydrology computations, hydraulic computations, scour computations, and eyewitness reports about flooding.

The PDR is the main source of hydrologic, hydraulic, and scour information for a bridge project. If there are any changes made to the project after the PDR has been completed that impacts hydrology, hydraulics, and/or scour, it should be documented and included in the PDR as an addendum.

It is often helpful and sometimes necessary to refer to plans, hydrology, hydraulic, and scour analyses long after the actual construction is completed. They can be useful in the analysis of an upstream or downstream structure, in the future replacement of the structure, or in the evaluation of the hydraulic performance of the structure after large floods. Documentation provides a quick reference and a construction aid for the Contractor and the Resident in the construction of a bridge structure. This information is also helpful to other state agencies such as Floodplain Management, as a source of best available data for Q100 elevation when a formal flood study has not been done for a river.

2.3.9 Hydrology

2.3.9.1 Introduction

Hydrologic analysis is a very important step prior to the hydraulic design of a bridge drainage structure. Such an analysis is necessary for determining the flow that the structure will be required to accommodate. The flow, or discharge, is a hydraulic "load" on the structure and the determination of its magnitude is as important as the determination of proper structural loads. These guidelines give a recommended approach to the hydrologic analysis of bridge drainage structures. The guidelines are not all-inclusive, nor are they intended to require strict compliance, but they are presented as a guide. Hydrology is not an exact science, and it requires the use of good engineering judgment to evaluate the available information and arrive at logical and suitable conclusions.

2.3.9.2 Discharge Rate Policy

The following discharge rates need to be computed for the hydraulic design of bridges and minor spans:

- Q1.1 - spring flood discharge
- Q50 - design discharge
- Q100 or flood of record - check discharge

Other discharge rates may need to be computed as follows:

- Flows less than Q1.1 - discharges used to check for fish passage in culvert-type structures
- Q10 - discharge used in designing temporary bridges
- Q500 - discharge used in evaluating scour

The determination of the design and check discharges are accomplished through the application of one or more discharge formulae given in this text, combined with the information obtained through information sources and/or through hydraulic analysis of existing structures. Discharge adjustment factors are found in Appendix C Hydrology/Hydraulics.

2.3.9.3 Discharge Formulae

Drainage studies for most projects are requested from the Hydrology Unit in the Environmental Office. The unit provides the Designer with a spreadsheet based upon the U.S.G.S. full regression equations discussed in Appendix C Hydrology/Hydraulics, and Section 2.3.9.4, Rural Watersheds, which follows. Unless gaged data is applicable to the project, dams are present on the section of waterway of interest, or if the U.S.G.S. full regression equation is not applicable, the spreadsheet provided is all that is required for hydrologic analysis. For cases were the spreadsheet provided by the Hydrology Unit is not adequate, refer to the following Sections 2.3.9.4 through 2.3.9.4B.

2.3.9.4 Rural Watersheds

Most watersheds for bridges in Maine are rural in nature. A rural area can generally be defined as one having a high percentage of woods, mixed cover, or fields, and is essentially an undeveloped area with respect to commercial sites and residences. The best source of flow data for rural watersheds is gaged data from the U.S.G.S. gaging station network. Methods for transposing gaged data are including on the following pages. If gaged data is not available, the U.S.G.S. full regression equation can be used. Appendix C contains this equation, as well as a hydrology tabulation form for use with the equation.

A. Urban Watersheds

The U.S.G.S. full regression equation does not apply to urbanized drainage basins or small drainage basins that may experience future development and land use changes. An urban area can generally be defined as one having a very low percentage of woods, mixed cover, or fields, and is essentially a developed area with commercial sites and residences. Potential future development in the watershed should be considered when determining the design flow.

The following methods can be used for small, urbanized drainage basins:

Size of Drainage Area	**Hydrologic Method**
Greater than 3200 acres	NRCS TR-20 or HEC-1 Method
Greater than 20 acres	Sauer and others (1983)

NRCS TR-20 and HEC-1 Methods are explained in the “Urban & Arterial Highway Design Guide.” Sauer and others (1983) is an urban regression equation (Hodgkins, 1999).

B. Hydraulic Analysis

Flows based on observed and recorded high waters at or near bridges may be determined by performing a hydraulic analysis using the methods discussed in 2.3.10.2 Hydraulic Analysis. For culverts, Bodhaine, 1968, can be used.

All of the applicable methods that may be used for the watershed in question should be utilized. However, large variations in answers may result. Knowledge of the limitations and accuracies of each method, coupled with other sources of information and good engineering judgment will be necessary to arrive at a reasonable selection of discharge values.

2.3.10 Hydraulics

2.3.10.1 Introduction

A major aspect in highway design and construction is the crossing of streams and rivers. A concurrent problem is the encroachment of the highway on the flood plain, or even the stream channel. The design of the crossing must be made to insure the safety of the traveler, must protect the river environment, must not create hazards or problems to adjacent landowners and the community, and must be economical. Good engineering judgment combined with knowledge of hydrology and hydraulic sciences, is required to determine the design of river crossings.

Bridges in Maine are designed for both riverine and tidal stream crossings. Riverine bridges are designed for steady flow at the peak discharge for the design storm. Hydraulics design for riverine bridges establishes:

- Minimum finished grades
- Bridge location
- Bridge length
- Span lengths
- Orientation of substructure
- Foundation requirements through scour analysis

Tidal bridges are designed for unsteady flow conditions during the complete rise and fall cycle of the tide. Hydraulic design for tidal bridges establishes the minimum finished grade and minimum depth requirements for the foundation through scour analysis. For special cases, other features may require hydraulic design. For sites further upstream, riverine flow becomes dominant. In some cases both riverine and tidal flow must be analyzed to determine the controlling flow at a bridge.

2.3.10.2 Hydraulic Analysis

The depth or extent of the hydraulic analysis for a bridge structure should be commensurate with the cost and complexity of the project and the problems anticipated.

The main tools for the hydraulic analysis of bridge structures are as indicated below. Additional analysis methods may be used as deemed necessary.

Culvert-type structures:

- Design charts from HDS No. 5, 1985
- HY 8 Culvert design and analysis program by FHWA (Part of Hydrain program)
- Principles of open channel hydraulics
- Other commercially available software programs

Bridges:

- The Army Corp of Engineers program HEC-RAS (preferred program)
- The U.S.G.S. Computer Program "WSPRO"
- Principles of open channel hydraulics

A. Structure Capacity (Riverine)

All bridges and minor spans should be designed for Q50 with the following constraints:

Culvert-type structures - The headwater depth versus structure depth ratio (HW/D) should be approximately equal to or less than 0.9. For twin pipes or pipe arches, the HW/D ratio should be less than 0.9. A minimum of 1 foot of freeboard at the edge of the pavement at Q100 or the flood of record is preferred when outlet conditions control.

Major riverine bridges - A freeboard depth of 4 feet minimum between the bottom of the superstructure and the backwater elevation should be provided on major river crossings. As much as 10 feet of freeboard depth should be provided when practical.

Other riverine bridges - A depth of 2 feet minimum is recommended on smaller streams where there has been no history of ice jams.

If providing the desired freeboard depth results in significant environmental and/or property impacts, a reduced freeboard depth should be investigated with the approval of the Engineer of Design.

All bridge-type structures should also be capable of passing the Q100, or the flood of record, whichever is greater, without any serious harm to the structure, roadway, or adjacent property. This may be accomplished by allowing an overtopping of the approaches if the structure cannot be reasonably sized to accommodate the flow, with the approval of the Engineer of Design. When possible, there should be 1 foot of freeboard at Q100.

Occasionally, freeboard depths may need to be increased for high waters caused by some occurrence other than the design flow, such as for an ice jam, the collapse of a dam, or some future construction that may affect the depth of flowage.

B. Structure Capacity (Tidal)

Culvert-type structures in tidal area - The headwater depth versus structure depth (HW/D) ratio should be approximately equal to or less than 0.9 at Q50 with flow at MHW. The HW/D ratio should be less than 0.9 for twin pipes or pipe arches.

Bridges in tidal area - Bridges on tidal rivers/streams should be designed to protect the bridge structure itself. Most of the surrounding land and the approach roadways may be inundated by relatively frequent tidal storm surges. The minimum design freeboard in these areas is 2 feet above Q10 (based upon MHW) including wave heights. The finished grade of the bridge will be set by considering this requirement, along with navigation clearance, the approach roadways, topography, and good engineering judgment.

There may be instances where a reduction in these requirements will be necessary to minimize high costs, environmental impacts, construction impacts in urban areas, etc. Good engineering judgment should be followed in making these decisions and the reasons should be documented.

C. Analysis Types in Tidal Areas

- *Qualitative analysis:* This method can be used if the criteria in Section 2.3.3 Level of Analysis are met, and if the team has decided to use the simplified approach.

- *Steady flow:* This type of analysis checks at least two or more points in the entire tide cycle. Typically the following cases would be investigated:

  Case 1: Q50 flow with tailwater at mean high water (MHW): This case typically determines the size of the opening and the bottom of beam elevation.

  Case 2: Q50 flow with tailwater at mean low water (MLW): This case typically results in the highest velocities. The velocity is used to design erosion and scour measures.

- *Unsteady flow:* This type of analysis checks the entire tide cycle at 15 minute intervals over a 48 hour period. The typical cases that would be analyzed include the following:

  Case 1: Typical everyday tides with low upland flow (used to verify the model).

  Downstream boundary condition - Typical tide cycle based on mean tide range

  Upstream boundary conditions - Constant Q1.1 flow or a lower more typical flow

  Case 2: High upland flows with no coastal storm.

  Downstream boundary condition - Typical tide cycle based on mean tide range

  Upstream boundary conditions - Constant Q50 flow

  Case 3: Late summer/early fall hurricane with low upland flow.

  Downstream boundary condition - Typical tide cycle based on mean tide range with storm surge due to a Category 1 hurricane. A Category 1 hurricane equates to about a 50 year storm surge. The peak of the storm surge should be checked for the following four different times:

  1. Peak of storm surge at mid rising tide
  2. Peak of storm surge at high tide
  3. Peak of storm surge at mid falling tide
  4. Peak of storm surge at low tide

Upstream boundary condition - Constant Q1.1 flow or a lower more typical flow.

