

**STATE OF NEW HAMPSHIRE  
DEPARTMENT OF TRANSPORTATION**

# Bridge Design Manual

**PART I - DESIGN GUIDELINES & PLATES**



**BUREAU OF BRIDGE DESIGN**

October 1, 2000

NOTICE TO AGENCIES OUTSIDE THE NHDOT

*THE BRIDGE DESIGN MANUAL - PART I* DOES NOT CONSTITUTE A SPECIFICATION AND IS INTENDED SOLELY AS A GUIDELINE. IT SHOULD NOT BE USED IN ANY MANNER WHICH REPRESENTS IT AS BEING A STANDARD OR POLICY OF THE STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION.

NHDOT BRIDGE DESIGN MANUAL  
PART I - DESIGN GUIDELINES & PLATES

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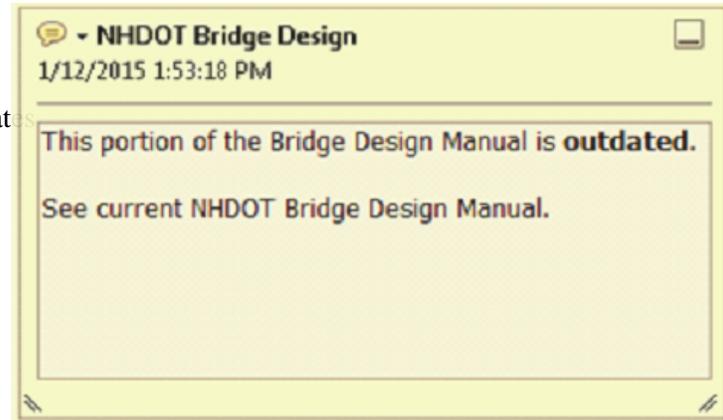
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## PART II - BRIDGE DESIGN STANDARD DETAIL SHEETS

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### 101 General

The *Bridge Design Manual* has been prepared by the Bureau of Bridge Design, Department of Transportation, State of New Hampshire and is comprised of two parts:

- Part I - Design Guidelines & Plates*
- Part II - Standard Detail Sheets*

#### 101.1 Part I - Design Guidelines & Plates

The *Bridge Design Manual Part I - Design Guidelines & Plates* is a working document for use as a guide in the design of bridges and related structures and in the preparation of contract plans. *Part I* includes design guidelines, plates, and administrative procedures intended to promote consistency and continuity of design work practices. *Part I* does not constitute a specification or contract document.

*Part I* promotes uniformity of practice and represents the current best thinking of the Bureau, yet at the same time permits the Engineer to exercise discretionary judgment in its implementation and provides for the incorporation of new ideas. Each Bureau member is encouraged to participate in keeping this document current as design practices change and improve. A departure from the guidelines should be recorded and approved in writing in the permanent project records. *Part I* should be reviewed and revised periodically as necessary.

"Design guidelines" are herein defined as written procedures, instructions, practices, and "rules-of-thumb" used by the Bureau in the design of bridges and transportation related structures during the preparation of contract plans.

"Plates" are herein defined as written details, drawings, sketches, tables, charts, and notes which reflect Bridge Design typical practice. Plates are used in the design process, contribute to the preparation of plans, may be incorporated directly, or in part, into contract plans but are not designated as Standard Detail Sheets because of their size, job-specific nature, or other features which make them unsuitable to be Standard Detail Sheets.

"Administrative Procedures" are herein defined as non-binding means and methods of conducting the internal administrative business of the Bureau of Bridge Design in the development of contracts, design of bridges and structures, organization of responsibilities, and other information involved in administrative tasks unless approved otherwise by the Design Chief and/or Administrator.

## **101.2 Part II - Standard Detail Sheets**

*The Bridge Design Manual Part II - Standard Detail Sheets* is a compilation of standard detail sheets which are herein defined as plan sheets with information generically applicable to many projects, which have been approved by the Bureau Administrator, and are intended for incorporation directly into contract documents with only minor revisions.

# **110 General Design Provisions**

## **110.1 Design Criteria**

Bridges and transportation related structures should be designed in accordance with the latest edition of the following specifications:

1. AASHTO *Standard Specifications for Highway Bridges*;
2. NHDOT *Standard Specifications for Road and Bridge Construction*.

## **110.2 Design Methods**

All superstructures and substructures should be designed in accordance with the Strength Design Method (LFD) of the AASHTO Specifications, unless noted otherwise.

## **110.3 Permanent Records**

Design and check calculations for bridges and structures should be made and recorded independently by different individuals. These calculations should be placed on file in the permanent project record folder. Calculations and reports should be legible, clearly marked, complete, and bound or stapled. The permanent project record folder should be placed in the "Active Project" file cabinet until construction is completed.

## 120 Reference Publications

The following publications, as revised and amended, may be referenced in this manual and are available within the Bureau for use in the design of bridges, related structures, and highways:

#	Reference Publication	Reference In Manual
1.	AASHTO A Policy on Geometric Design of Highways and Streets	430, 560
2.	AASHTO Bicycle Guide	<a href="#">691</a>
3.	AASHTO Construction Handbook for Bridge Temporary Works	<a href="#">612</a>
4.	AASHTO Guide Design Specification for Bridge Temporary Works	612
5.	AASHTO Guide for Bridge Management Systems 1993	<a href="#">920</a>
6.	AASHTO Guide for Fatigue Evaluation of Existing Steel Bridges	<a href="#">680</a>
7.	AASHTO Guide Specification for Fatigue Evaluation of Existing Steel Bridges	680
8.	AASHTO Guide Specifications for Bridge Railings, 1989	<a href="#">642</a>
9.	AASHTO Standard Specification for Structural Supports for Highway Signs, Luminaires and Traffic Signals	<a href="#">710</a>
10.	AASHTO Standard Specifications for Highway Bridges	(default spec)
11.	AASHTO Standard Specifications for Highway Bridges - Div 1-A, Seismic Design	<a href="#">603</a>
12.	AISC Modern Steel Construction Magazine	<a href="#">630</a>
13.	AISI Integral Abutments for Steel Bridges	<a href="#">613.5</a>
14.	AITC Timber Construction Manual	<a href="#">653</a>
15.	Americans with Disabilities Act	<a href="#">401</a>
16.	ANSI/AASHTO/AWS D1.5 Bridge Welding Code	630, 840
17.	AREMA Manual for Railway Engineering	<a href="#">692</a>
18.	AWS A3.0 Welding Terms and Definitions	<a href="#">260</a>
19.	American Wood Preserver's Association	<a href="#">653</a>
20.	FHWA Hydraulics Engineering Circular (HEC 18, HEC 20, HEC 23)	<a href="#">520</a>
21.	FHWA National Bridge Inspection Standards	<a href="#">430</a>
22.	FHWA Policy Memorandum, August 13, 1990 (on Bridge Rail)	<a href="#">642</a>
23.	FHWA Policy Memorandum, August 28, 1986 (on Bridge Rail)	642
24.	FHWA Position Paper, May 14, 1996 (on Bridge Rail)	642
25.	FHWA RD-77-159 Runoff Estimates for Small Rural Watersheds	<a href="#">440</a>
26.	FHWA RD-77-4 End Connections of Pretensioned I-Beam Bridges	<a href="#">620.4</a>
27.	FHWA RD-94-049 Analysis of Transfer and Development Lengths for Pretensioned Concrete Structures	620.4
28.	FHWA Recording & Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges (a.k.a. National Bridge Inventory)	<a href="#">910</a>
29.	FHWA SA 97-006 to 012 Seismic Design of Bridges - Design Examples	<a href="#">603</a>
30.	FHWA Seismic Design of Highway Bridges	603
31.	FHWA Technical Advisory on Weathering Steel, Oct 3, 1989	<a href="#">630</a>
32.	NCHRP 230 Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances	<a href="#">642</a>
33.	NCHRP 239 Multiple Service Level Highway Bridge Railing Selection Procedures	642
34.	NCHRP 271 Guidelines for Evaluation and Repair of Damaged Steel Bridge Members	<a href="#">960</a>
35.	NCHRP 321 Weld Repair of Cracks in Steel Bridge Members	960
36.	NCHRP 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features	642

37. NH RSA 234:2	<a href="#">430</a> , <a href="#">680</a> , <a href="#">1220</a>
38. NHDOT Consultant Selection Procedures	<a href="#">1150</a>
39. NHDOT Employee Health & Safety Program	<a href="#">1101</a>
40. NHDOT Highway Design Manual	<a href="#">401</a>
41. NHDOT Manual on Drainage Design for Highways	<a href="#">440</a>
42. NHDOT Policies and Procedures	<a href="#">1101</a>
43. NHDOT Public Involvement Procedures for NH Transportation Improvement Projects	<a href="#">310</a>
44. NHDOT ROW Public Hearings (yellow book)	<a href="#">480</a>
45. NHDOT Utility Accommodation Manual	<a href="#">940</a>
46. NHDOT Wetland Manual Permit Process	<a href="#">550</a>
47. PCI Journal April 1969 (Continuity Guidelines)	<a href="#">620.4</a>
48. PCI Precast Prestressed Concrete Bridge Design Manual, Vol. I & II	620
49. Soil Conservation Service Technical Release #20	440
50. Soil Mechanics, Terzaghi & Peck	<a href="#">614</a>
51. Steel Sheet Piling Design Manual, United States Steel	<a href="#">830</a>
52. US Army Corps of Engineers, Ice Engineering	<a href="#">615</a> , 830
53. US Steel Stringer Design Handbook	<a href="#">612</a>
54. Washington State Concrete Repair	<a href="#">960.2</a>

## 121 Design Resource Publications

The following publications are not referenced in this manual but are available in the Bureau as a resource for use in the design of bridges, related structures, and highways:

### # Design Resource Publications

1. AASHTO Guide for Commonly Recognized (CORE) Structural Elements 1991
2. AASHTO Guide For Fatigue Design of Steel Bridges 1989
3. AASHTO Guide for Maximum Dimensions and Weights of Motor Vehicles and the Operation of Non Divisible load Oversize and Overweight Vehicles
4. AASHTO Guide for Painting Steel Structures 1997
5. AASHTO Guide for the Development of Bicycle Facilities 1991
6. AASHTO Guide for Transportation Landscape and Environment Design 1991
7. AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges 1991
8. AASHTO Guide Specification for Design and Construction of Segmental Concrete Bridges, 1989
9. AASHTO Guide Specification for Design of Pedestrian Bridges
10. AASHTO Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridge using Braced Compact Sections 1991
11. AASHTO Guide Specifications for Aluminum Highway Bridge 1991
12. AASHTO Guide Specifications for Design of Pedestrian Bridges 1997
13. AASHTO Guide Specifications for Distribution of Loads for Highway Bridges 1994
14. AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members
15. AASHTO Guide Specifications for Horizontally Curved Highway Bridges
16. AASHTO Guide Specifications for Seismic Isolation Design 1999
17. AASHTO Guide Specifications for Shotcrete repair of Bridges 1998
18. AASHTO Guide Specifications for Strength Design of Truss Bridge (Load Factor Design) 1985
19. AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges 1989
20. AASHTO Guide Specifications for Structural Design of Sound Barriers 1989
21. AASHTO Guide Specifications for the Design of Stress-Laminated Wood Decks 1991
22. AASHTO Guide Specifications Thermal Effects in Concrete Bridge Superstructures 1989
23. AASHTO Guidelines for Preconstruction Engineering Management 1991
24. AASHTO LRFD Bridge Construction Specifications 1998
25. AASHTO Manual for Condition Evaluation of Bridges 1994
26. AASHTO Standard Specifications for Movable Highway Bridges
27. AISC Marketing, Inc. Highway Structures Design Handbook, Vol. I and II
28. ARTBA A Guide to Standardized Highway Barrier Rail Hardware
29. FHWA Bridge Inspector's Training Manual, 1979 & 1999
30. Finnish Design Code
31. NHDOT Construction Manual
32. Sectional Plate Handbook, Republic Steel
33. Specifications for Aluminum Bridge and Other Highway Structures, the Aluminum Association

## 122 Abbreviations

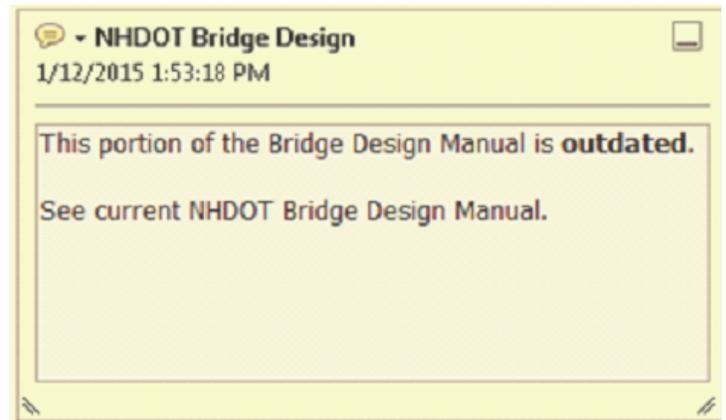
The following terms and abbreviations are used in this manual (see also 210.5):

#	<u>Abbreviation</u>	<u>Term</u>	<u>Reference In Manual</u>
1.	AASHTO	American Association of State Highway and Transportation Officials	default spec
2.	ACI	American Concrete Institute	620
3.	ACOE	Army Corps of Engineers	440, 1051
4.	ADT	Average Daily Traffic	690
5.	AISC	American Institute of Steel Construction	630
6.	AISI	American Iron and Steel Institute	613
7.	AREMA	American Railway Engineering and Maintenance-of-Way Association	692
8.	AWPA	American Wood Preservers Association	653
9.	AWS	American Welding Society	630
10.	CIP	Cast-In-Place	810
11.	DRED	Department of Resource and Economic Development	670, 691
12.	DTI	Direct Tension Indicator	630
13.	EPA	Environmental Protection Agency	630, 950
14.	EQF	Modified foundation seismic forces (earthquake foundation)	603
15.	EQM	Modified seismic forces (earthquake modified)	603
16.	GACIT	Governor's Advisory Commission on Intermodal Transportation	310
17.	HS	High Strength	810, 1050
18.	LFD	Load Factor Design	110, 602, 610
19.	LRFD	Load Resistance Factor Design	615
20.	MPO	Metropolitan Planning Organizations	310
21.	MSE	Mechanically Stabilized Earth	603, 614
22.	MSL	Multiple Service Level	642
23.	NDS	National Design Specification for Wood Construction	563
24.	NEPCOAT	Northeast Protective Coatings Committee	630
25.	NHS	National Highway System	642, 654
26.	NSBA	National Steel Bridge Alliance	630
27.	OSHA	Occupational Safety & Health Administration	630
28.	PCI	Precast/Prestressed Concrete Institute	620
29.	PL	Performance Level	642
30.	QC/QA	Quality Control / Quality Assurance	614
31.	ROW	Right of Way	310, 320, 480
32.	RPC	Regional Planning Commissions	310
33.	SHPO	State Historical Preservation Office	970
34.	SPC	Seismic Performance Categories	603
35.	SPGP	State Program General Permit	1051
36.	SSPC	The Society for Protective Coatings	630
37.	STIP	State Transportation Improvement Program	310
38.	TL	Test Level	642
39.	USCG	United States Coast Guard	1054
40.	VOC	Volatile Organic Compounds	630
41.	WSD	Working Stress Design	610
42.	MIO	Micaceous Iron Oxide	630.3.4
43.	NEBT	New England Bulb Tee	620

44. TCP	Traffic Control Plan	320
45. PS&E	Plans, Specifications, and Estimate	320
46. POW	Prosecution of Work	320
47. DBE	Disadvantaged Business Enterprise	320
48. DHV	Design Hourly Volume	430
49. AADT	Average Annual Daily Traffic	430
50. ADT	Average Daily Traffic	430
51. FHWA	Federal Highway Administration	
52. S POINT	Survey Point	510.1
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## SECTION 200 - PREPARATION OF PLANS

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### 201 General

The efforts of the Department are judged to a great extent by the clarity and neatness of its Contract Drawings. The need always exists for attention and care in this matter. It is the intent of the Department to insist on clarity and high standards of neatness on all drawings.

Of equal importance is the usability of the electronic submissions which the Bureau receives from Consultants. Refer to current "CAD/D Procedures and Requirements" manual

### 210 Graphic Guidelines

The following instructions should be followed:

#### 210.1 Line Styles

1. Dimension lines and grade elevations should be placed to avoid the lines of the drawing itself, wherever possible.
2. Intersecting dimension lines, extension lines and leaders should be broken.
3. Place leaders from notes at the beginning or end of the note blocks only.
4. Line styles should be chosen so that the primary subject of the drawing is in a bold line style. For example, on a reinforcing detail the reinforcing bar should be shown in a bold line style, and the masonry should be shown in a lighter line style.

## **210.2 Character Styles**

Refer to current "CAD/D Procedures and Requirements" manual.

## **210.3 Symbols**

1. On English drawings, use foot and inch marks on all dimensions.
2. On metric drawings, use meters and millimeters for all dimensions.
3. Label and Dimension in accordance with NHDOT Metric Conversion Guide (Appendix A).
4. Show a North Arrow to orient every Plan View.

## **210.4 Metric**

1. On metric drawings, the following note should appear on the front sheet:

ALL DIMENSIONS ON THE PLANS ARE SHOWN IN MILLIMETERS OR METERS.  
WHOLE NUMBERS INDICATE MILLIMETERS AND DECIMAL NUMBERS INDICATE  
METERS, UNLESS OTHERWISE NOTED.

2. On metric drawings, a bold rectangle with the word "METRIC" (minimum character height being 11 mm) contained therein, should appear on each sheet. (See "stamps" cell library)
3. Scales for full size drawings must be compatible with a usable scale on half-size drawings. (See current "CAD/D Procedures and Requirements" manual)

## 210.5 Abbreviations

The following abbreviations should be used on bridge plans where use of the abbreviations aids the flow of the sheet. Abbreviations should not be used in the text of notes unless they are conventional abbreviations, such as mm for millimeters or HS Bolt for High Strength Bolt.

<u><b>A</b></u> Abutment Alternate, Alternating And Approximate At	ABUT ALT & APPROX @	<u><b>F</b></u> Far Side Finished Grade	FS FG
	<u><b>B</b></u> Bearing Bench mark Bottom	BRG BM BOT	<u><b>G</b></u> Galvanized Gauge
<u><b>C</b></u> Centerline Clearance, Clear Concrete Construction Center to Center		$\text{¢}$ CLR CONC CONST C-C	<u><b>H</b></u> Hexagonal Head High Strength Highway Horizontal
	<u><b>D</b></u> Degrees, Minutes, Seconds (angular) Degrees (thermal) Diameter	-XX°-XX'-XX" °C (°F) $\text{Ø}$	<u><b>I</b></u> Inside Diameter Interior
<u><b>E</b></u> Each Each Face Elevation = Expansion Existing Exterior		EA EF EL = EXP EXIST EXT	<u><b>J</b></u> Joint
	Each Each Face Elevation = Expansion Existing Exterior	EA EF EL = EXP EXIST EXT	<u><b>K</b></u> Kilometers Hour
<u><b>L</b></u> Linear Feet Longitudinal Linear Meter			LF LONGIT Lm



4. Show the structure footing outline on the Boring Plan.
5. Include a Survey Layout in the plans. Orient Working Points so that they increase from left to right as the structure is being faced. Coordinates of Working Points shall be included with the survey layout. See Plate 210.6.
6. Normally, lay out the Plans and Profiles with stations increasing from left to right. See Plate 210.6.
7. Orient masonry cross-sections in the direction of the ahead station.
8. All dimensions should be aligned with the dimension lines so that they may be read from the bottom or from the right side of the sheet.
9. Details, enlargements, and blowups should be oriented the same as the original.
10. Title the quantity box on the plans as follows: "SUMMARY OF BRIDGE QUANTITIES" and, where applicable: "SUMMARY OF ROADWAY QUANTITIES".

## **220 Plans for Public Meetings**

### **220.1 General**

Plans for Public Hearings shall be kept as a matter of public record.

### **220.2 Colors**

The color codes for objects on drawings for public meetings should conform to the legend used by the Department. The legend is available in the "Legends" cell library.

### **220.3 Scale**

English plans should be to a scale of 1 in = 50 ft, Metric Plans should be to a scale of 1:500 unless otherwise directed. An exaggerated profile should be used unless otherwise directed. (Vertical scale, 1 in = 10 ft English, 1:100 metric; Horizontal scale, 1 in = 50 ft English, 1:500 metric).

## 230 Sequence Of Drawings

The sequence of drawings in a project should normally be:

1. General Plan & Elevation (Summary of Quantities)
2. Note Sheet
3. Site Plan & Profile (Survey Layout)
4. Channel Layout and Sections
5. Construction Access and Grading Plan
6. Boring Layout and Boring Logs
7. Footing Masonry (Plan & Sections, Excavation Limits, etc.)
8. Footing Reinforcement
9. Abutment (Frame) Masonry
10. Abutment (Frame) Reinforcement
11. Wing (MSE Wall) Masonry
12. Wing (MSE Wall) Reinforcement
13. Pier Masonry (Footing Plan, Pier Elevation, Sections, etc.)
14. Pier Reinforcement
15. Bridge Shoes
16. Superstructure Framing Plan, Details, Notes
17. Typical Deck Section, Details, Notes
18. Deck Reinforcement
19. Deck Panels
20. Approach Slabs
21. Fixed & Expansion Joint Details
22. Rail & Curb Layout
23. Bridge Rail
24. Bridge Approach Rail
25. Lighting
26. Concrete Barrier
27. Reinforcing Schedules

## 240 Quantities

1. A Summary of Bridge Quantities should appear on the General Plan and Elevation Sheet.
2. Do not include intermediate quantity summaries in the contract plans (e.g. Abutment Quantities, Superstructure Quantities). Quantity calculation sheets shall include intermediate quantity summaries. Roadway Quantity sheets shall be included in the Contract Plans when directed.

