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

Historical data from NOAA shows that the sea level along the Maine coast over the past 80-100 years has risen between 0.5 and 0.75 feet per 100 years relative to local datums. More detailed information is available from the NOAA Tides and Currents website in the Sea Level Trends section. Based on this historical data and NOAA projections, the proposed design should assume 4 feet of sea level rise per 100 years.

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 – ordinary high water (OHW) 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 where 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. The report that explains the 1999 USGS full regression equation is “Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals” by Glenn A. Hodgkins, published by U.S.G.S in 1999 and available from their website.

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.

At most sites, several factors affecting the roadway grade and hydraulic opening need to be considered. These factors generally fall into two categories:

<u>Impacts</u>

- Property impacts
- Wetland impacts
- Historical or archaeological impacts
- Marine traffic
- Constructability
- Cost

<u>Risk</u>

- Importance of the roadway
    - Corridor Priority
    - MEMA Evacuation Route
- Remaining bridge service life
- Accessibility of the bridge during flood events
- Feasible detour routes during flood events

There may be instances where meeting the minimum design criteria is unreasonable due to these factors. If the minimum design criteria is not met with the proposed design, the PDR should state and discuss the difference. In all cases, the reasoning and factors involved in the design process should be clearly documented in the PDR.

At no time should the design criteria be considered a substitute for careful evaluation of site specific factors and good engineering judgment. The level of risk allowed will vary based on site conditions. At some sites, maintaining clearances will be paramount regardless of the impacts; at others, accepting a higher level of risk may be the appropriate option.

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.

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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 equal to or less than 0.9 at Q50 with flow at MHW under inlet control conditions.

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 on MHW with sea level rise), including wave heights.

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.

B. Wearing Surfaces

Refer to Section 4.7 Membranes for membrane requirements under pavement. Concrete wearing surfaces should be avoided unless a minimum 6 inch composite leveling slab is used.

C. Leveling slabs

In general, a reinforced composite slab should be used on all voided slab and butted box beam structures, with a minimum thickness of 4.5 inches at the curb line and a cross slope that matches the finished slope.

In some cases, an unreinforced leveling slab may be used, when approved by the Engineer of Design. The minimum thickness is 2 inches at the curb line, and the cross slope matches the finish slope. In rare cases, the concrete slab may be omitted based upon project specific considerations.

D. Continuity Design

Prestressed girders should be made continuous for the maximum practical length to avoid expansion joints. In general, the design should follow AASHTO LRFD Section 5.14.1.2 - Precast Beams. The Structural Designer is also referred to Oesterle (1989).

1. Negative Moment Over Piers

As a minimum, sufficient continuity steel should be provided to control cracking at the pier in the wearing surface at service loads. Crack control should be checked in AASHTO LRFD Section 5.7.3.4. The following values should be used for the crack width parameter Z:

Bituminous with high performance membrane	170 k/in
Concrete wearing surface*	77 k/in

*A crack width parameter up to Z = 130 k/in may be allowed with the use of galvanized or epoxy coated reinforcing steel and low permeability concrete.

Crack width parameters of 170 and 77 k/in correspond to approximate crack widths of 0.016” and 0.007” respectively. More refined methods of determining crack width such as the Gergely-Lutz equation for crack width are allowed.

2. Positive Moment Over Piers

As a minimum, sufficient continuity steel should create a reinforced section that resists 1.2 times the cracking moment.