An unsteady flow analysis in a tidal area requires cross sections (for 1-D analysis using HEC-RAS) and/or a digital terrain model (DTM, for 2-D analysis) that covers at least 90% of the area within the drainage basin affected by the tides. Getting the survey information to create the hydraulic model for an unsteady flow model is difficult and expensive.

2.3.10.3 Discharge Velocities

The velocity at the outlet or downstream side of a bridge structure can be a controlling feature of the structure opening. The scour susceptibility of the stream and scour protection measures should be a major consideration in the sizing of a bridge. The velocity through the existing bridge and the scour conditions should be evaluated. If the present conditions do not show any cause for scour concern, the same velocities may be used in the design of a new structure. Higher velocities may be allowed if the site evaluation determines those velocities will not be detrimental.

2.3.10.4 Backwater

A bridge is generally an obstruction in a stream or river that can cause a rise in water level behind the bridge, known as backwater. The height of this backwater can also be a controlling factor in the sizing of a bridge. The affect of backwater on upstream property must be considered. The determination of water levels from an existing bridge is an important guide in evaluating the backwater height of a new structure. FEMA regulations require that the backwater at Q100 increase no more than 1 foot.

2.3.10.5 Dams

Bridges influenced by the presence of dams should be analyzed hydraulically for the following two situations:

- Existing dam remains in place
- Existing dam is removed

Many dams throughout Maine are now being removed. All new bridges should be designed so that any nearby dams can be removed with no adverse effect to the bridge. Some analysis may be needed for the case where a major dam (typical high head) will remain in place. The water level may be lowered for dam maintenance or emergencies for an extended period of time.

2.3.10.6 Fish Passage

MaineDOT's fish passage policy and design guide is available at the following website: http://www.state.me.us/mdot/finalfishpassage5.pdf. Designers should refer to this guide to insure that fish passage is maintained.

2.3.11 Scour

Commentary: Flooding is the most common cause of bridge failure, with the scouring of bridge foundations being the most common failure mechanism. The catastrophic collapse of the Interstate 90 crossing of Schoharie Creek near Amsterdam, NY on April 5, 1987, is one of the most severe bridge failures in the U.S. Two spans fell into the water after a pier supporting the spans was undermined by scour. Five vehicles plunged into the creek killing 10 people. The National Transportation Safety Board concluded that the bridge footings were vulnerable to scour because of inadequate riprap around the base of the piers and a relatively shallow foundation. The I-90 collapse focused national attention on the vulnerability of bridges to failure from scour and resulted in revisions to design, maintenance, and inspection guidelines.

MaineDOT initiated a scour-screening program in 1987 in response to FHWA Technical Advisory TA 5140.20 (succeeded by TA 5140.21 and TA 5140.23). The advisories ultimately require that a master list be generated of all bridges that require underwater inspection, and that all applicable bridge foundations be evaluated and prioritized according to their vulnerability to scour damage.

2.3.11.1 New Bridges

Bridges over waterways with scourable beds should be designed to withstand the effects of scour from a superflood (a flood exceeding Q100) without experiencing foundation movement of a magnitude that requires corrective action. A scour analysis will be performed for all bridge-type structures using the methods in the latest version of HEC-18. The design flood for scour is the lesser of Q100 or the overtopping flood. Maximum scour depths will be produced by the overtopping flood. Scour should also be computed for the superflood, defined as Q500 or the overtopping flood if it is between Q100 and Q500. Q500 can be estimated as 1.18 times the magnitude of the Q100, if Q500 cannot be computed by other means.

The bridge foundation should be designed for the normal factor of safety as specified in AASHTO Standard Specifications below the scour depths estimated for Q100. The bridge foundation should have a factor of safety of 1.0 for scour produced by the superflood. The footings should be placed a minimum of 2 feet below the design flood scour level. Where pile bents are used, the design friction or point bearing should be achieved below the depth of the design scour. There must be sufficient pile penetration below the scour line to provide lateral stability and structural capacity to support the calculated loads.

The geotechnical analysis of bridge foundations should be performed on the basis that all stream bed material in the scour prism above the total scour line for the scour design flood has been removed and is not available for bearing or lateral support.

When analyzing piers for local scour, the pier width should be increased by a minimum of 25% to account for the collection of debris.

The bottom of spread footings on soil for nonspill-through type abutments shall be located a minimum of 6 feet below the lowest streambed elevation in the immediate vicinity of the bridge (two bridge lengths upstream or downstream of the bridge or 50 feet, whichever is larger).

2.3.11.2 Existing Bridges

If there is a history of scour at an existing bridge that is to be rehabilitated, then a scour evaluation should be performed for the following project scopes to determine whether the bridge is scour-critical:

- Deck Replacement
- Superstructure Replacement
- Bridge Widening

A scour-critical bridge is one with abutment or pier foundations that are rated as unstable due to one of the following:

- Observed scour at the bridge site
- Scour potential as determined from a scour evaluation study (refer to HEC-18 Chapter 5)

Designers should consult with Bridge Maintenance on scour-critical bridges to determine if the use of non-designed countermeasures and/or regular inspections may be an acceptable method to reduce the risk of failure. If not feasible, a hydraulic analysis will be needed to properly design scour countermeasures or to analyze a new bridge structure.

A plain riprap apron can be used as a designed scour countermeasure around an existing pier, if the velocity at the design flow is less than 5.3 fps. A heavy riprap apron can be used as a designed scour countermeasure around an existing pier if the velocity at the design flow is greater than 5.3 fps, but less than 8.8 fps. The riprap apron should have a minimum width of 10 feet perpendicular to the centerline of the structure.

2.3.11.3 Riprap Slope Protection

Riprap slope protection should normally be plain riprap and be 3 feet thick with the toe constructed 1 foot below final ground or streambed elevation. Thicker riprap and/or deeper toe depths may be warranted at locations of severe stream velocities and/or in scour susceptible streambeds, as determined by hydraulic analysis. When heavier riprap protection is needed, it should be a 4 foot thick layer of heavy riprap with the toe constructed 3 feet below final ground or streambed elevation.

Bedding material, which will also serve as a filter blanket, should be provided beneath all riprap installations. In tidal locations, a geotextile filter material should be utilized under the riprap instead of the bedding material.

On stream crossing projects, riprap should be placed in front of spill through type abutments and wingwalls. The top of the riprap should be located to protect the abutment embankment from scour and to provide adequate cover above the bottom of footings in accordance with this section and Chapter 5 Substructure.

For culvert-type structures, riprap should be placed on the embankment slopes around the upstream and downstream ends of the structure. The top of the riprap should be located at the Q50 elevation. The Q50 elevation may be lower on the downstream end due to stream slope and/or upstream ponding as determined by the hydraulic analysis of the site and structure. The riprap should extend horizontally a minimum of 5 feet on either side of the culvert.

Scour and/or erosion protection of stream channel bottoms at the inlet and/or outlet of culvert-type structures should be provided where required to prevent scouring of the streambed and undermining of the structure. It should be designated as a plain riprap apron and be 2 feet thick. Culverts with high outlet velocities may require a 3 foot thick heavy riprap apron. Culverts with very high outlet velocities may need energy dissipators. Energy dissipators should be designed in general accordance with the procedures in FHWA HEC No. 14.

Riprap should also be provided on the roadway approach embankments of bridge and culvert-type structures to further protect the structure from erosion or scour damage. The lateral extent of riprap protection of the embankments from a bridge or culvert-type structure should be sufficient to provide protection from unimpeded flow upon the embankment slopes on the upstream side of the stream crossing, and for a distance of 5 feet beyond the top of stream banks on the downstream side of the stream crossing. The top of the riprap should be located at the Q50 elevation.

Protection of roadway embankments, other than by vegetative cover, is generally not necessary except at locations where a stream crossing is on a severe skew to the flood plain, and stream flow can occur along the embankment slopes.

At tidal crossings, the top of riprap should be located at a minimum elevation of 2 feet above MHHW. Consideration should be given to placing the riprap even higher due to waves and wave runup. Each site should be evaluated on a project-by-project basis.

Bridges located immediately on the ocean should use heavy riprap. Heavy riprap should also be used when the average velocity is 15 fps or greater. The use of heavy riprap should be given serious consideration when the average velocity is between 12 fps and 15 fps, especially when ice is a problem.

2.4 Maintenance of Traffic During Construction

2.4.1 General

The method of maintaining traffic during construction must be considered for all bridge projects. In general, the preferred method is to close the bridge and detour traffic on adjacent roads. This will usually result in the shortest construction time, and therefore, a less expensive project. However, this method is not always feasible due to long detour routes, poor quality roads, or high traffic volumes.

The following factors should be considered when determining the best method of maintaining traffic.

- *Traffic composition.* A high percentage of trucks, RV's, or school buses will require larger turning radii and wider lanes.

- *Mobile homes and other wide loads*. On projects where staged construction is required for extended periods of time on single access roads (only one way in and out) consideration should be given to coordinating the movement of mobile homes and other wide loads. This can be done by either coordination with the Contractor during construction, requiring the Contractor to open the bridge on preset days in the contract documents, or maintaining at least one 16 foot or wider lane during construction.

- *Traffic volume*. One lane can accommodate up to 1700 vehicles per hour in free flow conditions. Low volumes can be more easily absorbed on local roads.

- *Proposed lane width.* Eleven feet is the minimum width required, though 10 feet may be used in special circumstances. For high volume roads or roads with many trucks, lanes should be 12 feet wide or greater.

- *Required work zones*. Sufficient width must be provided for the Contractor to accomplish the scope of work.

- *Bridge length.* A bridge greater than 500 feet in length may cause unacceptable stop times when using alternating one-way traffic. Shorter work zones should be considered.

- *Adjacent side roads or driveways.* Provisions should be made to allow traffic to enter and exit.