## 250 Concrete Drawings

1. Reinforcement layout: Avoid "±" spacings. Lay out reinforcing steel to even spaces, preferably standard spacings [e.g.: 6 in (125 mm), 1 ft.-0 in (250 mm), etc.]. Label odd dimensions at the joints or ends of the wall, footing, etc.
2. List reinforcement bars separately for different portions of the structure (e.g. abutments, piers, decks, etc.) on the reinforcing schedule sheets of the Contract Plans.
3. Try to use reinforcing bar letter prefixes which will locate the bar in the structure (e.g. Abut. A Footing, AF; Abutment B, B; Northwest Wing, NW; Deck, D; Approach Slab, AS; Frame, F; Pier, P; Retaining Wall, RW; etc.).
4. Reinforcing steel should be designated by bar number (e.g. NW1, NW2, NW3, etc.).
5. Indicate reinforcing bars with heavy solid lines (MicroStation line weight of 3). Show masonry lines on the reinforcing sheets with a lighter linestyle (MicroStation line weight of 1).
6. Show all exposed corners and edges of concrete chamfered 3/4 in (20 mm) by note, when chamfer is specified.
7. Show the required lap of reinforcing bars for each portion of the bridge (e.g. footing, abutment, deck, etc.) on the first reinforcing steel drawing for each portion.
8. Concrete dimensions are given to the nearest 1/8 in (1 mm).

## 260 Structural Steel Drawings

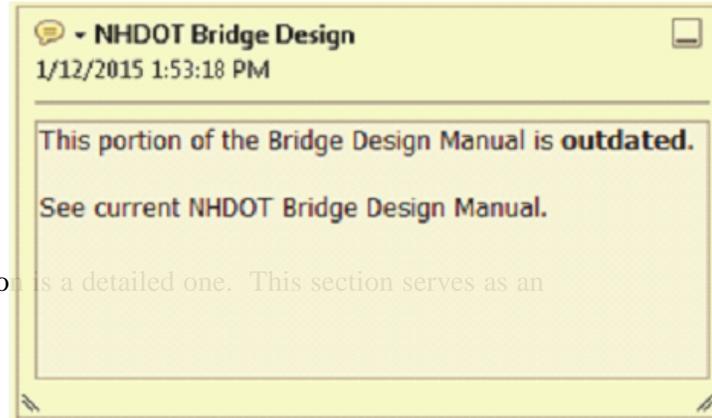
1. On structural steel drawings, avoid duplication in the designation of components. Give the complete designation for each component only once, usually in a major view, then use an abbreviated designation to locate the part in all other sections, details, etc.
2. Use current AISC symbols for plates, shapes, bars, etc.
3. Use current AWS symbols for welds.
4. Steel dimensions are given to the nearest 1/16 in (1 mm).

## **270 Changes to Contract Plans**

Changes to Contract Plans after a project has been advertised for bids shall be made by crossing out or circling portions of a drawing or, if appropriate, by superseding and replacing, or supplementing a drawing. No erasures are permitted. On the electronic copy of a drawing, no graphics are to be deleted; instead, mark or draw lines through the graphics which require revision. Make changes to the office copy of Contract Plans as well as reinforcing steel schedules, where appropriate. Revisions should be noted in the Revisions After Proposal box. Six full size and six half-size sheets shall be provided to Construction after bids are submitted. If the Contract Plans were developed by Consultants, one full size and one half-size sheet shall be forwarded to them.

# SECTION 300 - PROJECT DEVELOPMENT

- 301 General
- 310 Project Initiation & Authorization
- 320 Project Development Chart



## 301 General

The process to get a project from inception to construction is a detailed one. This section serves as an overview of the tasks involved in this process.

## 310 Project Initiation & Authorization

The Department, in accordance with State statutes, maintains a Ten Year Transportation Improvement Program for highway projects. Projects are added to this 10-year plan through input from the Bureau of Bridge Design and the Bureau of Bridge Maintenance and a review process which includes the Executive Office, the public (through MPOs (Metropolitan Planning Organizations) and RPCs (Regional Planning Commissions)), GACIT (Governor’s Advisory Commission on Intermodal Transportation), and the Governor and State Legislature. Projects contained in the first three years of each funding category constitute NH’s State Transportation Improvement Program (STIP). The RPC/MPOs and FHWA (Federal Highway Administration)/FTA (Federal Transit Administration) approve the three year STIP (TIPs at the local level). The process of developing each 10 year plan takes two years to complete, at which time it begins again. This process is described in the publication Public Involvement Procedures for New Hampshire Transportation Improvement Projects available from the Bureau of Transportation Planning. Three to five years before a project’s scheduled advertising date, the Design Chief will request Project Authorization (for Bridge projects with Bridge Design as lead Bureau) so that work necessary to develop contract plans may commence.

For federally funded projects, a Project Agreement Estimate authorizing PE & ROW money must be run prior to starting preliminary engineering. This money needs to have been included in the STIP in order for the estimate to be processed. If money has not been provided in the STIP, the Design Chief should notify Project Programming in Transportation Planning so that an amendment to the STIP can be requested.

## 320 Project Development Chart

The tasks required to prepare contract documents require attention to detail and thoroughness with procedures. They are listed on the following pages and are included in checklist form in Plates 320.1 through 320.8. Projects are tracked using an Access database, which can be located at s:\bridge.mdb.

## PROJECT DEVELOPMENT TASK LIST

### **A Project Start-Up**

- 1 Project Authorization
- 2 Initial Proj. Agreement Est. for PE and ROW
- 3 Site Visit
- 4 CAD/D User Account

### **B Type, Size, & Location (TS&L)**

- 1 Initial Survey Request
- 2 Survey Plotted
- 3 Detail Plan Developed
- 4 Detail Plan Checked
- 5 Contour Model Plan Developed
- 6 Contour Model Plan Checked
- 7 Traffic Data Request
- 8 Detail Plan to ROW for Title Abstracting if Hearing is Anticipated or Tax Map Level if Hearing is not Anticipated
- 9 Environmental Orange sheet & Detail Plan
- 10 Detail Plan to Utilities for Verification
- 11 Preliminary Hydraulic Study
- 12 Bridge Deck Evaluation Request
- 13 Pavement Recommendation
- 14 Paint Evaluation (ABC Survey)
- 15 Grade Control Request (from Highway)
- 16 Grade Control Elevation to Highway
- 17 Scope of Work Developed
- 18 TS&L Developed
- 19 TS&L Presentations to Exec Office
- 20 Design Exceptions
- 21 TS&L to Environment (Resource Agency Mtg)
- 22 TS&L to Town
- 23 Send Rolled Mylar to ROW for abstracting (Only include existing detail)
- 24 Preliminary Cost Estimate Update
- 25 Public Officials Meeting
- 26 Environment Green sheet
- 27 Public Informational Meeting
- 28 Hearing Plan Developed
- 29 Hearing Plan Checked
- 30 Public Hearing

### **C Preliminary Plans**

- 1 Boring Request
- 2 ROW Purchase Plans
- 3 Line & Grade Received
- 4 Finalize Hydraulic Report
- 5 Received Borings/Plotted
- 6 Develop Preliminary Plans
- 7 Preliminary Plans Checked
- 8 Preliminary Plan Review/Administrator
- 9 Preliminary Plans to Highway
- 10 Preliminary Plans to Bridge Maintenance
- 11 Preliminary Plans to Materials & Research
- 12 Preliminary Plans to FHWA
- 13 Preliminary Plans to Neighboring State
- 14 Prelim Plans to Utilities for Coordination

### **D Permits**

- 1 NH Wetlands Application
- 2 US Army Corps of Engineers
- 3 US Coast Guard

PROJECT DEVELOPMENT TASK LIST  
(continued)

**E Final Contract Plans**

- 1 Contract Plans Developed 60%
- 2 ROW Purchase Plans
- 3 Draft Prosecution of Work
- 4 Draft Traffic Control Plan
- 5 Prel Plans to Constr'n with Draft POW & TCP
- 6 Prel Plans to District Engineer
- 7 Prel Plans to Traffic for Sign Package &  
Temporary Signals
- 8 Informal Coordination w/ Hwy Design
- 9 60% Prel Plan Coordination Mtg
- 10 Checking Plans
  
- 11 Special Provisions Written
- 12 Contract Plans 90%
- 13 Printing Request to Print Shop
- 14 Plans to Construction
- 15 Quantities
- 16 Quantities Checked
- 17 (90%) Issues Meeting
- 18 PS&E Estimate
- 19 Contract Plans Completed
- 20 Plans to FHWA
- 21 Plans to City or Town
- 22 Plans to Neighboring State
- 23 Plans to Highway Design
- 24 Final Director's Review
- 25 Plans Stamped & Signed
- 26 Proposal Package to Specification Section
- 27 Requisition for Plans with Plans to Print Shop
- 28 Certification Of Coordination w/ Utilities
- 29 Complete Bridge Rating Form (Form 4)
- 30 PS&E Package to Planning

**F Project Advertisement**

- 1 Distribute Plans

**G After Bid**

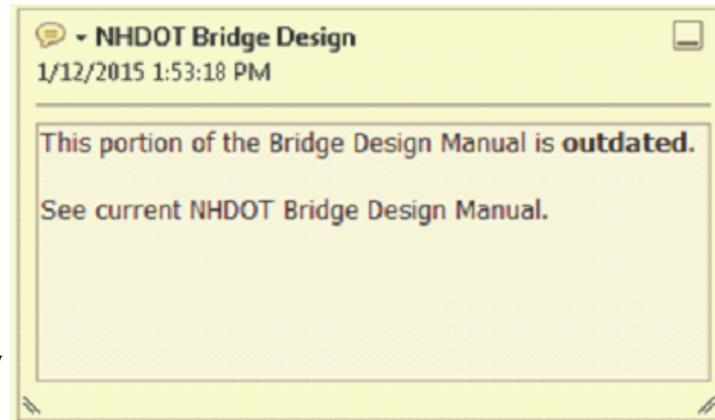
- 1 Bids Opened (Contractor/Item Bid Total)
- 2 Project Card (Vertical File Card; PS&E,  
Project Agreement and ABC-Bid Estimates in  
Estimate Binder)
- 3 Complete Bridge Flat Card
- 4 Project Agreement Estimate
- 5 Final ROW Plans (Mylar) for recording

**H Working Drawings**

- 1 Cofferdam
- 2 Deck Falsework
- 3 Bridge Rail
- 4 Bridge Approach Rail
- 5 Structural Steel
- 6 Bridge Shoes
- 7 Expansion Joint
- 8 Temporary Bridge
- 9 Erection Procedures

## SECTION 400 - BRIDGE TYPE SELECTION

- 401**            **General**
- 410**            **Site Visit**
  - 410.1           Scope of Work
  - 410.2           Traffic Control
  - 410.3           Hydraulics
  - 410.4           Survey Limits
  - 410.5           Pictures
- 420**            **Information Requests**
  - 420.1           Survey
  - 420.2           Traffic Data and Accident History
  - 420.3           Pavement Evaluation
  - 420.4           Bridge Deck Evaluation
  - 420.5           Paint Condition Evaluation
  - 420.6           Utilities
  - 420.7           Right-Of-Way
- 430**            **Bridge Geometry**
- 440**            **Preliminary Hydraulic Study**
  - 440.1           Hydraulic Calculations
  - 440.2           Waterway Opening
  - 440.3           Non-Bridge Structures
- 450**            **Conceptual Bridge Type Selection**
  - 450.1           Bridge Type Selection
  - 450.2           Sag Cambers
  - 450.3           Substructure
- 460**            **Conceptual Site Plan**
- 470**            **TS & L Review**
- 475**            **Estimate**
- 480**            **Public Meetings**
  - 480.1           Public Officials
  - 480.2           Public Information
  - 480.3           Public Hearing
  - 480.4           Display Equipment & Materials



## 401 General

The choice of a structure for a given site shall be the responsibility of the Design Chief with approval of the Administrator. The evaluation of bridge structure type should consider cost, constructibility, historic issues, aesthetics, safety, hydraulics, right-of-way, and environmental impacts. The cost factors should balance initial economy of the overall project as well as future maintenance. The structures evaluated should consider all superstructure and substructure options that are relevant for the site. Superstructure cost increases may be offset by substructure cost decreases such as using shorter abutments, which are set back from the feature crossed versus tall abutments.

The Project Engineer should endeavor to select the most serviceable structure while optimizing sight distance, design speed and clearance criteria at the proposed structure site. It is the general practice of the Department to design structures of girder-deck type construction. During the TS&L studies it is very important that the type of structure be approved before final profiles are set since the depth of the superstructure could greatly influence the roadway profile.

Many structures are now constructed at the same location as, or adjacent to, existing structures. Where substructures are in a suitable condition for re-use or can be rehabilitated to a serviceable condition, the merits and cost of re-using the existing substructure should be considered. If the existing substructure is not deemed serviceable for support of a new superstructure, the merits of saving it as an earth retaining structure should be considered.

Where the bridge structure is historic or is located in a historic area or in sensitive wetlands, the substructure may be retained without being incorporated into the new structure to minimize the impacts to these resources. The replacement bridge structure may be supported with piles driven behind and/or adjacent to the existing structures.

In selecting the type of structure, the following items should be considered:

1. Provide adequate clearance for the design flood.
2. Determine if the structure will be under fill.
3. Determine if a curved horizontal alignment will require curved girders.
4. Consider possible future widenings of the roadway under the bridge.
5. Provide a structure requiring minimum future maintenance.
6. Wherever possible, eliminate joints in the bridge deck.
7. Minimize environmental impacts.
8. Evaluate water control issues during construction.
9. Eliminate elements in the substructure that are a hazard to traffic.
10. Provide for maximum sight distance.
11. Provide a type of structure that is both functional and aesthetic.
12. Provide for placement of utilities in the superstructure as necessary.

13. For bridges with sidewalks, consideration needs to be given to adequate and safe access to both the bridge and its approaches for persons with disabilities. See the Americans with Disabilities Act Handbook.
14. Provide the required horizontal and vertical clearances in accordance with the appropriate drawings in Chapter 3 of the Highway Design Manual and in 430 of the Bridge Design Manual.
15. Continuous span design should be used whenever more than one span is required.

## **410 Site Visit**

The site visit provides an opportunity to visually examine and evaluate important aspects of the project.

### **410.1 Scope of Work**

Consider whether a structure is in need of widening, rehabilitation, or replacement.

### **410.2 Traffic Control**

If a detour around the project is required, it should be driven to verify that it meets current traffic requirements. The detour should be measured and accurately described.

### **410.3 Hydraulics**

A visual inspection should be conducted to determine the adequacy of the approach channel. Look for indications of the normal high water mark such as staining on abutments or piers or erosion along the channel embankments.

### **410.4 Survey Limits**

Establish the survey limits, using easily identifiable landmarks.

### **410.5 Pictures**

All noteworthy physical features of the project should be photographed.

## **420 Information Requests**

### **420.1 Survey**

The Bureau of Highway Design should be requested in writing to provide a survey of the site for Bridge Design initiated projects. For bridges crossing hydraulic channels the request should include a bridge grid for as much of the channel as will be necessary to perform the hydraulic design, including backwater and scour calculations if anticipated. In conjunction with this request, for water crossings, the Maintenance District should be requested, by way of the survey request, to fill out a Bridge Hydraulic Report which assists in establishing historic high water elevations and any hydraulic problems. (See Appendix C).

### **420.2 Traffic Data and Accident History**

The Bureau of Transportation Planning should be requested in writing to provide traffic data and an accident history when deemed appropriate for proposed Bridge Design initiated projects.

### **420.3 Pavement Evaluation**

The Bureau of Materials and Research should be requested in writing to provide an evaluation of the approach pavement to help determine the scope of approach work that will be necessary.

### **420.4 Bridge Deck Evaluation**

When deemed appropriate, the Bureau of Materials and Research should be requested in writing to provide an assessment of the existing bridge deck where deck rehabilitation is being considered.

### **420.5 Paint Condition Evaluation**

The Bureau of Materials and Research should be requested in writing to provide an Assessment of Bridge Coating (ABC) survey, which is an evaluation of the paint condition on existing steel beams, where bridge rehabilitation is being considered.

### **420.6 Utilities**

As soon as existing detail plans are available, the Design Services Engineer of the Bureau of Highway Design should be requested in writing to provide a list of all utilities that are within the project limits. (Use Utilities Request Form)

### **420.7 Right-of-Way**

As soon as existing detail plans are available, the Bureau of Right-of-Way should be requested in writing to provide complete title abstracting if a public hearing is anticipated (see 480.3) or tax map level abstracting if a public hearing is not anticipated. (Use Right-of-Way Request Form). This identifies the limits of the existing Right-of-Way and property ownership detail.

## 430 Bridge Geometry

The following features are to be determined:

1. Bridge/Roadway Width - The minimum width of bridge should be 24 ft (7.2 m) unless there are conditions that make this width extremely impractical (covered bridge or bridges servicing 1 or 2 households that would have severe impacts if 24 ft (7.2 m) width was constructed). Bridges on State Highways should use a typical minimum width of 30 ft (9 m).
2. Roadway cross slopes - For deck structures, if the shoulder width is 5 ft (1.5 m) or less, the cross slope of the travelled way should continue to the curb rather than match any breaks in the approach superelevation. For shoulder widths wider than 5 ft (1.5 m), the cross slope should be broken on the high side (only) of a superelevated deck section, to match the cross slope on the approaches.
3. Abutment or pier setbacks from roadways under the bridge per Chapter 3 of the Highway Design Manual clear zone requirements.
4. Preliminary span length(s).
5. Minimum roadway vertical clearances - Vertical clearance is measured between overhead structures and the finished roadway surface, or highest rail of the railroad. The designated minimum clearance must be provided over the entire usable roadway width including shoulders. The established minimum vertical clearances for New Hampshire are listed below.

### MINIMUM VERTICAL CLEARANCES

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Clearances should be shown on all profiles, both preliminary and final. (Clearances include an allowance for 6 inches for future paving.)

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Local road under Interstate with interchange	16 ft-6 in	(5.1 m)
Local road under Interstate without interchange	14 ft 6 in	(4.5 m)
Local road under all other roads	14 ft 6 in	(4.5 m)
Local road under railroads	14 ft 6 in	(4.5 m)
State Route under Interstate with interchange	16 ft 6 in	(5.1 m)
State Route under Interstate without interchange	14 ft 6 in	(4.5 m)
State Route under all other roads	14 ft 6 in	(4.5 m)
State Route under railroads	14 ft 6 in	(4.5 m)
Interstate Route under all roads	16 ft 6 in	(5.1 m)
Interstate Route under railroads	16 ft 6 in	(5.1 m)

Railroad under all roads (statutory\*)

22 ft 0 in

(6.7 m)\*

\*Exception -- Railroads normally require a vertical clear distance of 22 ft 6 in (6.9 m). All minimum clearances are from top of high rail to bottom of low edge of bridge. Check with the Chief of Design Services to verify required clearances when railroads are involved.

If site conditions will not allow these clearances for the railroad crossing to be achieved without considerable impacts, clearance may be reduced to 21 ft 0 in (6.4 m); lowering of the footing elevations should be investigated to allow for future lowering of the tracks by 1 ft-6 in (0.5 m) to achieve the 22 ft-6 in clearance.

6. Railroad vertical and horizontal clearances shall be coordinated with the Design Services Engineer. See Plate 430.1 for general guidelines.
7. Stations, angle of crossing, profiles, and typical approach roadway sections will be furnished by the Highway Design Bureau after the final line and grade determination. The stations and angle of crossing should be field measured and these measurements should be used on the contract plans.
8. Bridge grades should be 1.0% or more, when feasible, for rapid surface drainage and runoff of de-icing chemicals. In constrained areas (e.g. in urban location, or with environmental constraints such as wetlands, historical site, etc.) the minimum bridge grade should be 0.5%. For a symmetrical crest vertical curve the K value should be less than 105 for length of curve in feet (32 metric) to insure a minimum grade of 0.5% about 50 ft (15 m) from the crest.
9. Skew angles should be kept to 25° or less where feasible, in order to minimize expansion joint damage from plowing operations. If clearance allows, consideration should be given to increasing the span length and making a 90° crossing. See 641, Expansion Joints, for further discussion.
10. The State of New Hampshire in RSA 234:2 defines a bridge as a structure having a clear span of 10 ft (3.048 m) or more measured along the centerline of the roadway at the elevation of the bridge seats.
11. The National Bridge Inspection Standards (used by FHWA) defines a bridge as a structure having a clear span of more than 20 ft (6.1 m) measured along the center of the roadway. This definition includes multiple openings, where the clear distance between openings is less than half of the smaller contiguous opening.
12. Design exceptions are required to waive established criteria contained in AASHTO, A Policy on Geometric Design of Highways and Streets (aka "Green Book"). A design exception requires the Assistant Commissioner's approval and FHWA approval on Federal overview projects. Examples of details requiring design exceptions are design speed, lane and shoulder widths, bridge widths, horizontal and vertical alignments, grades, stopping sight distances, cross slopes, superelevations, and horizontal and vertical clearances. See Plate 430.2 for a sample application for a design exception.
13. Verify that the superelevation transition or runoff in the middle of a span does not create negative camber in a beam. Begin or end transitions off the structure or, if this is not possible, begin or end the transition at the centerline of bearings of an abutment or pier.
14. For Recreational Bridge vertical and horizontal clearances, see Section 691.