- *Emergency vehicles.* The effect of construction on response time of police, fire, and ambulances must be considered.

- *Geometric issues.* Advanced warning devices may be needed if visibility is compromised as the driver approaches.

- *Pedestrian and bicycle traffic.* A determination should be made whether pedestrian and bicycle traffic can be maintained during construction, and how it will be done.

- *Bridge curvature.* A curved bridge may have less usable width, and will likely require wider lanes.

A Traffic Control Plan (TCP) must be developed for every project. Responsibility for this plan is with either the Contractor or MaineDOT, as determined at the PS&E stage. The complexity of the project may steer the Structural Designer toward keeping this responsibility within MaineDOT, to assure compliance with the conceptual design. Any TCP must comply with the latest edition of the Manual of Uniform Traffic Control Devices (MUTCD).

2.4.2 Methods to Maintain Traffic

There are three ways commonly used to maintain traffic. They are discussed here in order of generally increasing costs. The fourth method is an innovative approach that has been used successfully on a number of projects.

2.4.2.1 Close the Road and Detour on Existing Roads

Care should be taken in evaluating proposed detour routes. Detours should be routed using state or state aid highways with input from both the Division Traffic Engineer and municipal officials. Exceptions to using these highways can be made with written concurrence of the town, with agreement to relieve MaineDOT of responsibility for any deterioration caused by the detoured traffic. It is prudent to discuss the detour with emergency services prior to advertising.

2.4.2.2 Staged Construction

This involves maintaining traffic on part of the existing bridge for the first phase of construction, building a portion of the new bridge, and then moving traffic to the new portion to complete demolition of the existing and construction of the new structure. If possible, two lanes of opposing traffic should be maintained during staged construction. If only one lane is maintained, alternating one-way traffic can be controlled either by using temporary signals, or by posting with a yield/stop condition. Yield/stop conditions may be considered if the average annual daily traffic (AADT) is less than 1500 vehicles per day, and the sight distance is adequate for the posted speed or the 85th percentile speed.

2.4.2.3 Temporary Bridge

A temporary bridge should be considered when other methods are not feasible. Depending on expected traffic volumes, the temporary bridge may carry one lane of alternating one-way traffic, or two lanes of opposing traffic. The Contractor is responsible for the design of the bridge, with approval obtained by MaineDOT. Sufficient right-of-way and environmental permitting must be obtained to allow the Contractor to design the structure adequately. Prior to construction, the Resident should carefully review the Contractor proposed design and drawings of the temporary bridge to assure compliance with Standard Specifications Section 510 – Special Detours. The Contractor proposed design must be within the right-of-way provided and the obtainment of additional right-of-way by the Contractor will not be allowed. The Structural Designer may be asked to review the Contractor's plans and computations.

2.4.2.4 Innovative Methods

The existing superstructure can sometimes be used to maintain traffic off the existing alignment at a significant savings over a temporary bridge. Temporary supports can be constructed, and the existing superstructure slid over to rest on the temporary supports. This has been done with both truss structures and conventional girder/deck systems. The proposed bridge is then constructed either in whole or using staged construction methods, while traffic is maintained on the existing superstructure.

When night work can be specified, wearing surface replacement on high volume bridges has been done using rapid construction methods, such as grinding the wearing surface and replacing it with a fast-setting topping.

The work is done in sections over several nights, keeping one lane open, with the bridge reopened to two lanes of traffic by morning each day.

For work on Interstate bridges, the use of crossovers has been incorporated on large deck replacement projects. Crossovers are constructed on both ends of the bridge allowing for two-way travel on one side of the divided highway and closure of the other side. This scheme has also been used for the construction of new overpass bridges.

2.5 Geotechnical and Survey

Prior to the start of field work, the team should agree upon the necessary field data. The Structural Designer may meet with the Survey Coordinator and the Geotechnical Designer to determine the limits of survey and optimal locations for test borings, respectively.

2.5.1 Geotechnical

Geotechnical design must be done in conjunction with structural design to optimize the selected structure type for the PDR. The Structural Designer and Geotechnical Designer will work together as part of the team process. Considerations include:

- The Geotechnical Designer will provide preliminary foundation and earthwork design recommendations for the PDR. This preliminary analysis may require a subsurface exploration, or may be done based upon existing subsurface data.
- Test borings will generally be required for each proposed substructure unit for final design. Precise boring locations cannot be determined until the Structural Designer has set the proposed alignment with stations for abutments and piers.
- Reuse of existing substructure units will usually require an analysis of the substructure stability under new loads. Refer to Sections 10.6 and 10.7 for information regarding substructure rehabilitation and substructure reuse, respectively.

2.5.2 Field Survey

Survey of the bridge site will be necessary for most projects (refer to Section 1.5 Small Bridge Initiative for exceptions). Ideally, the Designer should meet with the Survey Coordinator, preferably on site, to determine the limits of survey. However, many times the survey is done prior to the Structural Designer beginning work in order to advance the project schedule.

The “Survey Manual” gives guidelines used by survey crews to obtain project survey. The most discriminating characteristic is whether the project is a replacement or rehabilitation. For a replacement project, survey will tie in the structure by locating the corners. If accurate as-built plans are available, this will often be enough information to design the new structure. For a rehabilitation project, highly detailed structure information is necessary. For example, the information gathered will include curb lines, wingwalls top and bottom, breast walls, bridge seats, piers top and bottom, etc.

The following information is collected routinely on a bridge project:

- *Limits of survey along the roadway:* Most projects will require at least 150 to 200 feet on either end of the bridge to accommodate required guardrail lengths. If the roadway is curved, consider the need to match into the existing curve and obtain enough data points to do so. If the new structure is expected to be off alignment, additional length will be needed.

- *Limits of survey from the existing centerline:* Most projects will require at least 60 feet from the centerline, to accommodate toes of slope and to define drainage.

- *Stream data*: The edge of stream for 75 feet upstream and downstream will be obtained for right-of-way purposes. Bottom of stream points will be obtained 60 feet from the centerline, usually by wading or from a small boat. For larger structures, a string will be obtained at a distance of 2 times the span length upstream and 1 times the span length downstream for hydraulic analysis. Additional sections should be requested, if needed.

- *Wetlands:* This information is needed for permitting. It is obtained by the Environmental staff, either through a hand held GPS unit, or through flagging and later collection by traditional survey (preferred method).

- *Vertical control:* When a known datum is within a mile of the project, a level loop is run, providing accurate NGVD information. In a remote area more than a mile from a known datum, GPS will be used, which can result in the absolute elevation being inaccurate by as much as 8 inches. An effort should be made to tie down flood elevations to known elevations. However, relative elevations will be reliable within the project limits.

For some projects, additional information should be collected. For example, on culvert rehabilitation projects, if the shape of the existing culvert must be verified, the interior of the pipe or pipe arch should be surveyed. Points at the top, bottom and the quarter points of the culvert should be taken at roughly 10 foot intervals along the centerline of the culvert.

For culvert replacement or culvert rehabilitation projects with fish passage concerns, grade control structures may be needed to maintain fish passage. If so, survey will be needed along the centerline of the stream at least 40 feet downstream of the end of any scour hole. Survey should extend a minimum of 20 feet on both sides of the stream or up to an elevation roughly 1/3 the height of the culvert. Depth of water at the upstream and downstream end of the culvert is also obtained by taking shots of the water surface.

Some projects will also need stream cross sections to create a hydraulic model. Generally, an absolute minimum of four sections of the stream is needed. The stream/river sections should include the streambed under water and the entire stream bank.

For larger projects, other means of collecting data should be considered. Photogrammetry may save time when many data points will be required. Fathometry may be preferred for very deep rivers or tidal areas. These options may be discussed with the Survey Coordinator.

2.6 Utilities and Right-of-Way

It is important to involve utilities and right-of-way team members in the project from the beginning. Considering the impacts of the design throughout the process will best address utility relocation issues and property owner concerns as they arise. Refer to Section 4.10 for utility attachment restrictions.

2.7 Alignments

2.7.1 General Highway Design Guidelines

In general, the alignment of the road is chosen first, which then determines the alignment of the bridge. Hydraulic, environmental, and economic concerns may result in an exception.

The Designer should refer to the “MaineDOT Urban & Arterial Highway Design Guide” for uniform design practices of approaches for collector roads, and to the current edition of AASHTO A Policy on Geometric Design of Highways and Streets for arterials. For local roads, the “MaineDOT Urban & Arterial Highway Design Guide” should generally be used; however, a lesser standard may be acceptable, particularly with low current traffic volumes, limited potential for growth, and potential adverse impacts to property owners, the environment, and economics of the area.

When the approaches to a bridge must be on a curved horizontal alignment, the Designer should keep any superelevation transitions off the bridge, if at all possible. The geometry of a superelevation transition can create an undesirable level area on the bridge deck, resulting in poor drainage, and can increase the cost of structural steel due to the complicated geometry.

2.7.2 Bridge Guidelines

2.7.2.1 Horizontal Alignment

When possible, a bridge should be located on a tangent section, since curvature increases the cost of the superstructure and can result in an undesirable safety situation during inclement weather. The “Plan Development and Estimating Guide” has details showing general bridge layout on a tangent, curve, and partial curve, as well as layout of a buried structure.

2.7.2.2 Vertical Alignment

When possible, the vertical alignment should be such that low point of the sag vertical curve is not at the bridge. A minimum 1% grade should be maintained across a bridge in order to facilitate positive drainage. If a 1% grade creates undesirable environmental or right-of-way impacts, then the grade may be reduced to 0.5%.

2.7.3 Clearances

2.7.3.1 Railroad

For new structures over railroads, the minimum clearances are shown in Figure 2-1, and are subject to the approval of the utility. The typical section shown should be used as a guide only. All railroad sections must be coordinated with the railroad on a project-by-project basis for approval.