## 440 Preliminary Hydraulic Study

### 440.1 Hydraulic Calculations

A preliminary hydraulic study should be completed for water crossings. The level of detail for this preliminary study should be in direct proportion to the level of hydraulic design considerations that the site involves. At a minimum, design floods along with flood elevations should be calculated to determine if hydraulic considerations will be a major factor in the allowable depth of superstructure. Calculations should be well documented to assist in the preparation of the hydraulic summary.

The following hydraulic design methods are used within Bridge Design:

From Runoff Estimates For Small Rural Watersheds and Development of a Sound Design Method, Federal Highway Administration, Report No. FHWA-RD-77-159, 1977:

- 5 Parameter Method
- 7 Parameter Method

From Manual on Drainage Design for Highways, New Hampshire Department of Transportation

- Rational Method
- Potter's Method
- New England Hill and Lowland (NEHL) Method
- Adirondack, White Mountain, Maine Woods (AWM) Method

From Technical Release 20, Soil Conservation Service: TR-20 Method

The selection of the waterway opening should be based upon design flood requirements plus 1 ft (0.3 m) of freeboard unless otherwise approved, with consideration given to the amount of allowable upstream ponding, the passage of ice and debris, the possible existence of adjacent relief, and the possible scour of the bridge foundation. Where floods exceeding the design flood have occurred, or where superfloods would cause extensive damage to adjoining property or possible loss of a vital structure, a larger waterway may be warranted.

Flood relief may be considered by lowering approach embankments to provide overflow sections that would pass unusual floods over the highway as a means of preventing loss of structure.

## 440.2 Waterway Opening

The term "design flood" refers to the peak discharge and stage or wave crest elevation of the flood associated with the recurrence interval selected for the design of a highway encroachment on a flood plain. By definition, the highway will not be subjected to inundation from the stage of the design flood.

Selection of the appropriate design flood for bridges is based on several factors, including class of highway, traffic volume, length of detour, and general importance of the bridge. The following guidelines should be used in the selection of the design flood for new highway bridges:

1. The 100-year frequency flood, Q100, should be the design flood for all Interstate, Turnpike, and Primary System highway bridges and all other major bridges. The Q100 flood is equivalent to the FHWA basic flood and is also referred to by FHWA as the intermediate regional flood.
2. The 50-year frequency flood, Q50, should be the design flood for all Secondary and Off-System highway bridges with the exception that any major bridges on these highway systems should be designed for a 100-year frequency flood.
3. A 25-year frequency flood, Q25, may be allowed by the Administrator on Off-System bridges under special conditions determined by the Chief of Design. Notes will be added to the Plans explaining the consequences of the design, and the Town should be notified of this design through the Bureau of Municipal Highways.
4. The 10-year frequency flood, Q10, should generally be used as the design flood for temporary bridges. Unusual circumstances may justify modification to this Q10 requirement on a case by case basis at the discretion of the Chief of Design.
5. The 2.33 year frequency flood, Q2.33, is defined as the mean annual flood by the FHWA and may be calculated using the FHWA publication Runoff Estimates For Small Rural Watersheds and Development of a Sound Design Method (FHWA-RD-77-159). Mean annual flood is synonymous with Ordinary High Water (OHW). Although not a design flood, OHW is required on Wetlands Board applications. The Bureau of Environment determines the limits of OHW. These limits should be reviewed for consistency with hydraulic information and contours.

It is at this point that backwater calculations, if required, must be performed. Computer programs such as FHWA's WSPRO or USACOE's HEC-2 or HEC-RAS are available for this purpose.

## 440.3 Non-Bridge Structures

Waterway openings for non-bridge structures (clear span less than 10 ft (3.048 m)) should be designed to pass a 50-year frequency flood in accordance with NHDOT Manual on Drainage Design for Highways except where evidence of abnormal ice floes or passage of debris may require special consideration.

## 450 Conceptual Bridge Type Selection

### 450.1 Bridge Type Selection

1. See Plate 450.1a for a list of commonly used bridge types with typical span ranges.
2. Single span or multispan steel or concrete beam bridges are common alternatives for the majority of structures. The choice should be made on the basis of judgment, economy, appearance and serviceability.
3. Redundant type (multiple load path) systems are preferred. Non-redundant (single load path; also called "Fracture Critical") systems should be avoided.
4. In the span range up to 100 ft (30 m), steel girders should be considered when necessary to accommodate bridge mounted utilities or when vertical clearances are a controlling factor.
5. Steel structural plate pipes and pipe arches should not be used for crossing hydraulic channels due to their tendency to corrode at the waterline. For pedestrian and recreational trails, steel structural plate may be used. Aluminum structural plate may be an appropriate alternative for either hydraulic or dry crossings. See Section 670 for structural plate pipe-arch size.
6. Concrete rigid frames should be considered in locations where the structure can be placed under fill, for spans normally up to 65 ft (20 m). Leg heights should be 20% to 50% of the span length. This type of structure is also an excellent choice where aesthetics are a consideration.
7. Voided slabs can be considered for spans less than 46 ft (14 m) where vertical clearance is a significant design issue or where rapid or stage construction is required.
8. Precast concrete box beams can be considered for spans between 46 ft (14 m) and 80 ft (24 m) where expedient construction is required. Beams should be butted unless it is necessary to accommodate bridge mounted utilities.
9. Integral abutments should be considered for steel and concrete girder bridges. The maximum length for integral abutment steel bridges is 200 ft (60 m). The maximum length for integral abutment concrete bridges is 325 ft (100 m). Longer spans may be considered with approval of the Administrator.
10. Longitudinally post-tensioned precast concrete girders, precast segmental concrete girders, and steel box girders (proper inspection is difficult) should not be allowed without approval of the Administrator.

### 450.2 Sag Cambers

Because of the objectionable appearance of a sag camber in a beam, sag or negative cambers should be avoided. The following are a few guidelines on possible means of avoiding negative camber on a beam:

1. Avoid sag vertical curves on bridges.
2. Avoid placing a sag camber in a straight beam in order to accommodate the variation in the theoretical bottom of slab elevations. The variation should be taken up in the haunch with a maximum unreinforced haunch height of 6 in (150 mm).

### 450.3 Substructure

Frost cover for footings (ground elevation to bottom of footing) should be 5 ft (1.5 m). Frost cover for footings founded on piles should be 4 ft (1.2 m). Frost cover is not required for footings on competent bedrock. Depth of footings for river crossings will be dictated by scour potential.

The footing type, allowable soil pressure or pile load should be determined by the Design Chief based upon data from the Geotechnical Report.

### 460 Conceptual Site Plan

The project engineer should prepare a site plan to scale showing the wing layout, curb and sidewalk limits, and all stone layout including all existing and proposed contours to show how the proposed bridge and required grading will blend with the existing ground.

### 470 T S & L Review

The type, size, and location of the bridge shall be presented to the Administrator for comment and approval.

### 475 Estimate

An estimated cost of the proposed bridge should be determined using the slope-intercept method and the guidelines listed below. See also Plates [475.1a](#), 475.1b and 475.1c for details.

1. Measure width from face to face of rail and add/include any pedestrian requirements for sidewalks (normally 5 ft (1.5 m) wide).
2. Bridge Cost/SM (SF): Also see Plate 475.1a.

For stage construction add 20-25% to bridge cost.

Adjustments in square meter (sf) cost should also be made for other complications such as skew, difficulty in access, rapid construction, etc. (See list of Additive Costs for Estimating on Plate 475.1a.)

The above listed costs roughly include items in a normal bridge contract. The estimator should be aware and make note in the field of unusual costs that might occur during construction and add these costs to the bridge cost (such as unusual channel work requirements, a historical bridge, a historical site, or archeological considerations).

- |                    |                     |                    |
|--------------------|---------------------|--------------------|
| 3. Roadway Cost/LF | For 24 ft wide road | ≈ \$125 - \$150/lf |
| Roadway Cost/LM    | For 7.2 m wide road | ≈ \$410 - \$490/lm |

For narrower or wider roadway widths, adjust proportionally.

## **480 Public Meetings**

The public shall be kept informed as to the Department's objectives and intentions through the public involvement process (see 1180). The following meetings are typical for a project:

### **480.1 Public Officials**

This is an informal meeting where the Department seeks input concerning details of the project from the Public Officials.

### **480.2 Public Information**

This is an informal meeting where the Department seeks input concerning details of the project from the community.

### **480.3 Public Hearing**

Typically required if easements or land acquisitions are needed in order for the project to be built. If the Bureau of Right-of-Way can negotiate small easements or land acquisitions with the landowners and there is no public sentiment against the project, the hearing may not be required at the discretion of the Bureau of Right-of-Way.

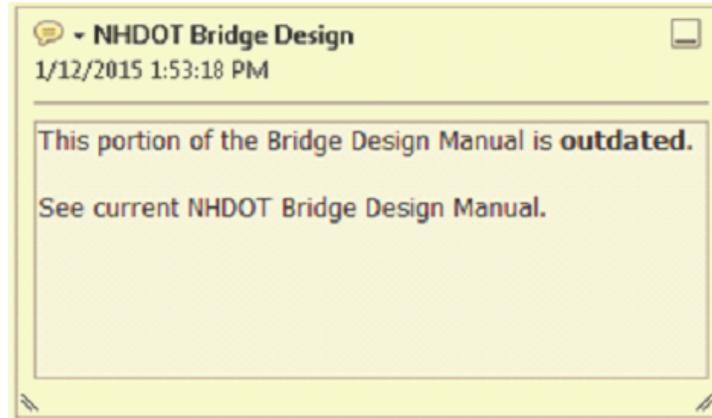
The Public Hearing is a formal meeting where the Department presents the project to a presiding panel (commission, special committee, or town officials). This panel also takes testimony from the public and then makes a decision concerning the necessity of the project.

### **480.4 Display Equipment & Materials**

On very large projects it may be necessary to prepare extensive displays for a public hearing in which case the display boards and easels should be requested from the Highway Garage woodworking section. A request should be submitted a minimum of 10 days prior to the hearing.

# SECTION 500 - PRELIMINARY DESIGN

<b>501</b>	<b>General</b>
<b>510</b>	<b><u>Boring Request</u></b>
510.1	Boring Layout
510.2	Transmittal of Boring Layout
510.3	Check of Boring Log
<b>520</b>	<b><u>Final Hydraulic Study &amp; Scour Analysis</u></b>
520.1	Hydraulic Summary
520.2	Channel Protection
520.3	Obstructions to Stream Flow
<b>530</b>	<b><u>Preliminary Design Requirements</u></b>
530.1	Superstructure
530.2	Substructure
<b>540</b>	<b><u>Preliminary Plans</u></b>
540.1	Format
540.2	Review of Preliminary Plans
540.3	Distribution
<b>550</b>	<b><u>Permits</u></b>
550.1	Wetland Permits
<b>560</b>	<b><u>Roadway Design Guidelines</u></b>
<b>570</b>	<b><u>Right-of-Way Plans</u></b>
570.1	Format
570.2	Review of Right-of-Way Plans



## 501 General

Preliminary Design is that part of the design/plan development process between the Hearing (design approval) and approval for final design from the Administrator. Design approval is constituted by Commissioner's approval and project approval from the presiding panel subsequent to the Public Hearing. In the case of Federal aid projects, approval from FHWA is also required.

## 510 Boring Request

### 510.1 Boring Layout Submittal

After selection of a preliminary structure type and span length, a test-boring layout, usually drawn to a scale of 1:250 (1 in = 20 ft), should be prepared. The Boring Layout submittal (see Plate 510.1) to the Bureau of Materials and Research should include the following information. Information not available at the time of the original boring request may be submitted to the Soils Engineer as it is developed.

1. Town, Bridge No., Project Location information and North Arrow.
2. Proposed boring location plan with requested boring locations [identified by number (numbers should increase from left to right as the observer faces the structure up-station, except for abutment A) and with an outline of the proposed structure]. A minimum of 3 borings per abutment are recommended with one boring on centerline bearing and one at the end of each wing. Maximum recommended spacing of abutment borings is 50 ft (15 m). 2 borings per pier are recommended. Borings should be located by coordinates and by station and offset.
3. Proposed boring location plan for any detour structure which is required.
4. Site plan with ground surface elevation contours at the proposed structure location.
5. Profile at the proposed structure location.
6. Available plans or other related information for existing structures at the proposed site, i.e. existing foundation types, bearing elevations, previous test boring information.
7. Hydraulic data for water crossings, i.e. design flood elevations, design velocity.
8. Information for any related roadway work, which would be designed by the Bureau of Bridge Design. This would include cross sections with plotted template, roadway profile and proposed roadway typical section with proposed structural section.
9. Reproducible Site plan which does not have the originally requested test boring locations plotted. This will be used for the final subsurface exploration plan.
10. Identification of whatever survey lines are currently laid out at the site (i.e. roadway, traverse line, grid base line, etc.).
11. Benchmark description and elevation.

## 510.2 Transmittal of Boring Layout

The Project Engineer should transmit the following copies of the Boring Layout:

To Bureau of Materials and Research	2*
To Chief of Survey, Bureau of Highway Design	1

\*The addresses of the utility owners within the project area shall be requested from the Design Services Section of the Bureau of Highway Design and transmitted with the Boring Request to the Bureau of Materials and Research.

## 510.3 Check of Boring Log

A check should be made, upon receipt of the boring logs, between the ground elevations shown on the borings and the survey plan. The ground elevations should be verified from a copy of the survey notes available in the Design Services Section of the Bureau of Highway Design.

# 520 Final Hydraulic Study & Scour Analysis

## 520.1 Hydraulic Summary

A hydraulic summary shall be completed for water crossings. This report shall show the design flow (Q), flood elevations, waterway opening, span length, channel protection, angle of crossing, and all other data required to complete the Preliminary Plan.

## 520.2 Channel Protection

A scour analysis utilizing FHWA's Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour At Bridges" and HEC-20, "Stream Stability at Highway Structures" shall be performed for each new bridge with a pier located in the waterway. The 500-year frequency flood (Q500) shall be used to evaluate structural stability;  $F.S \geq 1.0$ .

Protection against damage from scour should be provided in the design of bridge piers and abutments.

Embankment slopes adjacent to structures subject to erosion should be adequately protected according to the following guidelines:

Item 587.1, Keyed Stone Fill shall be used whenever practicable. A 2 ft (0.6 m) thickness of keyed stone fill should be considered for velocities of up to 8 ft/sec (2.4 m/sec) and a 3 ft (1 m) thickness should be considered for velocities up to 14 ft/sec (4.2 m/sec). These velocity thresholds should be adjusted downward as flow volumes increase. In selecting keyed stone fill, consideration must be given to the requirement that the material must be compacted and keyed. If this requirement cannot be met, stone fill should be considered.

Item 585, Stone Fill, Class A (3 ft-0 in) or Class B (2 ft-0 in), shall be used in locations where Keyed Stone Fill is not a reasonable option. Stone size should attempt to blend in with the surrounding area. Stone Fill, Class C is often used in the floor of box culverts.

### **520.3 Obstructions to Stream Flow**

When applicable, include a note stating, "Trees, brush, and other obstructions to stream flow in streambed and banks, upstream and downstream of the bridge should be selectively cleared as shown and as directed by the Contract Administrator." The wetlands permit should include this area as an impact area.

## **530 Preliminary Design Requirements**

### **530.1 Superstructure**

It is not necessary at the Preliminary Design stage to complete all calculations necessary to produce contract plans, but main design concerns need to be given careful thought in order to assure that the design concept can progress to a completed project. For example, if clearance above the design flood is a concern, stringer height calculations should be performed and deflections should be checked.

### **530.2 Substructure**

Foundation concerns should be resolved at this time. A decision should be made between options such as the use of spread footings or piles, and integral or stub abutments. Soil limitations, site characteristics, and engineering judgment will all be necessary in determining the substructure configuration. As with the preliminary superstructure design, it is not necessary to calculate all reinforcing sizes, but the substructure concept should be investigated sufficiently to assure that the concept is viable.

## **540 Preliminary Plans**

### **540.1 Format**

The Preliminary Plans shall be prepared to the following format:

1. Front Sheet: Project Titles, Location Maps, Signature Boxes, Traffic Data.
2. Sheet One: Bridge General Plan & Elevation, General Notes, Hydraulic Data, and Superstructure Cross Section.
3. Sheet Two: Profiles of upper and lower roadways (or channel), typical Approach Cross Sections for the upper and lower roadways (or channel), typical cross sections for lower roadway (or channel) and as modified for the bridge proximity. Show utilities data as applicable. Show the Site Plan with contours, etc., developed Abutment and Pier Elevations with pertinent soil information and other details as required.
4. Sheet Three: Boring Logs, Boring Layout Plan and Boring Notes.
5. Roadway Sheets (as required).
6. Roadway
  - a) detail plan
  - b) profile
  - c) cross-sections

See Plates 540.1a through 540.1c for typical notes on Preliminary Plans.

## **540.2 Review of Preliminary Plans**

The Preliminary Plans are subject to the review and approval of the Administrator, Bureau of Bridge Design.

## **540.3 Distribution**

The Preliminary Plans shall be prepared for distribution to Highway Design, Bridge Maintenance, Materials and Research, the FHWA, a neighboring state, Construction, Traffic, District Engineer, Design Services Engineer, etc., as appropriate. The Design Services Engineer receives two copies of the Preliminary Plans for each utility on the project and one copy for their use. Contact Design Services for the required number of plans.

# **550 Permits**

## **550.1 Wetlands Permits**

The NH Wetlands Permit Process Manual is the source of information for the NHDOT.

The necessary documents shall be prepared, as required, to obtain the permits listed below:

Delineation and Classification of Wetlands is the responsibility of the Bureau of Environment. Construction should review the proposed areas of impact prior to submission to the Wetlands Bureau.

### **1. NH Wetlands Permit**

A NH Wetlands permit is required when wetlands or surface waters are impacted by a proposed project.

- a) Prior to submitting a Wetlands Permit Application, the project impacts must be reviewed at one of the monthly natural resource agency meetings hosted by the Bureau of Environment.
- b) Allow at least 6 months from the time the application is submitted to the advertising date. (For large projects allow more than 6 months.)

### **2. US Army Corps of Engineers Permit**

A US Army Corps of Engineers (ACOE) permit is required for projects which impact wetlands or surface waters [river or stream banks below ordinary high water (See 440.2.5 for definition of ordinary high water.)].

There are two US ACOE permit types

- a) State Program General Permit (SPGP)
  - i) Minimum Impact - The SPGP is automatically obtained with issuance of the NH Wetlands Permit.

- ii) Minor Impact - A 30 day wait is required after issuance of the NH Wetlands Permit. After 30 days the project may proceed. (Written approval may or may not be received).
- iii) Major Impact - The project must receive written approval of the SPGP before construction can begin.

Specific criteria for the 3 impact levels of the SPGP (e.g. length of affected shoreline and impacted area) are listed in the NH Wetlands Permit Process Manual.

- b) An Individual ACOE Permit is required when:
  - i) More than 3 acres of wetland or surface water are impacted.
  - ii) Construction work is below ordinary high water on a wild and scenic river or wild and scenic study river.
  - iii) The Army Corps of Engineers feels an individual permit is necessary. Coordinate with the Bureau of Environment when preparing plans for an individual permit.

### 3. Water Quality Certification

- a) For SPGP, a water quality permit is automatically obtained.
- b) For individual ACOE permits a water quality certification must be requested. (Coordinate with the Bureau of Environment to obtain a water quality certificate. Allow approximately 2 weeks to obtain.)

### 4. Coastal Zone Management

A Coastal Zone Consistency finding is necessary for federally funded projects which require an individual ACOE permit or a Coast Guard Permit. Coastal Zone Consistency should be coordinated through the Bureau of Environment. The following is required:

- a) Environmental Study
- b) NH Wetlands Permit
- c) Water quality Certificate
- d) Army Corps of Engineers Permit Application

The process may require 6 months to complete.

### 5. US Coast Guard Permit

Required for impacts to all navigable waters. A USCG permit shall not be required if the FHWA determines that the proposed construction, reconstruction, rehabilitation, or replacement of the federally aided or assisted bridge is over waters:

- a) Which are not to be used or are not susceptible to use in their natural condition or by reasonable improvement as a means to transport interstate or foreign commerce; and
- b) Which are (a) not tidal, or (b) if tidal, used only by recreational boating, fishing, and other small vessels less than 21 feet in length.

The Bureau of Environment maintains a list of rivers that are generally considered to require a USCG permit. If a dam exists between the subject bridge and the ocean then FHWA typically does not consider the river to be navigable.

Required information includes:

- a) Wetlands Bureau Permit
- b) Coastal Zone Consistency findings

Coordination with the Coast Guard through the Bureau of Environment should be initiated as soon as potential impacts are known. The process may require one to two years to complete.

## **560 Roadway Design Guidelines**

The Bureau of Bridge Design is often required to design approaches for bridge projects as well as nearby intersections. The designer is referred to the NHDOT Highway Design Manual and AASHTO's A Policy on Geometric Design of Highways and Streets and AASHTO's Roadside Design Guide.

## **570 Right-of-Way Plans**

### **570.1 Format**

The Right-of-Way Plans shall be prepared for all projects where easements and/or land acquisitions are required. Refer to Appendix 10.3 of the Highway Design Manual for ROW plans check list. The Bureau of ROW will use these plans during negotiations with landowners.

### **570.2 Review of Right-of-Way Plans**

The Right-of-Way Plans are subject to the review and amendment by the Bureau of Right-of-Way. When all additions and alterations to the Right-of-Way Plans are completed, commonly near the completion of Final Contract Plans, a final set of Right-of-Way Plans should be provided on mylar to the Bureau of Right-of-Way for submission to the Registry of Deeds.

## SECTION 600 - FINAL DESIGN

601	<b>General</b>
602	<b><u>Design Loads and Methods</u></b>
603	<b><u>Seismic Design</u></b>
610	<b><u>Footings</u></b>
611	<b><u>Piles</u></b>
612	<b><u>Cofferdams</u></b>
613	<b><u>Abutments</u></b>
614	<b><u>Retaining Walls/Wingwalls</u></b>
615	<b><u>Piers</u></b>
620	<b><u>Prestressed Concrete Structures</u></b>
630	<b><u>Structural Steel</u></b>
631	<b><u>Bridge Inspection Access</u></b>
640	<b><u>Bridge Shoes</u></b>
641	<b><u>Expansion Joints</u></b>
642	<b><u>Bridge Rail and Bridge Approach Rail</u></b>
650	<b><u>Concrete Decks</u></b>
651	<b><u>Approach Slabs</u></b>
652	<b><u>Concrete Slabs</u></b>
653	<b><u>Timber Structures</u></b>
654	<b><u>Waterproofing Membrane</u></b>
660	<b><u>Rigid Frames and Box Culverts</u></b>
670	<b><u>Structural Plate Structures</u></b>
680	<b><u>Bridge Rehabilitation or Widening</u></b>
690	<b><u>Temporary Bridges</u></b>
691	<b><u>Recreational Bridges</u></b>
692	<b><u>Railroad Bridges</u></b>

## **601 General**

### **601.1 General**

[Blank]

## 602 Design Loads and Methods

- 602.1 General
- 602.2 Seismic Loads
- 602.3 Method

### 602.1 General

All bridges should be designed for HS 25-44 (MS-22.5) and 125% of Military Loading.

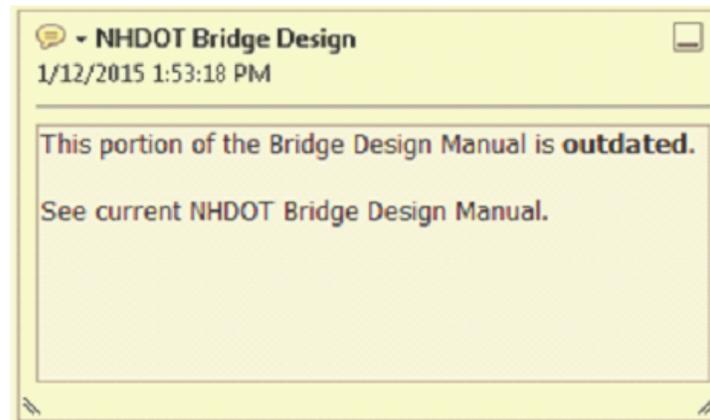
The 125% Military Loading consists of two 15 kip (66,750 N) wheels spaced 6 ft (1800 mm) apart on an axle, and the 30 kip (133,500 N) axles spaced 4 ft (1200 mm) apart.

### 602.2 Seismic Loads

All bridges should be designed for seismic loads in accordance with AASHTO and 603.

### 602.3 Method

All superstructure and substructure components should be designed in accordance with the Strength Design Method (LFD) of the AASHTO Specifications.



## 603 Seismic Design

### 603.1 General

603.1.1 Determination of Seismic Activity

603.1.2 Design of New Bridges

### 603.2 Rehabilitation of Existing Bridges

603.2.1 General

603.2.2 For SPC A Bridges

603.2.3 For SPC B Bridges

### 603.3 Design of New Bridges for Seismic Resistance

603.3.1 Design for SPC A

603.3.2 Design for SPC B - Single Span

603.3.3 Design for SPC B - Two or More Spans

### 603.4 AASHTO Division I-A - Seismic Design- (Section 6)

603.4.1 Group Load = 1.0 (D + B + SF + E + EQM)

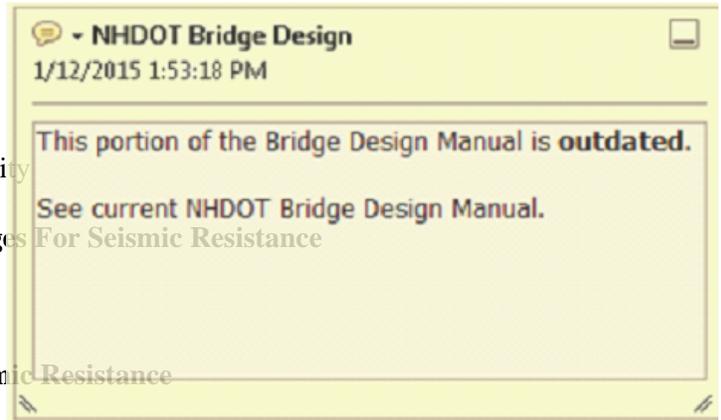
603.4.2 SPC B Substructure Design Guidelines

### 603.5 Seismic Isolation Bearings

603.5.1 Application

603.5.2 Design

### 603.6 Seismic Resistance Guidelines



### 603.1 General

AASHTO - Division I-A Seismic Design specifications should be applied to the design of conventional steel or reinforced concrete main members, using the Single-Mode Spectral Analysis Method. A more rigorous analysis using the Multi-Mode Spectral Analysis Method should be required for seismic design of non-conventional type bridges such as suspension bridges, arches, trusses and moveable bridges, etc.

#### 603.1.1 Determination of Seismic Activity

The level of expected seismic activity is indicated by the Rock Acceleration Coefficient. A contour map of Rock Acceleration Coefficients for NH is attached (see Plate 603.1a).

Based on this map, NH can be divided into regions of Seismic Performance Categories (SPC) A and B.

1. All bridges in SPC A should use a Rock Acceleration Coefficient equal to 0.09.
2. All bridges in SPC B should use a Rock Acceleration Coefficient equal to 0.17.

Major bridges (Interstate, Turnpikes, major river crossings, and others as determined by the Design Chief) should be designed to SPC B requirements using a rock acceleration coefficient as determined by the Design Chief.

### 603.1.2 Design of New Bridges

In designing new bridges for seismic loads, the designer should use AASHTO Division I-A - Seismic Design and should incorporate the seismic design guidelines presented herein.

To maximize seismic resistance in new bridges, special detailing standards have been developed for four basic components of the bridge system (superstructure, bearings, joints and substructure). These standards are described in 603.6 with the intent of improving resistance to seismic loading and reducing the potential of catastrophic bridge failure during a seismic event. Many of these standards are in our current practice. For example, for multiple span bridges our current practice of using continuous spans rather than a series of simple spans is consistent with the standards. However, the new standards recommend that skew angle of the supports be minimized to preclude rotation of the superstructure under seismic loading.

## **603.2 Rehabilitation of Existing Bridges for Seismic Resistance**

### 603.2.1 General

All bridges being rehabilitated should be evaluated for seismic performance as described in Section 6 (Retrofitting) of the FHWA "Seismic Design of Highway Bridges" Manual. Except as noted below, rehabilitated bridges should not be required to have the same expected seismic performance as a new bridge and, therefore, would not require the same seismic resistance as a new bridge. If a rehabilitated bridge needs to meet the design requirements for a new bridge as determined by the Design Chief, a higher level of seismic resistance design may be appropriate.

### 603.2.2 For SPC A Bridges

1. Bridge seat bearing lengths should be checked.
2. Bearings should be checked for seismic stability and horizontal load capacity.
3. Multiple simple span bridges should be evaluated for use of longitudinal joint restrainers or other means of attaching girders to one another.

### 603.2.3 For SPC B Bridges

For SPC B Bridges and all bridges on Interstates and Turnpikes, check all requirements of SPC A plus the following:

#### 603.2.3.1 Basic Rehabilitation

Basic rehabilitation is defined as minimal work, (i.e. strip pavement and patch deck.)

1. No additional requirements.

#### 603.2.3.2 Intermediate Level Rehabilitation

Intermediate level rehabilitation is defined as deck replacement work.

1. Remove and replace rocker bearings.
2. Tie together or restrain girders in multiple simple span structures.
3. Consider making the concrete deck continuous over piers.
4. Evaluate adequacy of confinement reinforcing in columns/piers.

#### 603.2.3.3 Extensive Rehabilitation

Extensive rehabilitation is defined as superstructure replacement or widening by 25% or more.

1. All of intermediate level requirements.
2. Design structure to resist all the forces as required for a new structure (complete review of superstructure and substructure).

### **603.3 Design of New Bridges for Seismic Resistance**

#### 603.3.1 Design for SPC A

A detailed seismic analysis (using one of the methods listed in Section 4.1 of AASHTO Division 1-A) is not required for bridges in the SPC A region. Single and multiple span bridges in this category should meet the following requirements:

1. Provide minimum support length  $N$  of 12 in (300 mm); see Plate 603.3a (603.3b). A bearing seat width of 24 in (600 mm) should meet this criteria and should otherwise meet the requirements of Section 3.10 and 5.3 of AASHTO Division I-A.
2. The resistance of the connections between the superstructure and substructure to seismic loading should be as specified in Section 3.11 and 5.2 of AASHTO Division I-A.
3. Foundations - No special design requirements are specified for this category (Section 5.4 of the AASHTO Division I-A). (Therefore, the load from Section 3.11 and 5.2 of AASHTO Division I-A should not be included in the foundation design.)

### 603.3.2 Design for SPC B - Single Span

A detailed seismic analysis is **Not** required.

1. Provide minimum support length N of 12 in (300 mm); see Plate 603.3a (603.3b). A bearing seat width of 24 in (600 mm) should meet this criteria and should otherwise meet the requirements of Section 3.10 and 6.3 of AASHTO Division I-A.
2. The resistance of the connections between the superstructure and substructure to seismic loading should be as specified in Section 3.11 of AASHTO Division I-A.
3. Foundations - No special design requirements are specified for this category (Section 3.11 of AASHTO Division I-A). (Therefore, the load from Section 3.11 of AASHTO Division I-A should not be included in the foundation design.)
4. For long simple spans, 150 ft (45 m) or greater, the foundation should be designed in accordance with Section 6.4.3 of AASHTO Division I-A.

### 603.3.3 Design for SPC B - Two or More Spans

Perform a detailed seismic analysis, using one of the procedures from Section 4.1 of AASHTO to determine the orthogonal forces to be modified and combined with the other loads shown in 603.4.1 for the design of:

1. The superstructure, its expansion joints, and the connection between the superstructure and the supporting substructure.
2. Supporting substructure down to the base of the columns and piers, and the footing, pile cap or piles and abutments.
3. Bearing connections and shear keys between a superstructure and an abutment.

Modify the seismic design forces for foundations and combine with the loads shown in 603.4.1 for designing footings, pile caps, piles, etc. (e.g. calculate EQF as noted).

## 603.4 AASHTO Division I-A - Seismic Design - (Section 6)

603.4.1 Group Load =  $1.0 (D + B + SF + E + EQM)$

Where

D = Dead Load

B = Buoyancy

SF = Stream-Flow Pressure

E = Earth Pressure

\* EQM = Elastic Seismic Force for either LOAD CASE 1 or LOAD CASE 2 modified by dividing by the appropriate R-Factor.

\* For foundations this term becomes EQF which is the same elastic seismic force except it is modified by 1/2 of the R-factor of the substructure to which it is attached.

Where

LOAD CASE 1 = 100% Long. + 30% Trans. (Due to Seismic Loading)

LOAD CASE 2 = 30% Long. + 100% Trans. (Due to Seismic Loading)

603.4.2 SPC B Substructure Design Guidelines

Abutment Design:

1. Seismic design is not required for single spans except as outlined in 603.3.2.
2. Reference the following sections of AASHTO Division I-A:

Sect. 6.4 - Foundations.

Sect. 6.4.3 outlines the analysis procedures. Abutments on battered piles or bedrock should be considered to be restrained and the maximum lateral earth pressure computed using a seismic coefficient of  $k_h = 1.5A$ , in conjunction with the Mononobe-Okabe analysis method.

3. Do not apply "static earth pressure" in combination with "seismic earth pressure".

Pier Design:

1. Reference the following sections of AASHTO Division I-A:

Section 6.2 - Design Forces

Section 6.4 - Foundations

Section 6.6 specifies requirements for transverse reinforcement in order to provide ductility. Maximum spacing of transverse reinforcement is equal to 6 in (125 mm).

Wingwall Design: See AASHTO 5.5.4, 5.5.5 and 5.8.9 (MSE Walls)

## **603.5 Seismic Isolation Bearings**

### 603.5.1 Application

Isolation bearings should be used only when approved by the Design Chief. Isolation bearings would normally be used only for rehabilitations and widening projects. This decision should be made at the Preliminary Plans (30%) stage, which will allow for the proper design of the seismic isolation bearings and the substructure.

When assessing the effects of using seismic isolation bearings, a quick and conservative method of estimating the seismic loading is to assume that the lateral seismic load in any direction is 0.12 x dead load reaction. If the force can be redistributed to other substructure elements, then the seismic load on a pier and/or abutment can be reduced as low as 0.07 x dead load reaction.

### 603.5.2 Design

Once approval has been given for the use of seismic isolation bearings see Appendix B for the steps that should be taken to ensure that approved Suppliers can provide the best product.

## **603.6 Seismic Resistance Guidelines**

1. Continuous load carrying main members on a multiple span bridge should be used rather than a series of simply supported spans.
2. Skew angle for bridges should be minimized as much as possible. Skewed supports encourage rotation of the superstructure about a vertical axis under seismic loads.
3. The type of bridge bearings provided should be either elastomeric pads, sliding plate, or multi-rotational (pot type or disc type). Steel rocker bearings should not be used.
4. All expansion bearings with sliding surfaces should be provided with guide bars to allow limited lateral movement.
5. All multiple span bridge bearings should be properly anchored to the substructure and superstructure to resist a horizontal force, in both the transverse and longitudinal directions, equal to 0.20 x superstructure Dead Load. However, for multiple span bridges in Category B this horizontal force should not be less than the AASHTO Seismic Design Section 6 group loading.
6. Bridge bearing seats supporting ends of girders should be designed to provide a minimum support length "N" measured normal to the face of the abutment or pier (Plate 603.3a and 603.3b) except if the existing practice requires a wider seat, then the existing practice will control. The minimum support length should also be provided perpendicular to the fascia girders.
7. Abutments and piers should be provided with a continuous footing.
8. Mechanically Stabilized Earth (MSE) abutments should not be used in Category B. (Per recommendation CALTRANS).
9. Tall concrete columns in a multi-column pier (slenderness ratio > 60 in the direction parallel to the support), should be provided with reinforced concrete strut(s) near the middle half of the column's height.

10. Vertical reinforcement for columns should be extended into the pier cap for full embedment length.
11. The spacing of lateral ties for pier column(s) should not exceed the least dimension of the compression member or 12 in (250 mm) (AASHTO Section 8.18.2.3). Additional ties should be provided to make the spacing at 6 in (125 mm) on-center at the top and bottom of the column over a length equal to the maximum cross-sectional column dimension (thickness of a wall pier) or one-sixth of the clear height of the column but not less than 18 in (450 mm). The ties should be continued for a distance equal to one half the maximum column dimension (thickness, for wall type piers) but not less than 15 in (375 mm) from the face of the column connection into the adjoining cap beam or the footing (Plate 603.6a). Transverse reinforcing for piers should be as required by AASHTO Section 8.17.3. Stagger location of splices when practical. A minimum number of ties should be provided within the plastic moment region even if the design calls for the pier to remain elastic ( $R=1$ ) (The minimum number of ties is considered a 12 in x 36 in (250 mm x 875 mm) grid using #5 (#16) bars.).

When using wall piers use the reduced section to determine tie requirements with a typical sizing of #5 (#16) bars on a 6 in x 12 in (125 mm x 250 mm) grid.

12. When a widened section (plinth) is provided at the base of the column, (normally not a recommended detail) the design vertical reinforcement for the column should be extended to dowel into the footing. Additional reinforcement in the plinth should be provided as required.
13. All stirrups and ties should be provided with a  $135^\circ$  hook for anchorage.
14. The main tensile reinforcing bars for abutments should be provided with hooks in the footing ( $180^\circ$  or  $90^\circ$ ). Cantilever type abutments and walls should be provided with J-bars, #5 bars @ 1 ft-0 in (#16 @ 250 mm) minimum on the compression face to connect the footing with the stemwall.
15. Footing reinforcement for piers:

Minimum top reinforcement for a continuous footing should not be less than #5 at 12 in (#16 bars at 250 mm) centers in the transverse and longitudinal directions.

Minimum top reinforcement for an individual footing should not be less than 50% of the area of the designed bottom reinforcement or #5 bars at 6 in (#16 bars at 125 mm) on centers in the transverse and longitudinal directions.

All pier footing dowels should be provided with hooks in the footing ( $180^\circ$  or  $90^\circ$ ).

16. Spread footings on deep, loose to medium dense deposits should be avoided since liquefaction risks are high.

## **610 Footings**

- 610.1 General**
- 610.2 Typical Sections**
- 610.3 Materials**
  - 610.3.1 Concrete
  - 610.3.2 Reinforcement
- 610.4 Guidelines for Design**
  - 610.4.1 Design Method
  - 610.4.2 Principal Layout Lines
  - 610.4.3 Design Calculations
  - 610.4.4 Friction Factor
  - 610.4.5 Live Load Surcharge
  - 610.4.6 Passive Earth Pressure
  - 610.4.7 Safety Factors
  - 610.4.8 Frost Cover
  - 610.4.9 Footing on Bedrock
  - 610.4.10 Footing Elevations
  - 610.4.11 Pay Limits
  - 610.4.12 Limits of Excavation
  - 610.4.13 Steps
  - 610.4.14 Footing Seal
  - 610.4.15 Layout of Reinforcing
  - 610.4.16 J-Bars
  - 610.4.17 Reinforcing Parameters

### **610.1 General**

Footings should be designed to handle both the final loads and construction loads anticipated during construction of the footing.

### **610.2 Typical Sections**

Footings should be reinforced concrete slabs with a minimum thickness of 2 ft (600 mm).

Footing widths should be multiples of X'-6 in (250 mm) to allow for reinforcement placing at 12 in (250 mm) intervals with 3 in (125 mm) of cover.

### **610.3 Materials**

#### 610.3.1 Concrete

Typical designs should be based on a 28-day concrete compressive strength of 3,000 psi (f'c of 20 MPa). Concrete compressive strengths greater than this may be used with the approval of the Design Chief.

### 610.3.2 Reinforcement

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (M 31M Grade 420).

## 610.4 Guidelines for Design

### 610.4.1 Design Method

Footing sizes and footing reinforcing should be designed by Strength Design Method (LFD).

### 610.4.2 Principal Layout Lines

Principal layout lines should be the CL Bearings, or the Face of Frame Leg, and the Face of Wing lines. The intersections of these lines at each corner of the Abutment or Frame should be designated as "Working Point" (WP). The intersection of the CL Construction and Bearings or Face of Frame Leg should also be designated as "WP".

### 610.4.3 Design Calculations

Design calculations should show toe and heel pressures at all sections for all loadings considered. Sources of unusual or exceptional design conditions should be noted in the calculations.

Consider possible loading conditions, which may occur during construction (e.g. crane loading with no fill in front of toe) and prohibit any conditions, which could dangerously overload any portion of the structure during that phase.

### 610.4.4 Friction Factor

To determine the sliding safety factor, refer to Table 5.5.2B of AASHTO Division I for the friction factor of concrete footing on soil.

The friction factor for concrete on structural fill should be 0.55.

### 610.4.5 Live Load Surcharge

Design for a minimum of 0.6 m (2 ft-0 in) of Live Load surcharge when traffic comes within half the height of the wall. This may be modified when traffic is carried on an approach slab: See 613.4.1 and 651 for more details.

### 610.4.6 Passive Earth Pressure

Do not include passive earth pressure in stability calculations, unless approved otherwise by the Design Chief.

Passive earth pressure for stability may be included for seismic loading cases.

#### 610.4.7 Safety Factors

See AASHTO Section 5.5.5 for WSD and Section 5.14 for LFD.

#### 610.4.8 Frost Cover

When the footing is founded on structural fill or natural soil, the footing should have 5 ft (1.5 m.) of frost cover unless approved otherwise by the Design Chief. When the footing is founded on piles, the footing should have 4 ft (1.2 m) of frost cover unless approved otherwise by the Design Chief.