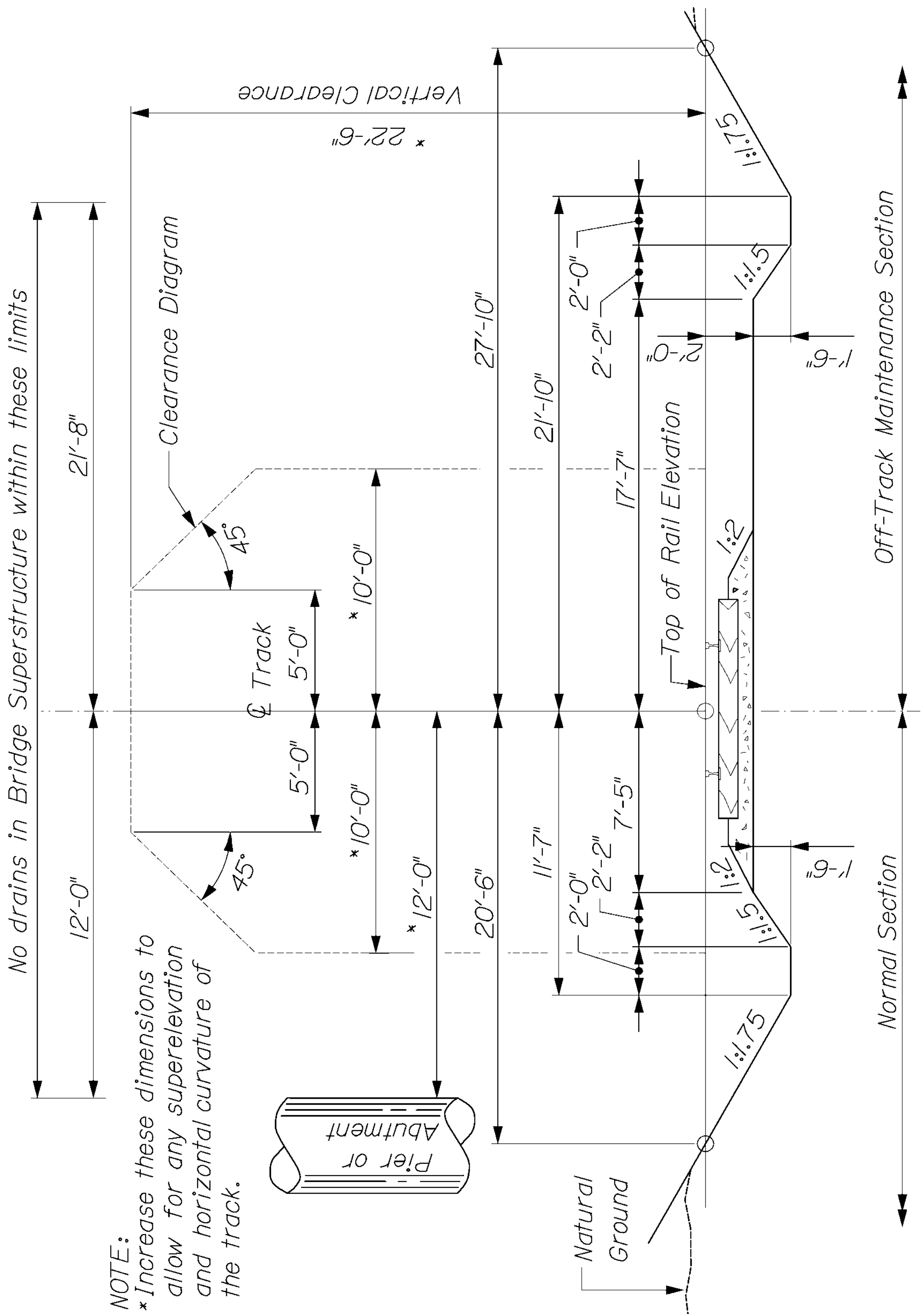

Figure 2-1 Typical Railroad Cut Section

The American Railroad Engineering and Maintenance of Way Association (AREMA) Manual for Railway Engineering (2002), Chapter 8 paragraph 2.1.5, requires that piers located within 25 feet of the tracks shall be of heavy construction or shall be protected by a crash wall.

2.7.3.2 Grade Separations

The legal minimum underclearance without posting is 14’-6”.

For new structures over roads other than Interstate roads, the minimum underclearance is 15’-0” and the preferred underclearance is 15’-6”. The minimum underclearance allows 6 inches for future pavement overlays and construction tolerances, and the preferred underclearance provides for an additional 6 inches of unknown conditions. The preferred underclearance is to be used for preliminary designs.

The corresponding underclearances for structures over the Interstate System are 16’-0” and 16’-6”.

When a roadway is resurfaced under a structure, it may be necessary to excavate the existing pavement prior to placing new pavement in order to maintain the minimum underclearance and avoid the need for posting. In general, 16’-0” clearance for the Interstate and 15’-0” for other roads should be provided after resurfacing improvements are made, if other bridges on the corridor segment have corresponding minimum underclearances. To avoid posting, there should be an actual underclearance of 14’-6” minimum after improvements are made.

2.7.3.3 Underclearance for Stream Crossings

Refer to Section 2.3 Hydrology, Hydraulics, and Scour. For guidance on Coast Guard clearances and permits, refer to the Outside Agencies Chapter of the Bridge Program’s “Project Management Guide.”

2.7.3.4 Clearance Between Parallel Structures

In order to provide adequate room for certain maintenance activities such as painting and inspection, 10 feet minimum should be provided between parallel structures.

Under extreme circumstances, a 6 foot clearance may be allowed with concurrence from Bridge Maintenance.

2.7.3.5 Underclearances for Non-Vehicular Bridges

Non-vehicular bridges should meet the underclearance requirements in Sections 2.7.2.2 and 2.7.3.3.

2.8 Approaches

2.8.1 Roadway Widths

This section is a guide for use in determining the appropriate width of the approaches to a bridge. For geometric design criteria of bridge widths, refer to Section 4.1 Bridge Widths.

For projects on the NHS, widths must comply with the current edition of <u>AASHTO A Policy on Geometric Design of Highways and Streets</u>. Rural NHS roadways should not be designed for less than 40 mph. Refer to Figure 2-2 for the designated NHS in Maine.

All roads and streets (excluding the Interstate) are classified according to function. The proper function can be found in MaineDOT's ProjEx system for any given project. The functions are as follows:

- Local roads
- Minor and major collector roads
- Minor and major arterials

Each of the classifications is further divided into two categories: urban and rural. For urban streets, existing approach widths should be investigated for their propensity to be widened or altered in the future. For rural roads, the Designer should determine from the Bureau of Planning whether the corridor is planned for widening in the future.

2.8.1.1 Local Roads

For local roads, the approach width should match the bridge width with the guardrail-to-guardrail width matching the rail-to-rail width on the bridge. Good engineering judgment is required when determining the appropriate width for a local road. Factors that need to be considered are:

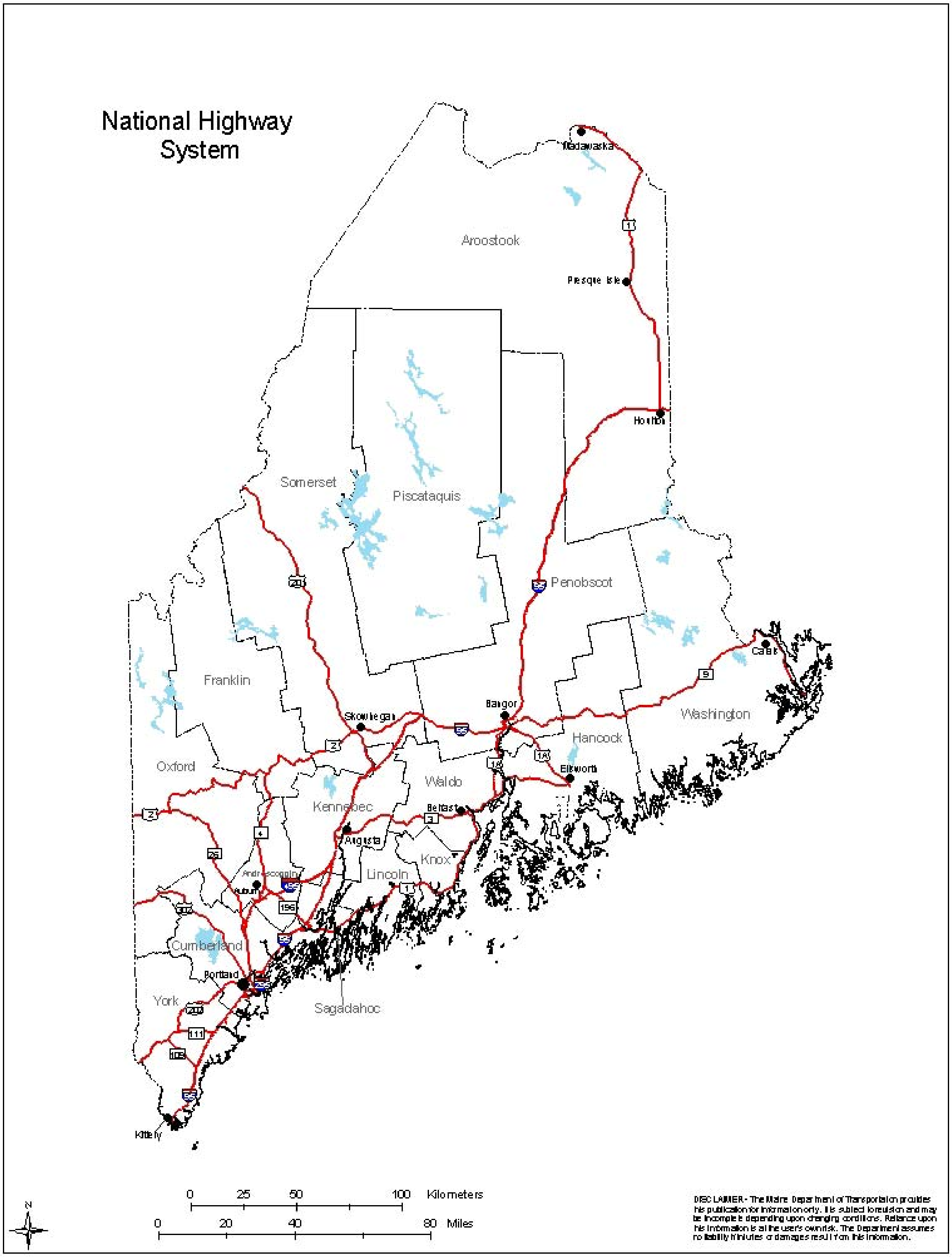

Figure 2-2 NHS in Maine

- Current right-of-way limits
- Geometric alignment
- Traffic volume
- Propensity for growth

2.8.1.2 Collector Roads

The approach guardrail (attached and immediate to the bridge) should be set at the same width as the bridge rail. For bridges on collector roads with extensive approaches, refer to the “MaineDOT Urban & Arterial Highway Design Guide” for appropriate shoulder widths and guardrail offsets.