When the footing is founded on sound bedrock, frost cover is not required. Depth of footings for river crossings will be dictated by scour potential.

#### 610.4.9 Footing on Bedrock

When the footing is founded on bedrock, the plans should show under breakage and over breakage as specified per the soils report. The footing should be designed and rebar set per the minimum thickness. Care must be taken to clearly note the pay limits. The quantities should be determined using the over breakage limits.

#### 610.4.10 Footing Elevations

Give all footing elevations at the bottom of footing, except in bedrock where top of footing and minimum and maximum footing thickness should be indicated.

#### 610.4.11 Pay Limits

Pay limits for all cases should be detailed with typical sections and dimensions. Coordinate roadway and bridge excavation pay limit lines.

#### 610.4.12 Limits of Excavation

All roadway and structure excavation should be classified whenever available soils information permits a reasonable breakdown. All structure excavation should be classified as common bridge excavation or rock bridge excavation.

Limits of excavation for the removal of the existing bridge should be as shown in Plate 610.4a.

#### 610.4.13 Steps

Footing steps are allowed and should follow the following criteria:

1. Maximum step height should be the thickness of the footing unless on sound bedrock.
2. The step should be sloped on a 1V to 2H on structural fill and may be vertical in bedrock.
3. Reinforcing should tie the footing together between steps.
4. A construction joint will be permitted at the step.

#### 610.4.14 Footing Seal

The footing seal and pay limits should be shown 2 ft wider than the footing on all sides.

#### 610.4.15 Layout of Reinforcing

Reinforcing bars should be spaced at 6 in (125 mm) or 12 in (250 mm) intervals.

#### 610.4.16 J-Bars

J-bars should be used front and back in all abutments, wingwalls and piers.

Inside bend radius for vertical cantilever wall footing J-bars should be 8 in (200 mm). Use ACI Standard Practice and AASHTO Table 8.23.2.1 for other bending radii.

The embedment of the J-bar should be 12 in (300 mm) minimum or 12 db. The horizontal straight section of the back J-bar may contribute to the toe steel requirement.

As a rule of thumb, J-bars should not extend more than the bar size in feet #6-6 ft (bar size in decimal meter #19-1.9 m) above the top of the footing.

#### 610.4.17 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

## **611 Piles**

### **611.1 General**

### **611.2 Typical Sections**

### **611.3 Materials**

611.3.1 Steel

611.3.2 Concrete

611.3.3 Timber

### **611.4 Guidelines for Design**

611.4.1 End Bearing

611.4.2 Pile Layout

611.4.3 Pile Embedment into Footing

611.4.4 Pile Lengths

611.4.5 Pile Driving Tips and Splices

611.4.6 Pile Driving Equipment

### **611.1 General**

Piles should be used when soil conditions do not permit the use of a spread footing founded on structural fill, natural soil, or bedrock.

### **611.2 Typical Sections**

For steel H piles the preferred sections are HP 310x79 and HP 250x62 (HP 12x53 and HP 10x42).

For steel pipe piles filled with concrete the preferred sections are 12.75 in diameter x 1/2 in and 10.75 in diameter x 1/2 in (325 mm diameter x 13 mm and 275 mm diameter x 13 mm.).

For concrete piles the preferred sections are square precast-prestressed piles.

### **611.3 Materials**

#### 611.3.1 Steel

Steel H-piles should be based on a yield strength  $F_y$  of 36,000 psi (250 MPa). If a yield strength of 50,000 psi (345 MPa) is needed a special provision is required. Pipe piles should be based on a yield strength  $F_y$  of 35,000 psi (240 MPa).

#### 611.3.2 Concrete

Concrete piles should be based on a 28-day concrete compressive strength  $f'_c$  of 5,000 psi (35 Mpa.)

#### 611.3.3 Timber

Timber piles should conform to ASTM D 25.

## **611.4 Guidelines for Design**

### 611.4.1 End Bearing

All piles should be end bearing, unless approved otherwise by the Design Chief.

### 611.4.2 Pile Layout

For battered piles, detail pile layout at the bottom of footing.

### 611.4.3 Pile Embedment into Footing

Pile embedment in the footings should be as directed. Typically, extend piles 1 ft-6 in (500 mm) into stub abutments; 1 ft (300 mm) into spread footings, and through pier seals 1 ft (300 mm) into the pier footings.

### 611.4.4 Pile Lengths

The estimated static load tip elevation should be calculated by using Wave Equation Analysis as supplied by the Bureau of Materials & Research or other method approved by the Design Chief.

Figure pile length from top of pile to the calculated static load tip elevation. Vertical and battered piles may be calculated separately. Round up the calculated lengths to the next higher 5 ft (2 m) increment. For most situations, add 5 ft (2 m) to this figure, and use this final estimated length on the plans.

### 611.4.5 Pile Driving Tips and Splices

Pile driving tips will be provided unless the Design Chief directs otherwise.

Pile splices should not be allowed if the anticipated pile length is less than 60 ft (18 m). A pile splice per pile should be allowed if the anticipated pile length is greater than 60 ft (18 m). See Plate 611.1a.

### 611.4.6 Pile Driving Equipment

A contract should include only one unit for Pile Driving Equipment. If more than one bridge in the contract requires pile-driving equipment, show a fraction of a unit for each bridge involved.

## **612 Cofferdams**

- 612.1 General**
- 612.2 Typical Sections**
- 612.3 Materials**
- 612.4 Guidelines for Design**
  - 612.4.1 Note
  - 612.4.2 Buoyancy
  - 612.4.3 Safety Factor

### **612.1 General**

[Blank]

### **612.2 Typical Sections**

[Blank]

### **612.3 Materials**

[Blank]

### **612.4 Guidelines for Design**

#### 612.4.1 Note

A note should be included in the contract plans pertaining to cofferdams of significant size (exposed height greater than 6 ft (2 m)) or that are required to protect the general public, which reads:

"The Contractor shall submit the proposed method of cofferdam construction to the Administrator, Bureau of Bridge Design for documentation in accordance with 105.02 prior to the start of construction. The cofferdam shall be designed by a NH Licensed Professional Engineer and the proposed plans shall include the Engineer's stamp, design calculations, and appropriate drawings."

#### 612.4.2 Buoyancy

Cofferdam seals in water should be designed to resist uplift caused by the buoyancy of the displaced water, with a factor of safety, which varies depending on site conditions (in general 10%). For guidance see the Design Chief. The minimum thickness of underwater seals should be determined by multiplying the total water depth to the bottom of seal, by the ratio of unit weight of water to unit weight of concrete ( $62.4/144 = 0.433$ ) plus added thickness to provide for the factor of safety of 1.1. Minimum seal thickness is 4 ft. If necessary, allowable friction on piles of up to 20 psi may be used, depending on soil conditions. Contract plans should specify the cofferdam vent elevation. For more information refer to the AASHTO Guide Design Specification for Bridge Temporary Works.

#### 612.4.3 Safety Factor

The safety factor for cantilevered sheeted cofferdams should be a minimum of 1.3 times the embedment dimension.

## **613 Abutments**

- 613.1 General**
- 613.2 Typical Sections**
  - 613.2.1 Straight Back
  - 613.2.2 MSE Walls with Stub Abutments
- 613.3 Materials**
  - 613.3.1 Concrete
  - 613.3.2 Reinforcing Steel
- 613.4 Guidelines for Design**
  - 613.4.1 Abutment Design Guidelines
  - 613.4.2 Passive Earth Pressure
  - 613.4.3 Live Load Surcharge
  - 613.4.4 Frost Pressure
  - 613.4.5 Loading Conditions
  - 613.4.6 Joints in Abutment
  - 613.4.7 Barrier Membrane
  - 613.4.8 Main Reinforcing Steel
  - 613.4.9 Development and Embedment Lengths
  - 613.4.10 Reinforcing Parameters
- 613.5 Integral and Semi-Integral Abutments**
  - 613.5.1 Introduction
  - 613.5.2 Design Guidelines

### **613.1 General**

[Blank]

### **613.2 Typical Sections**

#### **613.2.1 Straight Back**

The typical abutment wall should be a constant thickness (straight back).

#### **613.2.2 MSE Walls with Stub Abutments**

MSE walls with stub abutments should not be used in SPC B.

## 613.3 Materials

### 613.3.1 Concrete

Typical design should be based on a 28-day concrete compressive strength  $f'_c$  of 3,000 psi (20 MPa), unless approved otherwise by the Design Chief.

Backwall concrete above the bridge seat should have a 28-day concrete compressive strength  $f'_c$  of 4,000 psi (30 MPa) and be QC/QA for durability.

### 613.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (M 31M Grade 420).

Reinforcing should be epoxy coated when exposed to roadway deicing chemicals (e.g. backwalls with expansion joints).

## 613.4 Guidelines for Design

### 613.4.1 Abutment Design Guidelines

For purposes of evaluating an abutment for overturning and sliding, the equivalent fluid pressure diagram should be considered to extend from the bottom of footing to the top of the abutment (i.e. finished grade). In the design of the abutment stem the equivalent fluid pressure diagram for lateral soil load should be similarly applied.

Per AASHTO, abutments with properly designed approach slabs need not consider the additional effects of LL surcharge on the abutment. The approach slab should extend far enough back from the abutment to intersect the failure slope for backfill material (for overturning this would be 60° from the bottom of the back of footing for granular backfill), otherwise the effects of LL surcharge should be added to the soil pressure diagram.

In special circumstances, such as with high abutments, the Design Chief may modify the guidelines above.

See 651 for typical approach slab designs.

### 613.4.2 Passive Earth Pressure

Do not take advantage of any passive earth pressure in stability calculations, unless approved otherwise by the Design Chief.

Passive earth pressure for stability may be included for seismic loading cases.

### 613.4.3 Live Load Surcharge

Design for a minimum of 2 ft (0.6 m) of live load surcharge when traffic comes within half the height of the abutment. This may be modified when traffic is carried on an approach slab, see 613.4.1 and 651 for more details.

#### 613.4.4 Frost Pressure

See 614.4.2

#### 613.4.5 Loading Conditions

Analyze all possible loading conditions, which may occur during construction and prohibit any conditions which could dangerously overload any portion of the structure during that phase.

#### 613.4.6 Joints in Abutment

Abutment wall design should account for concrete shrinkage and constructibility of the wall.

No joints are required if the length is less than 60 ft (20 m). See Plate 614.4a (614.4b) for joint details.

#### 613.4.7 Barrier Membrane

Item 538.2, Barrier Membrane, Vertical Surfaces (F), should be specified for the protection of construction joints, contraction joints, and expansion joints. See Plate 614.4a (614.4b) for details.

#### 613.4.8 Main Reinforcing Steel

When designing the main reinforcing steel for cantilever walls, leave approximately 1 ft (300 mm) space between the top and bottom of adjacent splices. This will allow the splice to be Class B.

#### 613.4.9 Development and Embedment Lengths

See Plates 601.1d (601.1i) for development and embedment lengths.

#### 613.4.10 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

### **613.5 Integral and Semi-Integral Abutments**

Integral and Semi-Integral abutments are the preferred abutment type of the Bureau of Bridge Design. The benefit of less future maintenance is one of the major reasons for this preference.

#### 613.5.1 Introduction

Design of integral abutment bridges has evolved since the 1970's as transportation departments have gained experience.

#### 613.5.2 Design Guidelines

See 450.1 (9)

See AISI publication - Integral Abutments for Steel Bridges.

## **614 Retaining Walls/Wingwalls**

### **614.1 General**

### **614.2 Typical Sections**

#### 614.2.1 Butterfly Wing

### **614.3 Materials**

#### 614.3.1 Concrete

#### 614.3.2 Reinforcing Steel

#### 614.3.3 Steel Sheet Piling

#### 614.3.4 Precast Concrete Sheet Piling

#### 614.3.5 Stone Masonry

#### 614.3.6 MSE Systems

### **614.4 Guidelines for Design**

#### 614.4.1 Retaining Wall Design Guidelines

#### 614.4.2 Frost Pressure

#### 614.4.3 Passive Earth Pressure

#### 614.4.4 Live Load Surcharge

#### 614.4.5 Loading Conditions

#### 614.4.6 Determining Wing Lengths

#### 614.4.7 U-Back Retaining Wall Expansion Joint

#### 614.4.8 Flared Retaining Wall Expansion Joint

#### 614.4.9 Joints in Retaining Wall

#### 614.4.10 Barrier Membrane

#### 614.4.11 Main Reinforcing Steel

#### 614.4.12 Development and Embedment Lengths

#### 614.4.13 Reinforcing Parameters

#### 614.4.14 Curb Seat

### **614.1 General**

Permanent retaining walls and wingwalls should be detailed and designed to achieve an aesthetically pleasing appearance and be essentially maintenance free throughout their design service life. See AASHTO Chapter 5 for different types and limitations of retaining walls.

Flared wingwalls should normally be the first option considered when laying out wingwall geometry. The flared wingwall gives the shortest length, lowest initial cost, and fewer conflicts with guardrail posts, especially when an MSE system is used.

U-back wingwalls would be considered in the following situations:

- To avoid conflicts with cofferdams or substantially ease cofferdam construction.
- To avoid impacts with the existing bridge or approach roadway (maintenance of traffic).
- To avoid/minimize property or environmental impacts.
- To avoid having to construct the wingwall(s) in an additional construction phase.

## 614.2 Typical Sections

### 614.2.1 Butterfly Wing

The maximum butterfly length should be 10 ft (3 m). The typical butterfly length should be 8 ft (2.5 m).

Butterfly wings are typically not to be used on wings shorter in length than 18 ft (5 m). Typically the butterfly height should be less than approximately half the height of the wing. Butterflies should not be longer than one-third the total wing length.

Circumstances which might otherwise lead to the use of butterflies typically involve the advantages obtained by having a shortened footing; e.g. fewer piles (especially if piles are long), avoiding conflicts with adjacent structures (minimizes excavation), avoiding undermining of adjacent footings, minimizing ledge excavation, avoiding conflicts with temporary sheeting and avoiding utilities.

## **614.3 Materials**

### 614.3.1 Concrete

Typical design should be based on a 28-day concrete compressive strength  $f'_c$  of 3,000 psi (20 MPa), unless approved otherwise by the Design Chief.

The curbs/coping on U-Back wingwalls should specify  $f'_c$  of 4,000 psi (30 MPa) and be QC/QA concrete.

### 614.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60(AASHTO M 31M Grade 420).

### 614.3.3 Steel Sheet Piling

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### 614.3.4 Precast Concrete Sheet Piling

[Blank]

### 614.3.5 Stone Masonry

[Blank]

### 614.3.6 MSE Systems

[Blank]

## **614.4 Guidelines for Design**

Retaining Walls should be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads.

### 614.4.1 Retaining Wall Design Guidelines

Retaining Walls should be designed for the loading condition acting at a section located one-third of the panel (between vertical construction joints) length from the high end of the panel.

If the flared wing retaining wall has a butterfly section, the retaining wall may be designed for the maximum height with no butterfly effects or the loading condition located at a section one-third of the panel length plus the butterfly effects.

If the U-back wing retaining wall has a butterfly section, the retaining wall should be designed using the maximum height with butterfly effects distributed over the wall length.

Locate the resultant of the earth pressure at  $0.4H$  from the bottom of the wall per AASHTO 5.14.6.

For an extreme event (seismic) see LRFD 11.6.5.

#### 614.4.2 Frost Pressure

In special circumstances (e.g. high ground water, minimal use of granular backfill, gravel road) use 0.7 kip per linear ft (1000 N per m) frost pressure for U-back retaining walls where ice could be trapped and result in a build-up of pressure. Apply the frost pressure at the top of the wall. This loading condition applies for stem reinforcing steel design only, not for wall stability or footing design. Frost pressure should not normally be used in conjunction with live load surcharge.

#### 614.4.3 Passive Earth Pressure

Do not use any passive earth pressure in stability calculations, unless approved otherwise by the Design Chief.

Passive earth pressure for stability shall be included for sheet pile walls.

Passive earth pressure for stability may be included for seismic loading cases.

#### 614.4.4 Live Load Surcharge

Design for a minimum of 2 ft (0.6 m) of Live Load surcharge when traffic comes within half the height of the retaining wall. This may be modified when traffic is carried on an approach slab; see 651 for more details.

#### 614.4.5 Loading Conditions

Analyze all possible loading conditions which may occur during construction and prohibit any conditions which could dangerously overload any portion of the structure during that phase.

#### 614.4.6 Determining Wing Length

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#### 614.4.7 U-Back Retaining Wall Expansion Joint

At expansion joints in U-back retaining walls (for walls taller than 30 ft (9 m)), the face of retaining wall should be recessed a minimum of 3 in (75 mm) from the face of abutment so movement in the retaining wall will not be noticed.

#### 614.4.8 Flared Retaining Wall Expansion Joint

Flared retaining walls should be recessed 3 in (75 mm), if the total height of the retaining wall (measured from the top of wall to the bottom of footing) is greater than 30 ft (9 m).

#### 614.4.9 Joints in Retaining Wall

Retaining wall design should account for concrete shrinkage and constructibility of the wall (See AASHTO Section 5.5.6.5).

#### 614.4.10 Barrier Membrane

Item 538.2, Barrier Membrane, Vertical Surfaces (F), should be specified for the protection of construction joints, contraction joints, and expansion joints. See Plate 614.4a (614.4b) for details.

#### 614.4.11 Main Reinforcing Steel

When designing the main reinforcing steel for cantilever retaining walls, leave approximately 12 in (300 mm) space between the top and bottom of adjacent splices. This will typically allow the splice to be Class B.

#### 614.4.12 Development and Embedment Lengths

See Plates 601.1d (601.1i) for development and embedment lengths.

#### 614.4.13 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

#### 614.4.14 Curb Seat

A seat for granite approach curb should be provided at the ends of U-back wings.

## 615 Piers

### 615.1 General

### 615.2 Typical Sections

- 615.2.1 Wall Pier
- 615.2.2 Hammerhead Pier
- 615.2.3 Circular Column Pier
- 615.2.4 Rectangular Column Pier

### 615.3 Materials

- 615.3.1 Concrete
- 615.3.2 Reinforcing Steel

### 615.4 Guidelines for Design

- 615.4.1 Ice Pressure
- 615.4.2 Scour Protection of River Piers
- 615.4.3 Passive Earth Pressure
- 615.4.4 Loading Conditions
- 615.4.5 Joints in Pier
- 615.4.6 Pier Nose
- 615.4.7 Development and Embedment Lengths
- 615.4.8 Reinforcing Parameters

### 615.1 General

[Blank]

### 615.2 Typical Sections

#### 615.2.1 Wall Pier

The typical wall pier is 3 ft (1.0 m) thick and up to 20 ft (6 m) high.

The typical wall pier is tapered 1 to 12 $\pm$  on the leading and trailing edges of the wall pier. The taper is 1 to 50 for the sides of the wall pier when the pier is in the waterway or ice path. There should be no side tapers for locations not subject to ice.

#### 615.2.2 Hammerhead Pier

The typical hammerhead pier is 4 ft (1.25 m) thick and over 20 ft (6 m) high.

The typical hammerhead pier is tapered 1 to 12 $\pm$  on the leading and trailing edges of the hammerhead pier. The taper is 1 to 50 for the sides of the hammerhead pier when the pier is in the waterway or ice path. The Q50 flood elevation should be below the lowest elevation of the hammerhead. There should be no tapers for locations not subject to ice.

### 615.2.3 Circular Column Pier

The typical circular column pier is 3 ft (1 m) diameter columns with a 4 ft (1.25 m) thick pier cap. Circular column piers should not be used in the waterway or ice path without approval of the Design Chief.

### 615.2.4 Rectangular Column Pier

The typical rectangular column pier is 3 ft (1 m) wide columns with a 4 ft (1.25 m) thick pier cap. Rectangular column piers should not be used in the waterway or ice path without approval of the Design Chief.

## 615.3 Materials

### 615.3.1 Concrete

Typical design should be based on a 28-day concrete compressive strength  $f'_c$  of 3,000 psi (20 MPa), unless approved otherwise by the Design Chief. In locations of severe exposure, higher strength concrete may be considered as directed by the Design Chief.

### 615.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (M 31M Grade 420).

## 615.4 Guidelines for Design

### 615.4.1 Ice Pressure

#### Static Ice Pressure

The design of river piers should include provisions, if required, to resist the static ice pressure determined by AASHTO LRFD Section 3.9.3. As the commentary of Section 3.9.3 indicates "Little guidance is available for predicting static ice loads on piers. ... and ice usually acts simultaneously on both sides of the pier surrounded by the ice so that the resultant force is considerably less...". If the pier is situated as to produce a load only on one side the static load should be 5000 pounds per foot (See US Army Corps of Engineers Ice Engineering December 31, 1996 EM 1110-2-1612 Section 9-8 for more information.)

#### Dynamic Ice Pressure

The design of river piers should include provisions to resist the dynamic ice pressure determined by AASHTO Section 3.18.2.2. (See US Army Corps of Engineers Ice Engineering December 31, 1996 EM 1110-2-1612 Section 9-3 thru 9-7 for more information.)

#### 615.4.2 Scour Protection of River Piers

See 520.2

The design of river piers should include provisions to adequately protect the pier from undermining caused by scouring of the riverbed. Some of the scour prevention methods that should be considered in the design are increased depth of footing below streambed, stone fill for erosion control and cofferdam sheeting left in place. This requirement should also apply to abutments located in, or immediately adjacent to, the waterway.

#### 615.4.3 Passive Earth Pressure

Do not use passive earth pressure in stability calculations, unless approved otherwise by the Design Chief.

Passive earth pressure for stability may be included for seismic loading if the pier is not in a waterway. Check with the Design Chief for concurrence on using passive earth pressure for stability.

#### 615.4.4 Loading Conditions

Analyze all possible loading conditions, which may occur during construction and prohibit any conditions, which could dangerously overload any portion of the structure during that phase.

#### 615.4.5 Joints in Pier

Pier design should account for concrete shrinkage and constructibility of the wall.

#### 615.4.6 Pier Nose

An angle 8 x 8 x 3/4 in (203 x 203 x 19 mm) should be installed on the leading edge of the pier when the pier is located in the waterway or ice path.

#### 615.4.7 Development and Embedment Lengths

See Plates 601.1d, 601.1e, and 601.1f (601.1i, 601.1j, and 601.1k) for development and embedment lengths.

#### 615.4.8 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

## **620 PRESTRESSED CONCRETE STRUCTURES**

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### **620.1 General**

The use of precast prestressed concrete bridge members should be considered when doing structure type studies for bridge replacement and rehabilitation projects.

### **620.2 Typical Sections**

Prestressed precast concrete bridges would include standard AASHTO deck (solid and voided slab) beams, box beams and New England bulb-tee girders. Other sections may be used only with the approval of the Design Chief.

Standard New England bulb-tee girders are detailed in Plates 620.2a and 620.2d.

## 620.3 Materials

### 620.3.1 Concrete

Typical designs should be based on a 28-day concrete compressive strength  $f'_c$  of 6,000 psi (40 MPa). Concrete compressive strengths greater than this may be used with the approval of the Design Chief. The maximum design concrete compressive strength at release  $f'_c$  should be 4,800 psi (32 MPa). Design the required compressive strength at release as close to 30 MPa (4,000 psi) as practicable. Concrete release strengths greater than  $0.8 f'_c$  should not be used unless approved otherwise by the Design Chief. Plans should be noted with the actual release strength required for design.

A 28-day concrete compressive strength  $f'_c$  of 8,000 psi (55 MPa) may be used with the approval of the Design Chief.

### 620.3.2 Prestressing Steel

Prestressing steel should be uncoated, seven-wire, low relaxation strands meeting the requirements of AASHTO M 203 (ASTM A 416), Grade 270 (AASHTO M 203M (ASTM A 416 M) Grade 1860), low relaxation.

Strands for NEBT should be 0.6 in (15 mm) diameter.

Strands for deck panels should be 3/8 in (9.5 mm) diameter.

Strands for all other members should be 1/2 in (13 mm) diameter.

### 620.3.3 Reinforcing Steel

All non-prestressed reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (AASHTO M 31M Grade 420), epoxy coated. Detail the members so that all reinforcing steel is adequately embedded, developed and lapped.

## 620.4 Guidelines for Design

The design of prestressed concrete members should be in accordance with the latest AASHTO Standard Specifications except as modified herein.

### 620.4.1 Allowable Stresses

Design for zero tension in the precompressed tensile zone under final service conditions after all losses have occurred. Tensile stresses greater than this, up to  $6\sqrt{f'_c}$ , may be used only with the approval of the Design Chief.

### 620.4.2 Control of Tensile Stresses

For the reduction of tensile stresses at the ends of members either draped or partially debonded strands may be used. Mixing draped and partially debonded strands in a member is permitted.

If draping of strands is used, the total hold-down forces should not exceed 75% of the member weight.

#### 620.4.3 Partial Debonding of Prestressing Steel

The use of partially debonded strands should be subject to the following requirements:

1. The number of debonded strands should not exceed 25% of the total number of strands.
2. The number of debonded strands in any horizontal row should not exceed 40% of the strands in that row.
3. Debonded strands should be symmetrically distributed about the centerline of the member. Debonded lengths of pairs of strands, symmetrically positioned about the centerline of the member, should be equal. If practicable, debonded lengths of pairs of strands should be staggered within each row. This maximum debond length may be increased with approval of the Design Chief if it is under special circumstances, shown that the cracked section stays outside of the debond/transfer zone and there is also adequate embedment from the critical section to the end of the debond zone. See (FHWA-RD-94-049, "Analysis of Transfer and Development Lengths for Pretensioning Concrete Structures", page 71).
4. Exterior strands in each horizontal row should be fully bonded.
5. Center-to-Center spacing between debonded strands in a horizontal row should be 4 in (100 mm) or greater.
6. Debonded lengths at each end of the member should be limited to 15% of the member length.

#### 620.4.4 Horizontal Shear

The design of cross-sections subjected to horizontal shear should be in accordance with AASHTO 8.16.6.4, Shear Friction. The requirements of AASHTO Section 9.20.4.5 (a) and (b) Ties for Horizontal Shear, should also be satisfied.

#### 620.4.5 Development of Prestressing Steel

The minimum development length of the prestressing steel should be determined as 1.6 times AASHTO equation (9-42), Section 9.28. for fully bonded strands and 2.0 times equation (9-42) for debonded strands per AASHTO 9.28.3.

#### 620.4.6 Camber and Deflection

Camber and deflection should be calculated according to the following:

1. Deck beam and box beam superstructures: Camber and deflection should be calculated according to the multiplier method as outlined in the PCI Design Handbook. Sufficient camber should be induced such that a net positive camber will remain in the member at the final stage.
2. Composite bulb-tee girder superstructures: Camber and deflection should be summarized in a table on the contract drawings. The table should include the camber and deflection calculated at the following stages:
  - a) At Transfer: Camber due to prestressing force at release minus the deflection due to the dead load of the member. The multipliers are as follows:

Upward camber due to prestress:	1.00
Downward deflection due to member weight:	1.00
b) At Erection: Unless more accurate information is available as to time of delivery of the members, use the camber that is present at approximately 30 days after release. The multipliers are as follows:	
Upward camber due to prestress:	1.80
Downward deflection due to member weight:	1.85
c) *At Final Stage: Long-term camber after all dead load is applied to the member. The multipliers are as follows:	
Upward camber due to prestress:	1.80
Downward deflection due to member weight:	1.85
Downward deflection due to noncomposite dead loads:	1.00
Downward deflection due to composite dead loads:	1.00

\*[Note: These multipliers reflect research which has shown that prestressed concrete bridge girders acting composite with concrete deck slabs do not deflect as suggested in the PCI Multiplier Method. The stiffness of the composite section minimizes the continued creep of the girder.]

Camber at erection should be used to calculate blocking distances. The blocking distance at the centerline of bearings should be of sufficient height to provide for 1 in (25 mm) minimum at midspan and camber tolerances need not be considered.

Bottom of slab elevations should be determined using the calculated deflections of the deck slab and composite dead load without multipliers.

#### 620.4.7 Bearings

Bearings for prestressed concrete members should be elastomeric-type designed in accordance with AASHTO, Section 14 and 640 of this document.

For deck beam and box beam superstructures, bearings should be provided at both ends of each member for proper seating. For spans greater than 50 ft (15 m) use two bearings set sufficiently apart to prevent "rocking". For spans  $\leq$  150 ft (5 m) use strip bearings.

#### 620.4.8 Diaphragms

Intermediate diaphragms should be placed at midspan for all spans greater than 40 ft (12 m). Intermediate diaphragms should be placed normal to the members.

End diaphragms should be required on all spans and should be placed parallel to, and as close as practicable to, the centerline of bearings.

For simple-span members made continuous in multiple span bridges, consideration should be given to the placement of temporary diaphragms near the ends of members at piers in order to provide stability until the final end diaphragms are cast-in-place. Concrete end diaphragms, designed as part of the continuity connection, should be cast-in-place at the same time as the deck slab.

#### 620.4.9 Continuity

The design of simple-span members made continuous in multiple span bridges should be in accordance with the following:

1. Application of Loads: Members should be assumed to act as simple-spans for the application of the prestressing forces, member dead load and non-composite dead loads. Continuous members should be assumed to carry the superimposed dead loads, live loads and time-dependent effects (creep and shrinkage).
2. Positive Moment Connection: Provide a positive connection at interior supports sufficient to develop, as a minimum, a moment equal to  $1.2 * M_{cr} (+)$  of the composite section. The positive moment connection should be made by extending the prestressing steel beyond the member ends. The connection capacity should be determined using the method outlined in FHWA-RD-77-4, "End Connections of Pretensioned I-Beam Bridges".
3. Negative Moment Connection: The amount of negative moment reinforcing steel required in the deck should be determined by assuming the member to be a rectangular section with a compression block width equal to the bottom flange width of the member. The concrete compressive strength ( $f'c$ ) used should be that of the member. Both top and bottom mats of deck slab reinforcing steel within the effective flange width should be included in calculating the moment capacity of the section.
4. Time-Dependent Effects: Time-dependent effects should be accounted for using the method outlined in the PCI Journal, April 1969, "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders". With the approval of the Design Chief, these effects may be neglected if provisions are made in the contract to ensure that the concrete members cure for a minimum of 60 days prior to the application of additional dead loads.

#### 620.4.10 Deck Beams and Box Beams

For butted beam superstructures, the beams should be placed to follow the roadway cross-slope as much as practicable. A high-density concrete overlay should be placed on top of the beams and should extend from curb to curb. The minimum thickness of the overlay should be 3 in (75 mm) and should be considered non-structural. For live load deflection calculations, the overlay may be considered as part of the structural section.

Transverse reinforcement in the top slab of box beams should be designed for a concentrated AASHTO wheel load.

#### 620.4.11 Deflection

Members having simple or continuous spans should be designed so that the deflection due to service live load plus impact should not exceed 1/1200 of the span, except on bridges in urban areas used in part by pedestrians where the ratio preferably should not exceed 1/1600 unless approved by the Design Chief.

## 620.5 Precast Deck Panels

The Department's approach in the use of precast deck panels has been to take a methodical, step by step approach (proceeding from lower volume, less critical locations towards more critical bridge installations) in instituting the use of these panels over time. In keeping with this philosophy, the Department continues to take steps towards expanding the number of sites where panels may be used but proceeding in a cautious fashion that allows proper evaluation of past installations prior to proceeding to more difficult or important sites.

The Department will provide for a bid item that allows for either a CIP or SIP panel alternative, only as provided on the plans. (i.e. The Department will make the decision when it is appropriate to allow the SIP panels to be used as an alternate system.) The Deck Panel Detail Standard sheet shall be included in the Contract Plans when the SIP panel alternative is part of the contract. The following criteria should be used to determine when this item may be included in the Contract.

Criteria for when to include an item for CIP or SIP panel alternative:

1. For Low or Medium Truck Volume Bridges: (Current ADTT <400).

With High Truck Volume Bridges: (Current ADTT >400) the Department will proceed as follows:

- Monitor installations on NH Rte 51 EB over I-95 & at Tilton Exit 20 (US Rte 3 & NH 11 over I-93).
  - Construct experimental installations on:
    - 1) I-93, shorter span for construction in 2002 (bare deck).
    - 2) I-393, long span for construction in 2002.Provide instrumentation and monitoring for these decks and evaluate for 5 years.
2. Maximum Girder Spacings: Steel =10 ft-0 in; Concrete =12 ft-0 in.
  3. Design Deck Thickness: 8 1/2 in (3 1/2 in + 5 in) for deck panels; \*8 in for CIP deck.
  4. Minimum Girder Flange Width: 12 in (Need 6 in between panels).
  5. Bridge Length (Centerline to Centerline of bearings): Simple Span: Maximum 150 ft.  
Multi Span: Maximum of 175 ft for any individual span of the structure (or if ADTT <100, no limit of span length).
  6. Straight Girder Bridges Only.
  7. Deck cross-slope <.04
  8. Minimum Panel Length: 4 ft-6 in.

\*Girders should be designed for 1/2 in of additional WDLNC. In order to allow for field adjustments between the two options, an additional 1/2 in of haunch height should be provided when detailing the CIP option. (This additional 1/2 in should not be used as part of the composite section properties).

### 620.5.1 Design and Detail Requirements

1. Precast prestressed concrete deck panels used as permanent forms spanning between girders should be designed to act composite with the cast-in-place portion of the deck slab.
2. The concrete compressive strength at release  $f'c$  should be minimum 4000 psi (27.5 MPa). The 28-day concrete compressive strength  $f'c$  should be 6,000 psi (40 MPa).
3. Tension in the precompressed tensile zone under final service conditions after all losses have occurred should be  $\leq 6\sqrt{f'c}$ . Compression in the panel at release should be  $\leq 750$  +/- psi (5.2 MPa).
4. Prestressing strands shall be 3/8 in (9.5 mm) diameter and should extend 4 in (100 mm) minimum outside the panel edges.
5. Panel length should be set to provide a minimum grout bed width of 2 in (50 mm). This requires a minimum of 3 in (75 mm) from the edge of panel to the edge of steel girder flange and 3.5 in (87 mm) to the edge of a bulb-tee flange, assuming a 1 in (25 mm) minimum grout dam width.
6. The minimum thickness of the cast-in-place portion of the slab should be 5 in (125 mm).
7. The minimum thickness of the panel should be 3 1/2 in (90 mm). The top surface shall receive a broom finish.

### 620.5.2 Temporary Support System

Two Step System: After setting deck panels, a grout bed is placed in the girder haunch area and allowed to cure prior to placing the remainder of the deck.

1. Temporary supports for precast deck panels shall consist of continuous, high-density, expanded polystyrene strips (grout dam) with a minimum compressive strength of 55 psi (380 Pa). If leveling screws are used a 1.7 pound per cubic foot (27.2 kg per cubic meter) polyethylene foam seal shall be used as a grout dam. An approved adhesive should be specified to affix the dam to the girder and the deck panel.
2. If leveling screws are used temporary bracing between the ends of panels shall be installed as required to prevent transverse panel movement that could lead to loss of bearing on the leveling screws.
3. Deck panels should be specified to be grouted in place prior to placement of the cast-in-place concrete deck. The grout bed should extend for the full width of the girder flange completely filling the area between the grout dams. The top of the grout bed should be 1 in (25 mm) below the strand extensions.
4. If leveling screws are used they shall be completely removed after the grouting operation and prior to deck placement.

### 620.5.3 Haunch Height

The haunch height should take into account the following factors:

1. A minimum midspan blocking distance of 1 in (25 mm) plus an allowance for cross-slope and camber tolerance is required in order to obtain a minimum grout bed height of 1 in (25 mm).
2. The flange bolts for field splices of steel girders may interfere with precast concrete deck panels. Provide a haunch depth that accommodates the added thickness of the field splice. Modifying panel thickness to avoid conflict with field splices shall not be allowed. To avoid this conflict, the splice bolts may be installed with the bolt head oriented to be on the top splice plate (i.e. install bolts down through the splice rather than up through the splice).

### 620.5.4 Deflection Breakdown Table

A deflection breakdown at each .3, .5 and .7 point location, which lists separately the dead load deflection of the deck panels, cast-in-place slab, and composite loads, should be shown within a table on the contract plans.

### 620.5.5 Deck Reinforcement

The top mat of reinforcing steel as specified in Plate 650.4e (650.4f) should be provided in the cast-in-place portion of the slab.

For continuous bridges with precast deck panels, the requirements of AASHTO 10.38.4.3 should be modified to provide  $\frac{2}{3}$  of the "1% of the cross-sectional area of the concrete slab" in the top mat. The remaining  $\frac{1}{3}$  may be ignored. Maximum bar size allowed is a #6.

## **630 Structural Steel**

### **630.1 General**

### **630.2 Typical Sections**

- 630.2.1 Rolled Beams
- 630.2.2 Welded Plate Girders
- 630.2.3 Plate Availability
- 630.2.4 Tub Girders
- 630.2.5 Shear Connectors

### **630.3 Materials**

- 630.3.1 Steel
- 630.3.2 Weathering Steel
- 630.3.3 Steel Material Designations
- 630.3.4 Painting Structural Steel
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### **630.4 Design Guidelines**

- 630.4.1 Composite Design
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- 630.4.5 Bolted Field Splices
- 630.4.6 Welded Shop Splices
- 630.4.7 Stiffeners and Connection Plates
- 630.4.8 Notch Toughness Requirements
- 630.4.9 Welding and Fabrication
- 630.4.10 Deflections
- 630.4.11 Cover Plates
- 630.4.12 Shear Connectors

### **630.1 General**

The use of structural steel should be considered as a superstructure type for bridge replacement and rehabilitation projects.

### **630.2 Typical Sections**

#### 630.2.1 Rolled Beams

[Blank]

### 630.2.2 Welded Plate Girders

1. The minimum web plate thickness should be 1/2 in (12 mm).
2. The minimum flange plate thickness should be 3/4 in (20 mm).
3. The ratio of cross-sectional areas of flanges at a width or thickness transition should not exceed 1 to 2.
4. The maximum change of thickness of flange plates at a welded splice is 3/4 in (20 mm) for the thicker plate up to 2 in (50 mm) thick, and 1 in (25 mm) for the thicker plate over 2 in (50 mm) thick.
5. To avoid cracking of haunch girder flanges and to facilitate flange-to-web fit, a minimum flange bending radius should be detailed on the contract plans (see Plate 630.2a).
6. The minimum width of flange plates is 12 in (300 mm).
7. The minimum size of fillet weld is 1/4 in (6 mm) for plates up to a thickness less than 3/4 in (19 mm) and 5/16 in (8 mm) for a thickness of 3/4 in (19 mm) and up.

### 630.2.3 Plate Availability

AASHTO M 270 Grade 36, 50, 50W (M 270/M 270M Grade 250, 345 and 345W) plates are generally available in the following thickness and lengths:

Thickness Increment	Plate Thickness From	Plate Thickness To
in	in	in
0.0625	0.5	0.875
0.125	0.875	1.25
0.25	1.25	3.0

Metric plate thickness should be as follows (mm):

12, 14, 16, 18, 20, 22, 25, 28, 30, 32, 35, 40 (over 40 use 5 mm increments).

Plates are only available in a minimum width of 4 ft (1.2 m) and in maximum lengths of 90 ft (27.4 m). The usual method of welding flanges, therefore, is to weld wide plates (slabs) together and strip out flanges to their final width. Welded splices may be necessary to achieve the desired lengths (see chart). The fabricator should be given the option of eliminating a thickness-transition butt splice and continuing the heavier plate full length for cost savings.

Plates	Thickness		Max. Plate Length Available		In Plate Widths	
	mm	in	m	ft	m	ft
Web	12 to 14	0.5 to 0.56125	27.4	90	3.0	10.5
	16 to 19	0.625 to 0.75	27.4	90	3.2	10.5
Flange	19 to 44	0.75 to 1.75	27.4	90	1.2	4
	50 to 64	2 to 2.5	22.9	75	1.2	4

#### 630.2.4 Box Girders

Box girders have not been used in the past by the Department (See 450.1).

#### 630.2.5 Shear Connectors

Shear connectors should be 22 mm (7/8 in) diameter for composite design. (See Plate 630.2b.)

### 630.3 Materials

#### 630.3.1 Steel

A standard design should be based on a  $F_y$  of 50,000 psi (345 MPa) using AASHTO M 270 Grade 50 or 50W (M 270M Grade 345 and 345W) steel. The preferable design should use M 270 Grade 50W (M 270M Grade 345W) (i.e. weathering steel) with the ends painted at expansion joint locations susceptible to leaking (e.g. strip seals and finger joints).

#### 630.3.2 Weathering Steel

The use of unpainted weathering steel should follow the recommendations presented in FHWA Technical Advisory T5140.22, dated October 3, 1989, such as avoiding the following conditions:

1. Marine coastal areas with salt air environment.
2. Industrialized areas with a corrosive atmosphere.
3. Height clearance over stagnant water levels less than 10 ft (3 m) and over moving water less than 8 ft (2.4 m).
4. Areas of constant wetness or vegetation cover where steel cannot dry out.

Drip bars should be used on the low end of all girder lines.

##### 630.3.2.1 Painting for Weathering Steel

For weathering steel bridges with an expansion joint, the ends of the girders, bridge shoes, and cross frames at the expansion joint should be painted, except for the fascia surfaces of the exterior girders.

The convention for the painted length of girder ends is a distance from the center line of bearing equal to 1-1/2 to 2 times the beam depth with a 6 ft (2 m) minimum, to the nearest even foot (0.1 m) unless directed otherwise. Consideration should be given for skews and the painted area should extend to include drip bars.

The designer should note that when painting is required due to the requirements of this section the top flange embedded in concrete is to be painted with a light rust-preventative coat of primer (See 630.3.4) in the length of beam to be painted.

For integral and semi-integral steel girder bridges, the ends of the girders out a distance 12 in (300 mm) from the face of abutment should be shop painted with a full coating system.