2.8.1.3 Arterials

Roadway widths for approaches on arterials should comply with the latest AASHTO A Policy on Geometric Design of Highways and Streets.

2.8.2 Guardrail

2.8.2.1 General

On the NHS, terminal ends must meet the requirements of NCHRP 350 in conjunction with either guardrail type 3d on Interstate projects and 3c on non-Interstate NHS. Refer to Section 10 of the “MaineDOT Urban & Arterial Highway Design Guide” for further guidance. On non-NHS roadways with an AADT>500, use a Modified Eccentric Loader Terminal (MELT) as an end treatment with guardrail type 3 or 3b as appropriate. On non-NHS roadways with AADT of 500 or less, use the Low Volume Guardrail End with guardrail type 3 or 3b as appropriate. For more information on guardrail types, refer to the Standard Specifications and Standard Details.

2.8.2.2 Guardrail Treatment on Local Roads

Bridge approach guardrails protect motorists from roadside hazards such as non-negotiable foreslopes, telephone poles, trees, streams, and rivers, and provide safe transitions to the bridge rail system. For guidance on bridge rail systems, refer to Section 4.4 Bridge Rail. Termination of these systems is controlled by the steepness of the foreslopes, location of obstacles, and the geometry of the stream crossings. Termination design criteria are presented in the current edition of the AASHTO Roadside Design Guide and the “MaineDOT Urban & Arterial Highway Design Guide”.

The use of these criteria can result in lengthy terminations and can extend projects beyond the lengths required to meet the objective of the project.

Bridge projects on local roads are intended to upgrade deficient structures and provide cost effective guardrail systems. This section provides design criteria for local bridge projects that minimize guardrail termination lengths and also eliminate MELTs in some instances.

The termination and MELT design criteria set forth in this section are intended for use only on roads for which the functional classification is local. Other projects should be designed in accordance with the guidelines and policy set forth in the “MaineDOT Urban & Arterial Highway Design Guide”.

Use the following definitions in this section:

- *Clear zone:* The clear zone is an unencumbered area measured perpendicular to the roadway that allows out of control vehicles leaving the roadway to recover.
- *Non-recoverable slope:* A slope that motorists can traverse but from which most motorists will be unable to stop or return to the roadway. Slopes that are between 4:1 and 3:1 are considered traversable but non-recoverable.
- *Critical slope:* A slope on which a vehicle is likely to overturn. Slopes that are steeper than 3:1 are considered critical.
- *Recovery area:* Sum of the clear zone and the non-recoverable and critical slopes.
- *Lateral extent of hazard:*

  Stream that extends beyond the clear zone: The point where the outer limit of the recovery area intersects with the top of the non-negotiable slope at or near the stream edge.

  Fixed object such as a tree, pole, etc.: The distance from the edge of the traveled lane to the far side of the hazard.
- *Runout path*: Theoretical path an out of control vehicle will follow as it leaves the roadway at the point of need.
- *Point of need:* The last point at the face of guardrail where a vehicle can leave the road and follow the runout path without traversing a critical slope or hitting a Deadly Fixed Object.

Refer to Figure 2-3 and Figure 2-4 for further guidance.

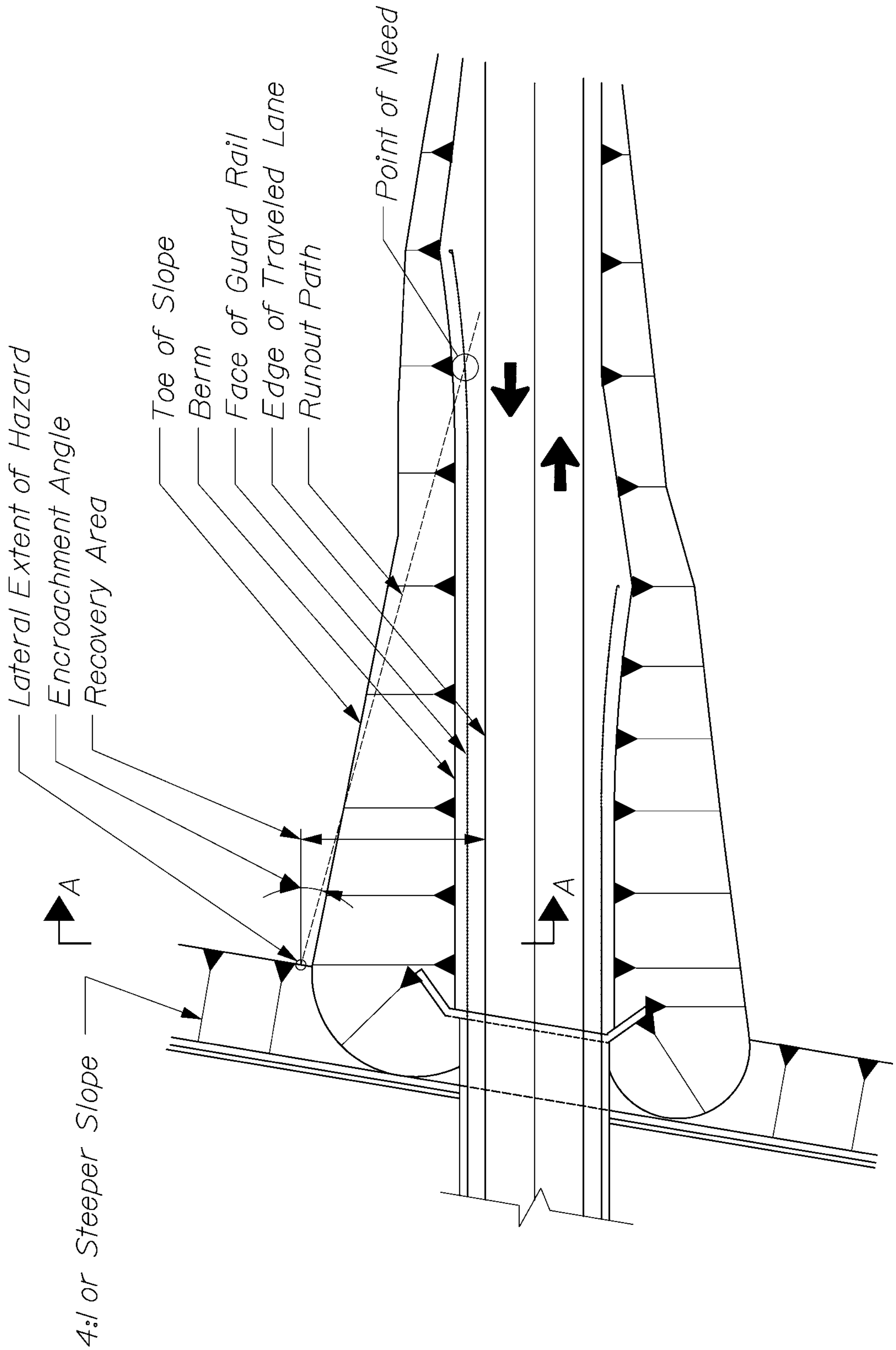

Figure 2-3 Point of Need Definition

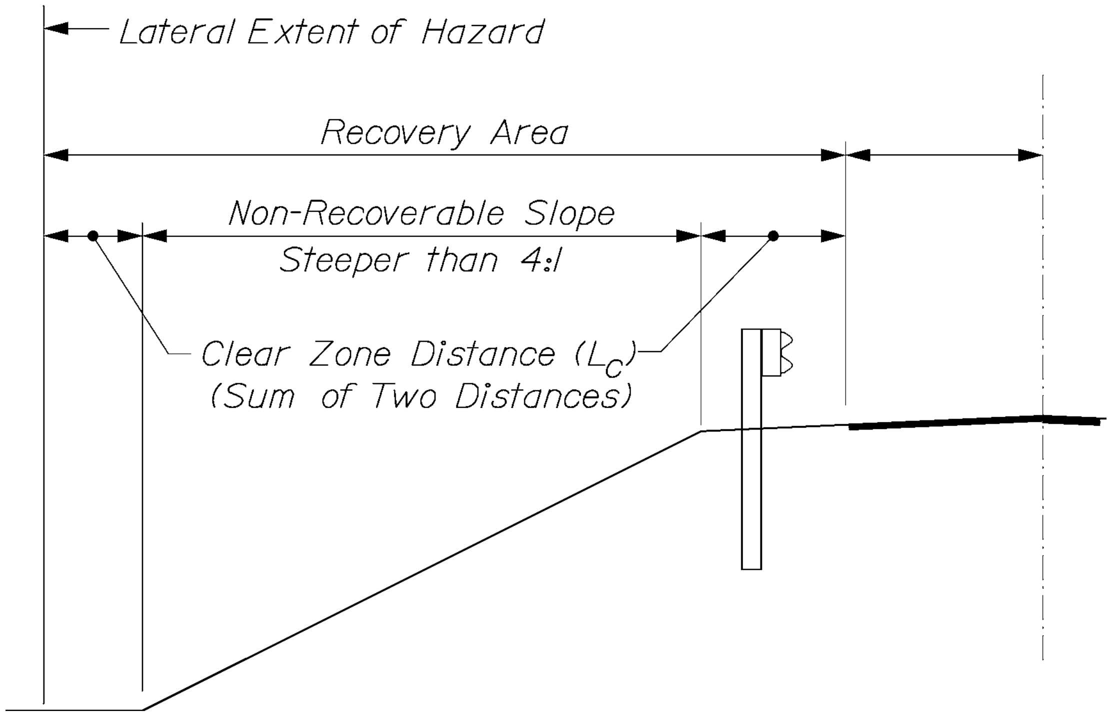

Figure 2-4 Lateral Extent of Hazard Definition

Procedure 2-1 has been developed to determine the proper treatment of the terminal end for the Leading End and Trailing End.