#### 630.3.3 Steel Material Designations

AASHTO requires that structural steel meet the requirements of AASHTO M 270(M 270M), Structural Steel for Bridges. The designer should be aware that the previous designations

(AASHTO M 183, M 223 Grade 50, M 222 and ASTM A36, A572 Grade 50, A588 respectively) are still available for use, particularly for secondary members and ancillary products. The principal difference between the "new" and "old" specifications are summarized as follows:

- All bridge steels are now listed under one specification.
- The supplementary requirements regarding Charpy V-notch testing are automatically included in the M 270 specification,
- Weathering steel is now designated with the letter "W",
- Killed fine grain practice is automatically specified in the manufacturing process.

### 630.3.4 Painting Structural Steel

#### 630.3.4.1 New Steel

The requirements for painting new steel should be stated in the Special Provision for Section 550. This specification addresses coating system selection and application issues, briefly summarized as follows: Paint systems should meet the environmental VOC limits of the Clean Air Act (i.e., 3.5 lbs./gal, 420 g/L). Approved systems should meet the performance criteria of NEPCOAT testing and any other criteria established by Bridge Design and Materials & Research (see approved Systems A, B, and C below). Application requirements should meet SSPC standards.

System A:	NHDOT 708	Name of Coating	DFT (mils)
Primer	1.70	Inorganic zinc rich	3-5
Intermediate	3.21	High build epoxy polyamide	4-6
Finish	3.81	Aliphatic polyurethane	2-4

System B:	NHDOT 708	Name of Coating	DFT (mils)
Primer	1.20	Organic zinc rich primer	3-5
Intermediate	3.21	High build epoxy polyamide	4-6
Finish	3.81	Aliphatic polyurethane	2-4

System C:	NHDOT 708	Name of Coating	DFT (mils)
Primer	1.40	Zinc-rich; Single-part moisture-cure urethane	3-4
Intermediate	2.40	Aromatic SC MC Urethane w/MIO	3-4
Finish	3.43	Aliphatic SC MC Urethane	2-4

All paint systems should be non-lead, non-chromate systems.

Water-based systems should not be permitted, unless approved otherwise by the Administrator.

When structural steel is painted, the steel elements of the bridge shoes (except sliding surfaces) should also be painted with the same paint system.

#### 630.3.4.2 Connections

All bolted structural steel connections are considered 'slip critical'. The Department intends that on painted bridge members the faying surfaces of bolted connections should be painted with a primer coating having an AASHTO Class B slip coefficient.

#### 630.3.4.3 Existing Steel

The requirements for painting existing steel should be stated in the Special Provision for Section 556. This specification addresses lead paint pollution controls, coating system selection, surface preparation requirements, and application issues, briefly summarized as follows: Paint systems should meet the environmental VOC limits of the Clean Air Act (i.e. 3.5 lbs./gal, 420 g/L). Approved systems should meet the performance criteria established by Bridge Design and Materials & Research (see approved Systems B and C above). Application requirements should meet SSPC standards. Lead paint activities should conform to EPA environmental regulations and OSHA worker health and safety requirements.

#### 630.3.5 Buy America

The "Buy America" requirement should be used on Federal Aid projects but not on non-Federal Aid projects. When "Buy America" does not apply, foreign material supplied and fabricated should be certified to meet quality standards equal to AASHTO and all documentation should be written in the English language.

### **630.4 Design Guidelines**

The design of structural steel members should conform to the latest AASHTO Standard Specifications and NHDOT Standard Specifications except as modified herein.

#### 630.4.1 Composite Design

Girders and rolled beams should be designed to act in composite action with the concrete deck. 100% of the reinforcing steel may be used to determine section properties.

#### 630.4.2 Continuous Spans

The design of continuous span bridges should include composite action in the negative moment region. Pay particular attention to the fatigue stress at the shear connector weld.

#### 630.4.3 Optimization of Material

1. Weight reduction of material is not the final consideration in determining span type selection or girder design. Material weight of the stringers may represent up to 25% of the completed, in-place cost of structural steel. Most of the cost is in fabrication, shipping, and erection. Initial construction cost "savings" in reduction of material weight may be lost after adding fabrication, testing, inspection, and erection costs. Use of simple details, reduction of fabrication operations, and ease of erection may be a better way to minimize final costs.
2. For further information, refer to AISC *Modern Steel Construction* magazine issues September 1992, March 1994, January 1997, and September 1997; also materials or contacts from the NSBA.

3. The recommended practice for flange design is to maintain a uniform width and only change the thickness at a welded splice to reduce fabrication costs.
4. A change in flange thickness may be made at a bolted field splice by using a filler plate. A filler plate should have the same width as the flange and a minimum thickness of 1/4 in (6 mm). Only one plate is permitted at one location. Filler plates should not be permitted in a web splice (due to concern for the thin plates involved, possible handling damage, and susceptibility to corrosion section loss).
5. Changes in flange width may be made at a bolted field splice.
6. Consideration should be given to minimizing the number of welded flange splices.
7. A "rule of thumb" to consider is that (in 1998) a welded flange butt splice stripped from a slab costs approximately \$375 - \$400 that is roughly equivalent to the cost of 1,050-1,150 pounds (475-575 kg) of plate at \$ 0.35 per pound (\$0.75 per kg).

#### 630.4.4 Fatigue

1. All structural steel details should be checked for AASHTO allowable stress range and fatigue criteria.
2. The minimum fatigue design case considering AASHTO Table 10.3.2A should be Case II, 500,000 cycles. Use traffic numbers that are for the 20-year design.

#### 630.4.5 Bolted Field Splices

1. All bolted bridge connections in primary and secondary members are considered "slip-critical" for painted bridge components.
2. The location of a bolted field splice should be adjusted to avoid the placement of shear connectors (see Plate 650.4c).
3. Direct Tension Indicators (DTI's) shall be used for all bridge connections.
4. Reuse of High Strength Bolts. The NHDOT Standard Specifications (550.3.11.6.6) do not permit the reuse (i.e. retightening) of ASTM A325 bolts. However, the designer should be aware that AASHTO (Div. II 11.5.6.4.1) permits the reuse of ASTM A325 (A325M) bolts one time only. The following comment by Bethlehem Steel and NYDOT may be helpful in determining the suitability of reuse in limited situations, if permitted by the Design Chief:

Comment: "Tests have demonstrated that ungalvanized ASTM A325 (A325M) bolts which have been tightened previously by the turn-of-nut method can usually be reused once provided no significant permanent elongation or "stretch" has occurred in the threads during previous use. The bolt and nut shall be removed for visual inspection of the bolt and nut threads for damage. A practical means of determining whether "stretch" has occurred is to assemble the nut on the bolt. If the nut runs freely the full length of the threads when turned by hand without the aid of a wrench, the bolt has not been stretched. Undamaged fasteners may be re-installed one time."

#### 630.4.6 Welded Shop Splices

1. The location of bolted field splices and the extension of thicker flanges should be such that the distance from a welded splice to the centerline of a bolted field splice should be 5 ft (1.5 m) minimum, or the distance from a welded splice to the nearest bolt hole should be 1 ft (0.3 m) minimum, whichever is greater (see Plate 630.4b).
2. There should be a minimum distance of 9 in (225 mm) between a welded splice in a web and a welded splice in a flange (see Plate 630.4b).
3. Transverse stiffeners or connection plates should be placed a minimum distance of 6 in (150 mm) from a welded splice in the web or flange (see Plate 630.4b).
4. The preferred detail for a welded flange splice at a width transition is to use a 2 ft-0 in (600 mm) radius starting 3 in (75 mm) from the butt weld. The 3 in (75 mm) offset permits the use of run-off tabs during welding and edge blocks during radiographic testing. The radius makes it easier to grind a smooth transition after removal of the run-off tabs than with a straight bevel detail.

#### 630.4.7 Stiffeners and Connection Plates

1. For welded girder design, consideration should be given to eliminating transverse intermediate stiffeners and longitudinal stiffeners by designing a web of sufficient thickness. The use of transverse and longitudinal stiffeners should be approved by the Design Chief.
2. Weld Size - The weld size for stiffeners and connection plates should be governed by plate thickness and design stress according to AASHTO Specifications.
3. Intersecting Welds - Welds used for attachments should not intersect with main or secondary member welds. Corners should be coped or welds stopped short from the point of intersection by a minimum distance of 1-1/2 in (38 mm).
4. Transverse Stiffeners - Single or paired stiffeners may be used. For paired stiffeners, use the same detail as shown for a single stiffener (see Plate 630.4c). When longitudinal stiffeners are used, transverse stiffeners should all be placed on one side of the web and on the opposite side from the longitudinal stiffener (see Plate 630.4d).
5. Bearing Stiffeners - For straight girders, the bearing stiffener should be welded to both flanges at abutment and pier locations with a fillet weld on both sides. For horizontally curved girders the bearing stiffeners should be welded to the top flange with a fillet weld on both sides and welded to the bottom flange with a full penetration groove weld. Use the same corner cope dimensions as for the transverse stiffeners and connection plates (see Plate 630.4c).
6. Minimum plate thickness - The minimum thickness for transverse stiffeners, connection plates, and bearing stiffeners should be 1/2 in (12 mm). Thinner plates have been observed to distort due to the heat of welding them to the web.
7. Skewed plates - Intermediate transverse stiffeners and connection plates should be positioned normal to the web. On a skewed structure, the bearing stiffeners or end connection plates may be skewed up to 30° using fillet welds. When plates are skewed at angles greater than 30° to the web, partial penetration groove welds are required (which is

not recommended). A preferable detail would be to weld the stiffener normal to the web and use a bent gusset connection plate (see Plate 630.4a).

8. Support Plates - The details of the transverse stiffener should apply to utility supports, sign hanger supports, and scupper supports (see Plate 630.4c). If a utility has any appreciable weight (e.g. 12 in water line) the stiffener should extend the full height of the girder web.
9. Longitudinal Stiffeners
  - a) Longitudinal stiffeners should preferably be placed on one side of the web and all transverse stiffeners placed on the opposite side.
  - b) Longitudinal stiffeners should be continuous for their full length as shown on the plans unless interrupted by a girder field splice.
  - c) When longitudinal stiffeners must be spliced to achieve the required length, they should be assembled full length using complete penetration groove welds before attachment to the web with full length continuous fillet welds.
  - d) If a longitudinal stiffener is interrupted by a field splice, it should be terminated on each side of the splice with a radiused termination (see Plate 630.4d).
  - e) In areas of tension or stress reversal, longitudinal stiffeners should be continuous through intersecting transverse stiffeners or connection plates (see detail on Plate 630.4d). If this is not possible, a radiused termination should be used (see Plate 630.4d).
  - f) In areas of compression, longitudinal stiffeners should be terminated at the bearing stiffener with a full penetration groove weld.

#### 630.4.8 Notch Toughness Requirements

1. The notch toughness requirements of NHDOT Standard Specification 550.2.2 should apply for main load-carrying member components subject to tensile stress or stress reversal. NH is Zone 2.
2. For rolled beam structures, the notch toughness requirements should apply to the beams, cover plates, and splice plates.
3. For welded girder structures, the notch toughness requirements should apply to the web, tension flanges, splice plates and longitudinal stiffener plates in stress reversal zones.
4. For horizontally curved steel structures, the cross-bracing members and connection plates should also be considered as main load-carrying members subject to tensile stress. The notch toughness requirements should apply to those members.
5. For structures with a girder-floorbeam-stringer system, the notch toughness requirements should apply to the girder webs and tension flanges, floorbeams, stringers, and all connection components.
6. Both top and bottom flange plates should be subject to notch toughness requirements if they are the same size plate and could be interchanged during fabrication.

7. The members listed above subject to notch toughness requirements should be clearly identified on the structural steel drawings. The following typical notes are suggested:

(For rolled beam structures): "The notch toughness requirements of NHDOT Standard Specifications 550.2.2 apply to the rolled beams and cover plates."

(For welded girder structures): "The notch toughness requirements of NHDOT Standard Specifications 550.2.2 apply to the web, tension flanges, and splice plates of the main girders. These members are identified by the symbol (T)."

#### 630.4.9 Welding and Fabrication

All welding and fabrication should conform to the latest edition of AASHTO/AWS D1.5, "Bridge Welding Code", as modified by NHDOT "Standard Specifications for Road and Bridge Construction", with pertinent amendments and special provisions inclusive.

The Fabrication Inspection Engineer should be consulted as necessary for assistance in preparing welding and fabrication details for contract plans, and may review the completed contract plans and shop drawings.

All welds, particularly full penetration groove welds at butt splices, should be specified on contract plans in accordance with AWS.

#### 630.4.10 Deflections

Members having simple or continuous spans should be designed so that the deflection due to service live load plus impact should not exceed 1/1200 of the span, except on bridges in urban areas used in part by pedestrians where the ratio preferably should not exceed 1/1600 unless approved by the Design Chief.

#### 630.4.11 Cover Plates

Cover plates may be used on rolled beams if a larger rolled beam cannot accommodate vertical clearance.

Cover plates on rolled beams should be 2 in (50 mm) narrower than the beam flange unless otherwise directed and have square rather than tapered ends. The cover plate-to-flange connection should use a bolted end detail in lieu of an all around weld.

For single span bridges, extend the cover plate to within 5 ft (1.5 m) of the supports.

For continuous rolled beam bridges, cover plates should be avoided at the pier. If cover plates are used at the pier, extend the cover plate to the proximity of the inflection point, while checking fatigue criteria but maintaining a minimum clear distance from the end of the cover plate to the splice plate of 1 ft-0 in (300 mm).

#### 630.4.12 Shear Connectors

See 630.4.4 for fatigue design considerations. The contract plans should include a typical shear stud connector detail (see Plate 630.2b). Construction and welding requirements for shear connectors are found in AWS D1.5, Chapter 7.

## **631 Bridge Inspection Access**

### **631.1 Inspection**

In the design of bridges, consideration should be given to the accessibility of the bridge for inspection by Bridge Inspectors and Maintenance personnel. Inspections are usually made with ladders or the Underbridge Inspection Crane ('Snooper'). For reach limits of the inspection crane, see Plate [631.1a](#).

Future inspection needs may require handrails and/or access walkways to be provided if conventional accessibility is unavailable.

## **640 Bridge Shoes**

### **640.1 General**

### **640.2 Typical Sections**

640.2.1 Elastomeric Bearing

640.2.2 Steel Sliding Shoes Assembly

### **640.3 Materials**

640.3.1 Natural Rubber

640.3.2 Neoprene

640.3.3 Steel

640.3.4 Stainless Steel

640.3.5 Anchor Bolts

### **640.4 Guidelines for Design**

640.4.1 Abutment Tip

640.4.2 Deck Over Backwall Structures

640.4.3 Elastomeric Bearing

640.4.4 Steel Sliding Shoe Assemblies

640.4.5 Seismic Isolation Bearing

### **640.1 General**

Elastomeric bearings should normally be the first option considered when selecting bridge shoes. The elastomeric bearings give the best initial cost and least future maintenance cost.

### **640.2 Typical Sections**

#### **640.2.1 Elastomeric Bearing**

Both round and rectangular elastomeric bearings may be used. See 640.4 for more information.

Elastomeric bearings should have top and bottom cover layers of 6 mm (1/4 in.). A good starting point for the internal elastomer layer thickness would be 5/8 in (16 mm).

Elastomeric bearings should have side cover of 1/4 in (6 mm).

Elastomeric bearings steel reinforcing should be 1/8 in (3 mm).

If sole or masonry plates are used, they should be vulcanized to the elastomeric bearing. Steel girders should use a beveled sole plate to account for all profile grades in excess of 1%. Concrete girders should use a beveled top steel reinforcing plate to account for all profile grades in excess of 1%.

#### **640.2.2 Steel Sliding Shoes Assembly**

Steel sliding plate bearings should be used when elastomeric bearings cannot meet the design loads.

### **640.3 Materials**

#### **640.3.1 Natural Rubber**

Elastomeric compound should be 100 percent virgin natural rubber, Grade 3, with Hardness (Shore "A" Durometer) of 60.

### 640.3.2 Neoprene

Elastomer should be neoprene, Grade 3, with Hardness (Shore “A” Durometer) of 60.

### 640.3.3 Steel

Typical designs should be based on a  $F_y$  of 50,000 psi (345 MPa) with the use of AASHTO M 270 Grade 50 or 50W (AASHTO M 270M Grade 345 or 345W), the preferable steel being AASHTO M 270 Grade 50W (AASHTO M 270M Grade 345 or 345W) (weathering steel).

### 640.3.4 Stainless Steel

Stainless steel for bridge shoes should be ASTM A 240 Type 304 (ASTM A 240M Type 304) with a #8 mirror finish on the sliding surface.

### 640.3.5 Anchor Bolts

Anchor bolts for bridge shoes should be fabricated from deformed billet steel reinforcing bars conforming to AASHTO M 31 (ASTM A615), Grade 60 (AASHTO M 31M Grade 420), and should be galvanized.

## 640.4 Guidelines for Design

### 640.4.1 Abutment Tip

In addition to the amount of movement required for temperature, some provision should be made for possible abutment tip. For each 10 ft (3 m) of height, allow 1/4 in (6 mm) for rotation, or more for unusual conditions. Foundations, which provide sure resistance to lateral movement, may use one-half these values (i.e. 1/8 in (3 mm)). Provisions for abutment tip do not need to be included in bridge rehabilitation projects.

### 640.4.2 Deck Over Backwall Structures

For structures where the deck extends over the backwall and the compression of the elastomeric bearing exceeds 1/8 in (3 mm) under live load, superimposed dead load and creep, a means of accommodating this compression should be included between the backwall and the deck if elastomeric bearings are to be used.

### 640.4.3 Elastomeric Bearing

Elastomeric bearings should be designed with Grade 3 Natural Rubber or Grade 3 Neoprene. NH is considered Low Temperature Zone C.

On normal bridge layouts the bridge deck is not free to translate per the following e-mail from Dr. Roeder, “The concept of no translation does not mean that there must be no movement. It means that the movement in that direction must be limited by some positive restraint somewhere on the bridge.”

If the sole plate or top steel reinforcing plate is not beveled to the profile grade, the bearing should be designed for this additional amount of rotation.

Dead load rotation should not be accounted for in the design of bearings if the girder is cambered for full dead load deflection. If the girder is not cambered for all of the dead load rotation, the bearing should be designed for the remaining dead load rotation.

#### 640.4.3.1 Steel Girder Structures

For steel girder structures, shoes may be elastomeric bearings. Round bearings are a good choice for skewed structures; however, round bearings are typically not a good choice for structures with a large amount of live load rotation.

#### 640.4.3.2 Concrete Girder Structures

Round elastomeric bearings are typically a good choice for concrete girder structures.

### 640.4.4 Steel Sliding Shoe Assemblies

#### 640.4.4.1 Masonry Plates

All bridge shoe masonry plates should have slotted anchor bolt holes with a welded plate washer to allow the plates to be repositioned in the future to correct for any differential motion between the substructure and the superstructure, such as abutment tip. See Plate 640.4b (640.4a) for more information.

#### 640.4.4.2 Sole Plates

When sole plates are shop welded to the bottom flange, the transverse weld is considered the primary strength weld. The sole plate should either be wider or narrower than the flange to allow for placement of a longitudinal weld 5/16 in (8 mm) minimum.

#### 640.4.4.3 Girder Bottom Flange

The bottom flange on a steel plate girder should be narrowed to 12 or 14 in (300 mm or 350 mm) at the abutment bearing locations. See Plate 630.4g.

#### 640.4.4.4 Stainless Steel

When specifying a 3/16 in (5 mm) thick stainless steel plate, attach to the sole plate with 1/8 in (3 mm) fillet weld all around.

#### 640.4.4.5 Preformed Fabric Pad

When using the 100% Virgin Teflon bonded to a preformed fabric pad, call for a confining substrate, which is bonded between the Teflon and the preformed fabric pad. (Suggested Note: "100% Virgin TFE (Teflon) bonded to preformed fabric pad with confining substrate.")

#### 640.4.4.6 Inclined Grades

For spans on inclined grades greater than 1%, the sole plates should be beveled so that the bottom of the sole plate is level, even if the bottom of the sole plate is radially curved.

#### 640.4.4.7 Expansion Shoe Minimum Gap

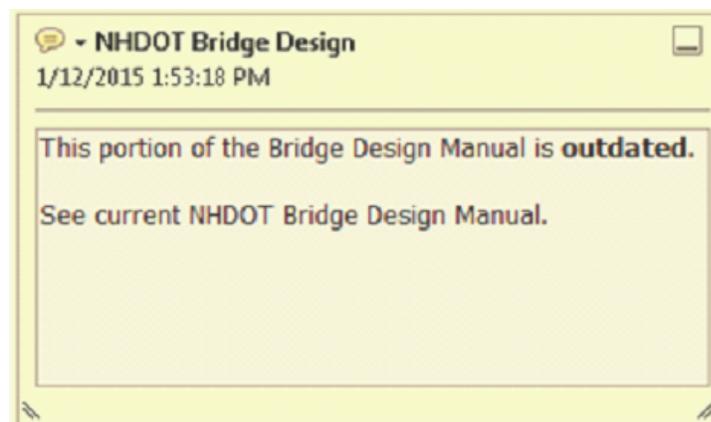
The minimum gap between the teflon pad on the keeper plate and the stainless steel plate on the sliding plate should be 5/16 in (8 mm).

#### 640.4.5 Seismic Isolation Bearing

For guidelines on seismic isolation bearings see 603.5.