Procedure 2-1 Guardrail End Treatment on Local Roads

For the Leading End, follow the procedure below.

a. Establish the clear zone distance (L_c) based upon the design future traffic volume and the design speed. (Refer to Table 2-4)

b. Locate the lateral extent of hazard.

c. Establish the runout path and the point of need by extending a line from the limit of hazard point to the face of guardrail at the encroachment angle based upon the design speed. (Refer to Table 2-5)

d. Provide an end treatment beyond the point of need:

AADT< 500: Extend the rail 50 feet with a low volume guardrail end.
AADT> 500: Use a MELT

The use of MELTs should be examined on local road projects where maintenance will be provided by the local government. These facilities may not be maintained, and after a MELT is hit and damaged, it may be more dangerous than a standard flared terminal end.

e. Where possible provide a minimum length of 100 feet from the bridge to the end of the guardrail. The length of the project should be extended if necessary to provide this minimum length of guardrail.

A minimum length of guardrail should be provided regardless of the project length to provide adequate protection at the approach rail - bridge rail interface.

Guardrail may be extended onto the approach transitions or even beyond the transitions by rehabilitating the existing shoulders and defining a limit of work beyond the end of the transition.

Table 2-4 Clear Zone

AADT (Future)	Clear Zone (Lc, ft)		
	30 mph	40 mph	50 mph
<200	5	7	8
200 to <400	6	8	10
400 to <800	7	10	12
800 to <2000	10	12	14
2000 to <6000	12	15	18
6000+	14	17	20

Table 2-5 Encroachment Angle

Design Speed	Encroachment Angle
30 mph	15°
40 mph	12°
50+ mph	10°

For the Trailing End, follow the procedure below.

a. The required clear zone width for the trailing end (measured from the centerline of the road to the lateral extent of the hazard) is within the width of the adjacent lane plus the shoulder for an AADT less than 6000. Stream protection need not be considered unless the AADT equals or exceeds 6000, or unless terrain features (such as a stream which is skewed to or nearly parallel with the roadway) require consideration.

b. Establish the point of need at the face of guardrail adjacent to the first 3:1 slope. (Where the transition from a 3:1 to a 2:1 slope begins.)

c. Provide an end treatment beyond the point of need:

- AADT< 500: Extend the rail 50 feet with a low volume guardrail end.
- AADT> 500: Use a MELT

d. Where possible, provide a minimum length of 50 feet from the bridge to the end of the guardrail.

Other special conditions may also require consideration for guardrail treatment on local roads, including terrain features, approach curves, ditches, intersections, and driveways.

Certain terrain features can reduce the need for long guardrail lengths. If the calculated guardrail length exceeds the minimum requirement of 100 feet, examine the terrain along the runout path and within the clear zone. Will a motorist likely avoid the hazard by entering a field or open space before reaching the hazard? Will a motorist likely become hung-up in the brush before reaching the hazard? Is the stream bank flat (3:1 or flatter) and the stream shallow (3 feet or less at normal water) so that the motorist will be safer entering the stream than hitting the guardrail? These features must be evaluated on a project-by-project basis, and proposed guardrail reductions approved by the project team.

Longer guardrail lengths may be required to protect vehicles from utility poles and non-breakaway signs located within the clear zone.

When an approach curve is present, along with a high accident history, increasing the clear zone width, L_c, may reduce accident potential. For sharp approach curves, the runout path should follow a line tangent to the curve to the lateral extent of hazard.

Ditches may affect guardrail length. Trapezoidal approach ditch sections (2 feet wide at the bottom) should have 3:1 or 4:1 (preferred) foreslopes and 2:1 backslopes in areas where the ditches are parallel to the direction of travel. In areas where traffic could be expected to cross the ditch at a sharp angle such as the outside of a curve, the slopes should be flattened to conform to the recommendations in the AASHTO Roadside Design Guide.

If intersections, drives, or field entrances are found within the runout length, adequate sight distance must be provided. Guardrail should be wrapped into the entrance and terminated with a standard terminal end. MELTs should be used on side roads where AADT exceeds 500.

The following Example 2-5 illustrates concepts shown in Procedure 2-1.

Example 2-5 Guardrail End Treatment on Local Roads

Given: Design Speed= 45 mph
AADT= 650
11 ft Lane width
4 ft to face of rail
3 ft from face of rail to berm

Problem: Determine the point of need for the leading and trailing ends.

Solution: Follow the Guardrail Treatment on Local Roads Criteria. Refer to Figure 2-5 and Figure 2-6.

Leading End

Step 1: Determine the clear zone distance from Table 2-4. The 45 mph design speed must be rounded to the next highest design speed given in the table, 50 mph. L_c = 12 ft

Step 2: Determine the lateral extent of hazard. In this example, the stream is the hazard. Since the stream extends beyond the recovery area, the lateral extent of hazard is the point where the limit of the recovery area meets the first non-recoverable slope (steeper than 4:1) at the edge of the stream.

Step 3: Establish the runout path. For the 45 mph design speed, round to 50 mph then select the encroachment angle from Table 2-5. Encroachment angle is 10°

Step 4: Locate the point of need. Extend the runout path to the face of guardrail. The intersection is the point of need. The length of guardrail exceeds the minimum of 100 ft.

Step 5: Provide an end treatment. The AADT exceeds 500, therefore use a MELT. The last 3:1 foreslope should be located 50 ft from the point of need. The slope should be transitioned to 2:1 in 50 ft.

Trailing End

Step 1: From above, the required clear zone is 12 ft. Since the distance from the edge of the traveled lane (in this case the centerline of the roadway) to the face of rail of 15 ft is greater than the clear zone, stream protection is not necessary.

Step 2: Establish the point of need as the last 3:1 slope. In this case the side slope 50 ft from the bridge is 3:1, therefore use 50 ft from the bridge to the point of need.

Step 3: Since the AADT of 650 is more than 500, extend the rail 50 ft and use a MELT.

Step 4: The length of rail is 100 ft, exceeding the 50 ft minimum distance from the bridge.

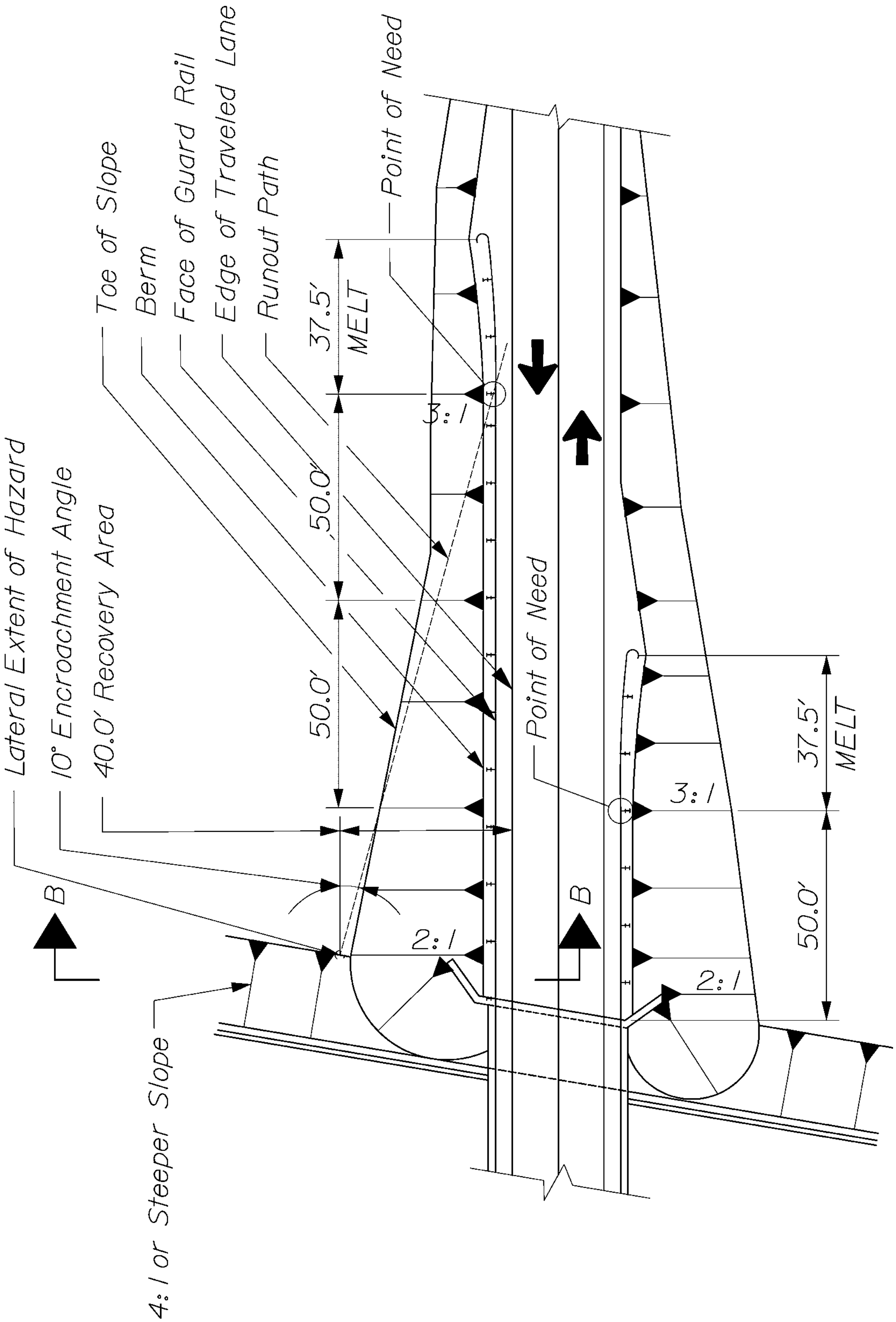

Figure 2-5 Point of Need Example

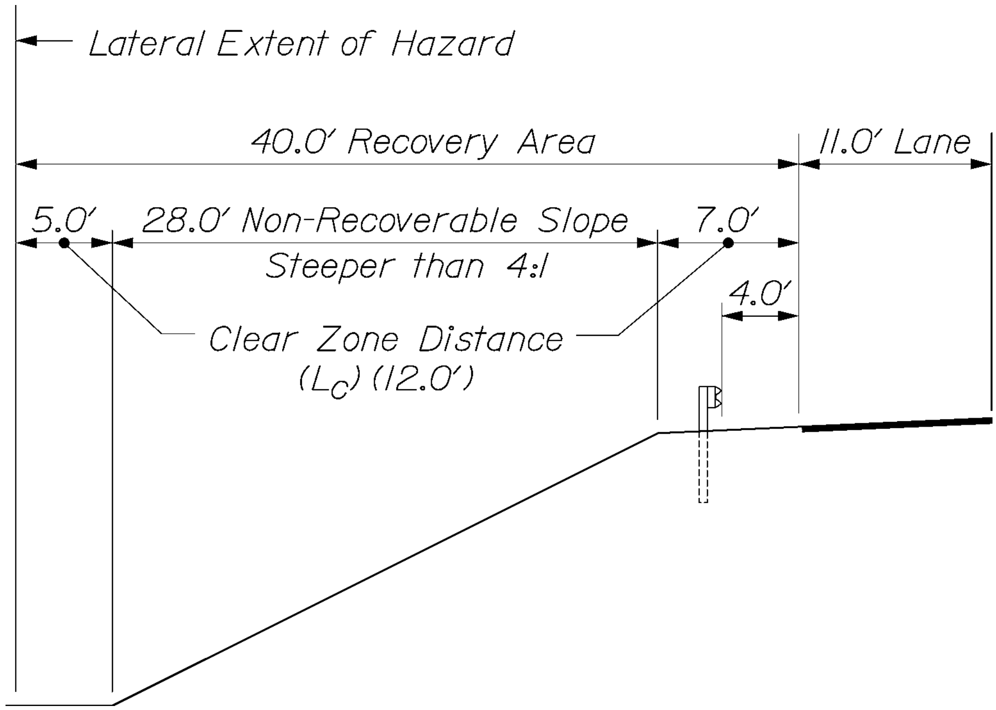

Figure 2-6 Lateral Extent of Hazard Example

2.8.3 Reduced Berm Offset

For projects on low volume, low speed local roads, consideration may be given to reducing the 3 foot offset from the face-of-guardrail to the berm to 2 feet in order to minimize wetland, right-of-way, or other construction impacts.

When reduced berm offsets are utilized, the guardrail posts must be lengthened and the embedment increased, as shown in Figure 2-7.

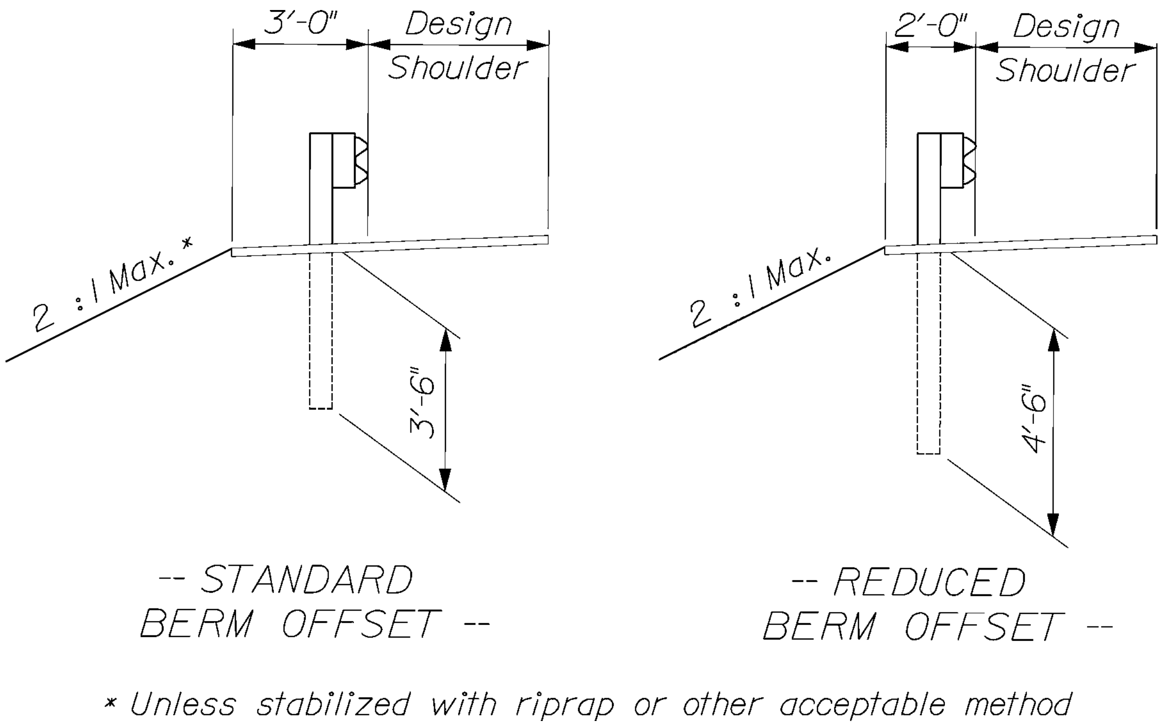

Figure 2-7 Reduced Berm Offset

2.8.4 Pavement Design

2.8.4.1 General

A. Layer Thickness

Use Table 2-6 for maximum and minimum layer thickness for a particular grade of pavement, in order to achieve the required density. Each grade may require more than one layer.

Table 2-6 Pavement Layer Thickness

Item Number	Description	Minimum Thickness (in)	Maximum Thickness (in)
403.210	3/8 in	1	1-1/2
403.208	1/2 in	1-1/8	2
403.207	3/4 in	2	3-1/4
403.206	1 in	2-1/2	4

B. Layers Across Roadway

Use Table 2-7 to estimate typical pavement layer thickness for traveled way and shoulders. For wearing courses on bridges, refer to Section 4.6 Wearing Surfaces.

Table 2-7 Number of Layers Across Roadway

Travelway Depth (in)	Number of Layers	Shoulder Depth (in)	Number of Layers	Mix Type
3	2	1-1/2	1	1/2 in
3	2	3	2	1/2 in
4	1@ 1-1/2 1 @ 2-1/2	1-1/2	1	1/2 in 3/4 in
4	1 @ 1-1/2 1 @ 2-1/2	3	2	1/2 in 3/4 in
5	1 @ 1-1/2 1 @ 3-1/2	1-1/2	1 1	1/2 in 3/4 in
5	2 1 @ 2	3	2 1	1/2 in 3/4 in

2.8.4.2 Arterials and Collectors

Pavement for approaches located on all arterials and collectors, on and off the NHS, should be designed in accordance with the 1993 AASHTO design standards. The DARWin™ Pavement Design System designs pavement and aggregate subbase course gravel thicknesses based on the 1993 AASHTO Standards. Establishment of a new design standard is currently in progress.

For assistance in determining the Terminal Serviceability and Reliability Level (%), consult with a Designer in the Urban and Arterial Program.

Table 2-8 contains sample input data for the DARWin™ program.

Table 2-8 DARWin Input

<table>
<tr><td>18-kip ESALs over Initial Performance Period</td><td colspan="2">Use equivalent 18k loads from AADT, expanded over the entire pavement design period, typically 20 years.

Example: 95 axles/day x 365 days/yr. X 20 yr. = 693,500 ESALs</td></tr>
<tr><td>Initial Serviceability</td><td colspan="2">4.5</td></tr>
<tr><td>Terminal Serviceability</td><td colspan="2">2 on Local Roads
2.5 on Arterials and Collectors</td></tr>
<tr><td>Reliability Level %</td><td colspan="2">95% on Interstate
95% on NHS
90%-95% on Rural State Routes: look at traffic volumes
85%-95% on Urban State Routes: Look at traffic volumes, turning movements, braking movements.
85% on Local Roads</td></tr>
<tr><td>Overall Standard Deviation</td><td colspan="2">0.45</td></tr>
<tr><td rowspan="6">Roadbed soil Resilient Modulus</td><td>Given Soil Support</td><td>M_r (psi)</td></tr>
<tr><td>3.0</td><td>2800</td></tr>
<tr><td>3.5</td><td>3600</td></tr>
<tr><td>4.0</td><td>4300</td></tr>
<tr><td>4.5</td><td>5100</td></tr>
<tr><td>5.0</td><td>6100</td></tr>
<tr><td>Staged Construction</td><td colspan="2">1</td></tr>
<tr><td rowspan="5">Layer Coefficients</td><td>Top 4 inches (maximum) of pavement</td><td>0.44</td></tr>
<tr><td>Pavement below top 4 inches</td><td>0.34</td></tr>
<tr><td>Aggregate Sub base Course Gravel</td><td>0.09</td></tr>
<tr><td>Reclaim</td><td>0.14</td></tr>
<tr><td>Reclaim with additive</td><td>0.22</td></tr>
</table>

2.8.4.3 Local Roads

Pavement on approaches located on local roads can be designed using Table 2-9.

Table 2-9 Pavement & Subbase Thickness

Equivalent Daily 18 k Single Axle Application	Pavement Thickness (in)	Aggregate Subbase Course - Gravel (in)	Total Subbase and Pavement Thickness (in)
0-25	3	21	24
26-50	3	24	27
51-100	3	27	30
101-150	4	26	30
>150		Design According to Collector and Arterial Standards, using Terminal Serviceability = 2	

2.8.5 Approach Drainage

Well-drained pavements can outlast poorly drained ones by at least three to four times. When most subgrade soils are compacted sufficiently to support vehicle loads, their permeabilities are cut down to a level that allows only miniscule amounts of water to drain downward (Cedergren, 1989). Positive drainage of the pavement (through crowning) and subgrade is critical to the long-term performance of the roadbed. Total drainage design should be studied, with reference to the drainage section of the "MaineDOT Urban & Arterial Highway Design Guide".

In planning approach construction, the subgrade layer should be allowed to daylight on the foreslope of the roadway a minimum of 12 inches above the ditch line. If it is not possible to daylight the subgrade soils in this manner, consideration should be given to the use of an underdrain. Where underdrain is used, it must be positively drained away from the roadway.

Water should not be allowed to drain into the underside of slope protection. Permanent erosion control measures should be considered at the bottom of ditches.

2.8.6 General or Local Conditions

Good engineering judgment is required in all locations to determine the overall needs of the community by taking into consideration safety, future growth, and current needs. The Designer should also consider the geometric configuration of the corridor adjacent to the project during the design process. The design should reflect aesthetic, scenic, historic, and cultural considerations.

2.9 Structure Type Selection

A multitude of issues must be considered when the Structural Designer chooses the best structure type for a given project. The project team will contribute input according to each member's expertise.

2.9.1 Span

Span length will influence the optimal structure type and section to use. Spans less than 50 feet are discussed in Section 1.5 Small Bridge Initiative. Longer spans will generally be girder/deck bridges made of either steel or concrete. Rolled steel beams and precast, prestressed concrete box beams are used up to about 100 feet. Precast, prestressed concrete girders are used up to about 150 feet. Welded steel girders are used up to about 250 feet due to the practical limit of about 150 feet for shipping pieces. Longer spans will require steel girders with additional field splices, steel box girders, or segmental concrete girders.

The optimal span configuration will depend upon the cost of the proposed substructure units. Fewer piers will reduce the overall substructure cost, but will increase the span lengths and overall superstructure cost. Often the Structural Designer must balance the cost of the superstructure with the cost of the substructure to determine the best design.

2.9.2 Maintenance of Traffic

If staged construction is planned, the Structural Designer must lay out the proposed traffic scheme to be certain the existing and proposed bridges can support the traffic. The configuration of the existing bridge girders must be examined to determine the width remaining to support traffic once some of the girders are removed. Precast deck panels may be preferred for staged construction projects due to faster construction times. On precast structures, the width of available precast units must be considered.

The ability of the proposed structure to support traffic before the structure is complete must also be explored. For example, a structural plate structure is very difficult to stage, due to difficulty connecting the plates in place, the need to temporarily reinforce the ends, and concerns about non-uniform backfill.

2.9.3 Constructability

The Structural Designer and Construction Resident must agree that the proposed structure can be constructed. This can be of particular concern on rehabilitation or staged construction projects. The sequence of construction and an acceptable method of construction of both the foundation and structure must be studied before submitting a considered design. In particular, adequate space must be available for the Contractor to perform the necessary work, and existing subsurface and stream conditions must be carefully examined. Difficulty in construction of substructure units due to site conditions may favor the use of longer more expensive superstructure units. Other examples of common constructability issues include the method of cofferdam construction, the use of mechanical couplers in tight spaces, and the limitation of commonly used forms in the construction of a wide slab overhang.

As one form of scour protection, consideration should be given to the practice of leaving the sheet piling used for cofferdams in place and cutting them off at the streambed elevation after construction is complete. Refer to Section 5.2.3 Cofferdams.

2.9.4 Environmental Impact

The goal when applying for environmental approvals is always to avoid or minimize environmental impacts. The Structural Designer often must balance this reduction of impacts with the additional cost that may be added to the project. With this in mind, the Structural Designer will design a water crossing bridge long enough to minimize stream impacts. In some cases, tight in-stream work windows may force the design to stay out of the stream altogether. Return wingwalls and headwalls on culverts are used to minimize impacts to the stream and to adjacent wetlands. Reduced berm offsets are considered on local roads to keep toes of slope out of wetlands.

For culvert-type structures, attention must be given to the impact of the structure bottom on the stream. In some cases, environmental restrictions may force the Structural Designer to use a three-sided structure without a bottom instead.

2.9.5 Right-of-Way Impact

Whenever possible, the impacts to adjacent property owners should be kept to a minimum. Methods such as wingwalls and reduced berm offsets on local roads can be used. Other considerations include maintaining accessibility to homes and businesses during and after construction.

The cost of right-of-way issues can impact both the budget and schedule. The lengthy right-of-way process can cause project delays when people are displaced from acquired buildings. Dollar cost of property acquisition can also be high in some areas. The existence of gas stations, mills, or factories can herald the presence of hazardous materials that must be removed at significant cost.

2.9.6 Maintainability

Long-term maintenance is always part of the equation when determining the optimal structure type, and has influenced Bridge Program policy throughout this guide. For example, weathering steel has lower maintenance cost than painted steel. Policies have been developed for issues such as these by balancing first cost with maintenance cost. The Designer should always try to keep future maintenance costs as low as practical.

The Designer should keep the following in mind when choosing design options:

- Look at how the bridge will be maintained. Will high traffic volumes limit maintenance activities? Will maintenance be very expensive? If so, it will be even more important to design low frequency maintenance structures.

- Consider how parts of the bridge will be repaired, such as bearings. Is there room for temporary support? Is there adequate access? Catwalks should be considered around abutments and piers for large or extraordinary projects. The bearing seats for abutments and piers should be wide enough to accommodate jacks for future bearing replacements.

- Is the bridge wide enough to maintain traffic during deck repairs and wearing surface replacements? Is the approach wide enough where return wingwalls are used?

- Use standard sizes and coatings when possible to facilitate prompt repair with off-the-shelf items.

- Consider the need to remove winter sand from bridge seats and rails. Avoid designs that allow winter sand accumulation on bearings and beam ends.

- Consider under-bridge crane limitations for inspection. Vertical reach will limit fences to 6 feet high, and horizontal reach will limit sidewalks to 8 feet wide.

- Consider bridge width needs for snowplows to facilitate plowing and to limit potential damage in accordance with Section 4.1 Bridge Widths.

- Consider the need to inspect substructures for scour. If inspection is impossible due to high velocities, provide additional protection.

2.9.7 Historical/Archeological Issues

It is critical that any project that has historical or archeological interest is flagged early in the process. Working with the Maine Historic Preservation Commission (MHPC) and relevant historic districts as the design is developed will save considerable time in the process.

2.9.8 Cost

The Structural Designer should attempt to find the lowest cost option that satisfies the requirements of the applicable code, MaineDOT guidelines, and the traveling public, but does not sacrifice quality. First cost must be considered, as well as life cycle cost in some cases (refer to Section 2.2 Economic Comparisons). The program cost should be identified, and every attempt made to design a project that falls within that budget.

2.9.9 Aesthetics

The consideration of aesthetics in every design is encouraged. Often there are low cost methods that can be incorporated into a design that can greatly increase the aesthetic value of the project. Refer to Section 1.7 Aesthetics for more discussion.

References

AASHTO, 2001, *A Policy on Geometric Design of Highways and Streets*, Washington, DC

AASHTO, 2002, *Roadside Design Guide*, Washington, DC

AASHTO, 1996 and Interims, *Standard Specifications for Highway Bridges*, Sixteenth Edition, Washington, DC

American Railroad Engineering and Maintenance of Way Association (AREMA), 2002, Manual for Railway Engineering, USA

Bodhaine, G. L., 1968, Measurement of Peak Discharge at Culverts by Indirect Methods, *U.S.G.S. TWRI Book 3*, Chapter A3

Bridge Life Cycle Cost Analysis, Computer program, NCHRP Project 12-43

Cedergren, Harry A. 1967, 1977, 1989, *Seepage Drainage and Flow Nets*, Third Edition, John Wiley & Sons New York

DARWin™ Pavement Design System, 1993, AASHTO, Computer program

Federal Highway Administration, 2001, Bridge Scour and Stream Instability Countermeasures - Experience, Selection, and Design Guidance, *Hydraulic Engineering Circular No. 23 (HEC #23)*, Publication No. FHWA NHI-01-003, March

Federal Highway Administration, 1995, Evaluating Scour at Bridges, *Hydraulic Engineering Circular No. 18 (HEC #18)*, Publication No. FHWA-IP-90-017, November

Federal Highway Administration, 1991, Evaluating Scour at Bridges, *Technical Advisory T 5140.23*, October 28

Federal Highway Administration, 1985, Hydraulic Design of Highway Culverts, *Hydraulic Design Series No. 5 HDS*

Federal Highway Administration, *Hydraulic Engineering Circular No. 14*

Federal Highway Administration, 2001 Stream Stability at Highway Structures, Third Edition, *Hydraulic Engineering Circular No. 20 (HEC #20),* Publication No. FHWA NHI 01-002, March

Federal Highway Administration, Runoff Estimates for Small Rural Watersheds and Development of a Sound Design, *FHWA Reports - FHWA-RD-77-158 and 159*, Volumes I and II

Federal Highway Administration, 1967, Use of Riprap for Bank Protection, *Hydraulic Engineering Circular No. 11 (HEC #11)*

HEC-RAS, Computer program and User Manual, 1998, U. S. Army Corps of Engineers, September

Hodgkins, Glen, 1999, Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals, *U.S. Geological Survey, Water - Resources Investigations Report 99-4008*

HYDRAIN Version 6.1, Computer program and User Manual, 1999, Federal Highway Administration, March

MaineDOT, 2001, *Survey Manual*

MaineDOT Bridge Program, *Plan Development and Estimating Guide*

MaineDOT Bridge Program, *Project Management Guide*

MaineDOT Urban and Arterial Highway Program, 2001, *Urban & Arterial Highway Design Guide*, December

U.S Army Corps of Engineers, 1997, Document EM 1110-2-2902

WSPRO, Computer program and User Manual, 1998, Federal Highway Administration, June