## 641 Expansion Joints

- 641.1 General**
- 641.2 Typical Sections**
  - 641.2.1 Elastomeric Plug Joint
  - 641.2.2 Strip Seal
  - 641.2.3 Finger Joint
  - 641.2.4 Modular Joint
- 641.3 Materials**
  - 641.3.1 Steel
  - 641.3.2 Coating
  - 641.3.3 Anchorage
- 641.4 Guidelines for Design**
  - 641.4.1 Temperature Range
  - 641.4.2 Mid-Point
  - 641.4.3 Abutment Tip
  - 641.4.4 Joint Location
  - 641.4.5 Located on the High End
  - 641.4.6 Concrete Apron
  - 641.4.7 Trough Slope



### 641.1 General

The use of expansion joints should be avoided if possible. Plow plates will not be allowed.

### 641.2 Typical Sections

#### 641.2.1 Elastomeric Plug Joint

The elastomeric plug joint system should be the preferred joint when joints are required on a bridge. See Plate 641.2b (641.2a) for the limitations on using the plug joint system.

#### 641.2.2 Strip Seal

The strip seal joint system is less expensive than the finger joint and allows movements up to 3 in (75 mm). The strip seal has limitations concerning snow removal operations. Plow plates will not be allowed on the strip seal joint system. See Plate 641.2b (641.2a) for the limitations on using the strip seal joint system.

#### 641.2.3 Finger Joint

The finger joint-elastomeric trough system should be considered as an alternative to the strip seal where large movements are involved.

The finger joint-elastomeric trough system should be used when there are large movements (greater than 3 in (75 mm)) and the skew angle is 25° or greater, in order to prevent the snowplows from catching the expansion joint. See Plate 641.2b (641.2a) for the limitations on using the finger joint-elastomeric trough system. Minimum finger overlap should be 2 in (50 mm). The minimum joint opening (maximum design temperature) in the longitudinal direction should be 1 in (25 mm). Bicycle safety needs to be considered when using this joint.

#### 641.2.4 Modular Joint

The modular joint system is an alternative to the finger joint which accommodates movements greater than 3 in (75 mm). The modular joint system has limitations concerning snow removal operations. Plow plates will not be allowed on the modular joint system. See Plate [641.2b](#) (641.2a) for the limitations on using the modular joint system.

### 641.3 Materials

#### 641.3.1 Steel

Typical designs should be based on a  $F_y$  of 36,000 psi (250 MPa) with the use of AASHTO M 270 Grade 36 (M 270M Grade 250).

#### 641.3.2 Coating

All expansion joints, except for elastomeric plug joints, should be required to be painted with one coat of approved zinc-rich primer (all strip seal and modular assemblies) or galvanized (finger joint assemblies).

#### 641.3.3 Anchorage

Anchorage to the backwall and deck (between curbs) should be made using rebar and should be spaced at 1 ft-0 in (300 mm) maximum. Brush curb and sidewalk anchorages should be made of stud anchors and should be spaced at 1 ft-6 in (450 mm) maximum. The anchorage reinforcement should extend into the backwall or curb reinforcement cage for proper anchorage.

### 641.4 Guidelines for Design

Expansion joints should be eliminated if possible with the use of integral or semi-integral abutment design. See 613.5 for more information.

The guidelines on Plate 641.2a (641.2b) are working guidelines and meant to be flexible. Adjustments to these guidelines are to be made as experience is gained and new technology becomes available.

#### 641.4.1 Temperature Range

Expansion joints should be designed to accommodate a temperature range from -30°F to +120°F (-35°C to +50°C) in steel girder bridges and 0°F to +80°F (-18°C to +27°C) in concrete girder bridges.

#### 641.4.2 Mid-Point

Set expansion joints at the mid-point of the temperature range at 45°F (8°C).

For strip seals, check the manufacturer's minimum installation width against the set temperature range; an adjustment of the mid-point may be required.

#### 641.4.3 Abutment Tip

In addition to the amount of movement required for temperature, some provision should be made for possible abutment tip. For each 10 ft (3 m) of height allow 1/4 in (6 mm) for rotation, or more for unusual conditions (for example lateral squeeze). Provisions for abutment tip do not need to be included in bridge rehabilitation projects.

#### 641.4.4 Joint Location

Bridges should be designed to avoid placement of expansion joints over piers whenever possible. If an expansion joint is required at a pier, the finger joint-elastomeric trough system should be used.

On continuous spans, the pier can be fixed to allow the use of elastomeric joints at the abutments.

#### 641.4.5 Located on the High End

The expansion joint should typically be located on the high end of the bridge.

On a one-way bridge, the expansion joint should be located on the departure end.

Provide PVC drains at all expansion joints, including plug joints when the joint is located at the low end.

#### 641.4.6 Concrete Apron

The top of expansion joints should be set flush with a concrete apron on each side of the joint. The top surface of the joint and concrete apron should be set 1/8 in (3 mm) below finished grade. For joints where the asphalt-wearing course comes in contact with the steel edge beam (usually a channel or angle), the edge of the top flange of the beam will be chamfered to provide an inclined surface to guide snowplows up and across the joint (See standard detail sheet for more information.).

#### 641.4.7 Trough Slope

Finger joint troughs should have a minimum slope of 3/8 in per ft (3 mm per 100 mm).

## **642 Bridge Rail and Bridge Approach Rail**

### **642.1 General**

### **642.2 Typical Sections**

- 642.2.1 Steel Bridge Rail
- 642.2.2 Aluminum Bridge Rail
- 642.2.3 Texas T101 Rail
- 642.3.4 Concrete Barrier
- 642.2.5 Timber Bridge Rail
- 642.2.6 Steel-Backed Timber (Roadway) Rail

### **642.3 Materials**

- 642.3.1 Steel
- 642.3.2 Aluminum
- 642.3.3 Concrete
- 642.3.4 Timber
- 642.3.5 Weight

### **642.4 Design Guidelines**

- 642.4.1 Background
  - 642.4.1a Reference Documents
  - 642.4.1b Timeline
  - 642.4.1c Rail Level Selection Procedures
  - 642.4.1d Crash Tested Systems
  - 642.4.1e FHWA Requirements
- 642.4.2 NHDOT Practice
- 642.4.3 Pedestrian Rail
- 642.4.4 Protective Screening & Snow Fence
- 642.4.5 BLANK
- 642.4.6 Geometry
- 642.4.7 Bikeway Rail
- 642.4.8 Rail Finishes
- 642.4.9 Concrete Barrier
- 642.4.10 Loads Transmitted to Substructure
- 642.4.11 Concrete Curb

### **642.1 General**

The selection and use of bridge rail and bridge approach rail systems involve a number of factors, including the following: (See discussion in Section 642.4.1.)

- design criteria (AASHTO),
- crash-testing requirements (FHWA),
- rail level selection procedures (MSL-1, PL-2, TL-3, etc.),
- funding (Federal aid, state, etc.)
- geometry (brush curb, sidewalk, rail height, etc.), and
- aesthetics (historical, material, color, etc.)

## 642.2 Typical Sections

The NHDOT has standard detail sheets for each of the following bridge rail and bridge approach rail systems:

### Steel Bridge Rail

- a) T2 Steel Bridge Rail
- b) T4 Steel Bridge Rail
- c) ST Steel Bridge Rail (Use only with approval)

### Aluminum Bridge Rail

- a) 2-Bar Aluminum Bridge Rail
- b) 3-Bar Aluminum Bridge Rail

### Texas T101 Rail

- a) Texas T101 Bridge Rail with 7 in (175 mm) curb
- b) Texas T101 Bridge Rail with 3 in (75 mm) curb

### Concrete Barrier

- a) New Jersey Safety Shape
- b) Vertical Face Parapet

### Timber Bridge Rail

- a) Glu-lam Timber Bridge Rail on Timber Deck

### Steel-Backed Timber (Roadway) Rail

- a) Steel-backed timber rail

## 642.3 Materials

642.3.1 Steel. See Standard Detail Sheets for material specifications.

642.3.2 Aluminum. See Standard Detail Sheets for material specifications. Welding of aluminum other than specified shall not be permitted.

642.3.3 Concrete. See Standard Detail Sheets for material specifications.

642.3.4 Timber. See Standard Detail Sheets for material specifications.

642.3.5 Weight.

The estimated / calculated weight of rail to use for computing dead loads and quantities is approximately as follows:

- T4 steel bridge rail                      80 lbs per lin. foot                      (120 kg per lin. m).
- T2 steel bridge rail                      50 lbs per lin. foot                      (75 kg per lin. m).
- 3-bar aluminum bridge rail              35 lbs per lin. foot                      (50 kg per lin. m).
- 2-bar aluminum bridge rail              25 lbs per lin. foot                      (35 kg per lin. m).

## 642.4 Design Guidelines

### 642.4.1 Background

The selection of bridge rails requires an appreciation of the background of bridge rail development which is briefly summarized in 642.4.1. The selection process is somewhat confusing because (1) each guidance or specification document has introduced different terminology and requirements which do not directly correlate from document to document; and (2) crash testing procedures are separate from (but closely related to) the rail level selection procedures.

a) <u>Reference documents.</u>	<u>(Abbreviation)</u>
• AASHTO Standard Specifications for Highway Bridges	(AASHTO)
• AASHTO Guide Specification for Bridge Railings, 1989	(AASHTO Guide)
• AASHTO LRFD Bridge Design for Highway Bridges	(AASHTO LRFD)
• FHWA Policy Memorandum, August 28, 1986	(FHWA 1986 Memo)
• FHWA Policy Memorandum, August 13, 1990	(FHWA 1990 Memo)
• FHWA Position Paper, May 14, 1996	(FHWA 1996 Paper)
• NCHRP Report 230, 1981 (re. crash test procedures)	(NCHRP 230)
• NCHRP Report 239, 1981 (re. rail level selection procedures)	(NCHRP 239)
• NCHRP Report 350, 1993 (re. crash test procedures)	(NCHRP 350)

### b) Timeline.

1964 - present	Since 1964 the AASHTO requirement for the design of bridge rail is the application of a 10-kip static transverse load, with related requirements.
1976 - 1983	<p>FHWA crash tested several commonly used bridge rail systems designed by the AASHTO 10-kip load criteria. Several systems failed dramatically. The static load criteria was shown to be insufficient to ensure adequate rail crash test performance. (The original design for the NH curb-mounted 2-bar aluminum rail was tested in 1978 and failed. It was later modified per FHWA requirements.)</p> <p>Two reports were developed concurrently, NCHRP 230 (crash testing procedures) and NCHRP 239 (rail level selection procedures).</p>
1986	FHWA 1986 Memo directed that all bridge rail <u>used on Federal-aid projects shall be crash tested</u> to NCHRP 230 or equivalent. (This was understood to mean two crash tests were required, one with a 1.8 kip compact and one with a 4.5 kip sedan.) A list of 22 crashworthy bridge rail systems was distributed.
1989	<p>AASHTO Guide was published. It differed from NCHRP 230 and 239 by including design criteria recommendations, new performance levels, and a rail level selection procedure. Use of the Guide was, and remains, optional.</p> <p>The AASHTO Guide introduced a major change in that the selection procedures required a PL-2 for interstate and most primary highways, and the PL-2 level in turn required an additional crash test with an 18-kip truck. Such a test required that considerably stronger rail systems be developed.</p>
1990	FHWA 1990 Memo stated that NCHRP 230 crash testing would be considered equivalent to PL-1. A list of 25 additional crash worthy bridge rail systems was

distributed.

- 1993 NCHRP 350 was published and superseded NCHRP 230. NCHRP 350 introduced six test levels which differed from NCHRP 230 and the AASHTO Guide. No rail level selection procedures were included.
- 1994 AASHTO LRFD was published and included recommendations in Section 13 different from the AASHTO Guide but intended to match NCHRP 350. The LRFD includes considerable guidance on bridge rail design issues.
- 1996 FHWA distributed the Equivalency Table to correlate service levels for different crash tests. A third list of 27 additional crashworthy bridge rail systems was distributed.
- 1998 FHWA set an October 1, 1998 deadline after which all bridge rail systems used on Federal aid projects on the National Highway System (NHS) shall be crash tested to a minimum TL-3. (Note that TL-3 is more stringent than PL-1). This memo superseded the FHWA 1986 Memo.
- 2002 FHWA set an October 1, 2002 deadline after which all bridge rail transition systems used on Federal aid projects on the NHS shall be crash tested to a minimum TL-3.
- 2002 Anticipated completion of NCHRP Report 22-12 which will establish new rail level selection procedures and which FHWA will adopt and recommend to AASHTO.

c) Rail Level Selection Procedures

Rail level selection procedures are separate from the crash testing procedures. Rail level selection procedures determine the appropriate rail system needed for a specific site. Levels are based upon such factors as the highway system (NHS, state, etc.), traffic and truck volume (ADT), site conditions (grade, curvature, facilities crossed, etc.), and so forth. These procedures have changed over the years as follows:

- AASHTO assigned one service level.
- AASHTO Guide assigned three performance levels (PL-1 rural, PL-2 interstate and primary, and PL-3 metropolitan).
- NCHRP 350 provided six test levels (TL-1 thru TL-6) and assigned TL-3 as the basic level for highway safety features.
- AASHTO LRFD assigns the same six test levels as NCHRP 350.
- FHWA makes specific requirements for rail selection (see 642.4.1e).
- It is the responsibility of the State to determine which test level is most appropriate for a site.

d) Crash-tested systems

- NETC. NHDOT participated in the New England Transportation Consortium (NETC) program which developed and crash tested two rail systems, the curb-mounted 2-bar and the sidewalk-mounted 4-bar steel bridge rail systems. NETC is currently designing and crash testing bridge rail transitions to match these two bridge rails.
- Other crash-tested systems. FHWA has distributed lists of crash-tested bridge rail systems from around the country acceptable for use on NHS projects. The 1996 lists are on file in the Bridge Design Office. Newer rail systems have since been crash tested and approved, such as the Massachusetts 3-bar steel rail with balusters (MA S3-PL2).
- See website <http://safety.fhwa.dot.gov/roadside/know/barriers/bridge.htm>.

e) FHWA Requirements

- FHWA set an October 1, 1998 deadline after which all bridge rail systems used on Federal-aid projects on the NHS shall be crash tested to a minimum TL-3.
- A higher level may be required based upon the need for a specific site.
- Existing bridge rail systems may remain but must be upgraded to the current service level when rehabilitated or replaced.
- The State is responsible for determining the service level need for a bridge site. The level of service is to be determined by (1) the AASHTO Guide Specification or (2) a rational, experienced-based, cost-beneficial, consistently-applied procedure proposed by the State.
- FHWA set an October 1, 2002 deadline after which all bridge rail transitions used on Federal aid projects on the NHS shall be crash tested to a minimum TL-3.

- FHWA requires use of NCHRP 350 as the testing criteria and has summarized the results of crash testing under previous documents (NCHRP 230, AASHTO Guide, AASHTO LRFD) and considers them meeting the current NCHRP 350 without further testing, as indicated in the Equivalency Table below.

FHWA RAILING LEVEL EQUIVALENCY TABLE						
BRIDGE RAILING TESTING CRITERIA	ACCEPTANCE EQUIVALENCIES					
NCHRP Report 350 (1993)	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
AASHTO LRFD Bridge Specifications (1994)		PL-1		PL-2	PL-3	
AASHTO Guide Specs (1989)		PL-1		PL-2	PL-3	
NCHRP Report 230 (1981)		MSL-1 MSL-2		MSL-3		

Key: TL = Test Level      PL = Performance Level      MSL = Multiple Service Level

#### 642.4.2 NHDOT PRACTICE

With 642.4.1 in mind the NHDOT has established the following practices:

- Use of crash tested bridge rail on all bridges is suggested unless otherwise approved.
- Use the FHWA NHS requirements (TL-3 min.) for all comparable non-NHS highways (e.g. turnpikes, primary routes, and major arteries) in the State.
- Determine the service level need for each site using the AASHTO Guide. Assume TL-4 is required unless a different determination is found.
- The (default) system to use is the T2 or T4 steel bridge rail.
- Use bridge rail anchorage and deck reinforcement that meet the bridge rail standard requirements (since they are an integral part of the bridge rail system). Refer to the Appendix in AASHTO LRFD for additional guidance.
- For added height use the T3 rail as a variation of the T2 bridge rail.
- The T4 rail system may be used in two configurations; with rails (top to bottom) either oriented 4 in-8 in-4 in-4 in (100-200-100-100 mm) with a sidewalk, or 4 in-4 in-8 in-4 in (100-100-200-100) with a curb mount.
- Use of 2-bar and 3-bar aluminum rail should be restricted to TL-2 situations.
- The NHDOT uses the following bridge rail systems:

<b>NHDOT BRIDGE RAIL SYSTEMS</b>	<b>Year Tested</b>	<b>Tested Level</b>	<b>Equiv. Level</b>	<b>COMMENTS</b>
T2 (NETC 2-bar) steel rail on brush curb	1994	PL-2	TL-4	Primary rail system to use.
T4 (NETC 4-bar) steel rail on sidewalk	1997	PL-2	TL-4	Primary rail system to use with sidewalk.
ST 2-bar steel rail on brush curb	--	--	TL-4	Was original NETC design and has heavier tubes than the later crash tested NETC version.
Texas T101 (W-beam with steel tube backing)	1978	--	TL-3	For use on short (less than 20 ± ft (6 ± m.) span bridges. Low height of 27 in is a concern. May use with snow fence for pedestrians. Thrie beam with tube backing may be used to increase height to 32 in.
NH 2-bar aluminum on brush curb	1978	*	TL-2	*Original design failed crash test but features were modified per FHWA and have been accepted for PL-1. Limited to job-by-job use.
NH 3-bar aluminum on sidewalk	--	--	TL-2	For use with speeds below 45 mph.
Vertical concrete parapet	yes	--	TL-4	May add architectural treatment (e.g. Scammell Br.)
New Jersey Concrete Safety Shape	yes	**	TL-4	**Commonly used nationwide.
Glu-lam Timber Rail on Timber Deck (side-mount)	1990	--	TL-3	NHDOT has used variation with T101 rail.
Steel-backed timber railing	1991	--	TL-2	For use on fill-over structures only.

#### 642.4.3 Pedestrian Rail

A bridge with a sidewalk should use 42 in (165 mm) minimum height rail, which may consist of T4 steel rail, or T2 with protective screening, or 3-bar aluminum with balusters, or 2-bar aluminum with protective screening.

On brush curbs opposite a sidewalk use T2 steel rail or 2-bar aluminum rail.

#### 642.4.4 Protective Screening & Snow Fence

##### **For Bridges over Active Railroads**

- A. For all classes of highways with sidewalk, use 9 ft vandal fence.
- B. For all classes of highways with no sidewalks use 5 ft standard snow fence with the exception of Guilford Transportation Rail System, which should be 9 ft straight.

##### **For Bridges over Interstate Highways**

- A. For all classes of highways with sidewalks, use 9 ft high vandal fencing on sidewalk side and 9 ft straight fencing on non-sidewalk side.
- B. For those bridges with no sidewalks, use 5 ft high standard snow fence.

**For Bridges over All Other Roads**

- A. For those bridges with sidewalks, use 9 ft high vandal fencing on sidewalk side and 5 ft high straight fencing on side without sidewalk.
- B. For those bridges with no sidewalks, use 5 ft high standard snow fence.

The above recommendations are made regardless of ADT of Roadways running underneath the bridge or the class of that road. The 5 ft height was arrived at in an effort to remove any dependence on posted speeds or plow speeds. We feel the above recommendations will allow us to have minimum inventory parts for repair or maintenance. Adoption of these recommendations minimizes the risk of claims against the Department relative to vehicle drivers and reduces our risk to our employees.

642.4.5 **Blank**

642.4.6 Geometry

a) Post Spacing

The maximum design spacing of posts for bridge rail systems is as follows:

Material	Rail System	Mounted on	Curb reveal	Offset	Max. Post spacing
Steel	T2	Brush Curb	7 in (175 mm)	5 in (125 mm)*	8 ft-0 in (2.4 m)
"	T4	Sidewalk	7 in (175 mm)	5 ft-0 in (1.5 m)	8 ft-0 in (2.4 m)
"	ST	Brush Curb	7 in (175 mm)	5 in (125 mm)	10 ft-0 in (3.05 m)**
"	Texas T101	Brush Curb	7 in (175 mm)	6 in (150 mm)	8 ft-4 in (2.54 m)***
"	Texas T101	Brush Curb	3 in (75 mm)	zero	8 ft-4 in (2.54 m)***
Aluminum	2-Bar	Brush Curb	7 in (175 mm)	6 in (150 mm)	6 ft-9 in (2.0 m)
"	2-Bar	Sidewalk	7 in (175 mm)	5 ft-0 in (1.5m)	8 ft-0 in (2.4 m)
"	3-Bar	Brush Curb	7 in (175 mm)	6 in (150 mm)	7 ft-5 in (2.25 m)
"	3-Bar	Sidewalk	7 in (175 mm)	5 ft-0 in (1.5 m)	8 ft-0 in (2.4 m)
Timber	Glulam	Timber deck	12 in (305 mm)	zero	6 ft-3 in (1.9 m)
Timber	Steel-backed	In-ground	7 in (175 mm)	zero	10 ft-0 in (3.05 m)

\* Offset used in crash testing was 6 in (150 mm) NH Standard details use 5 in (125 mm).

\*\* The maximum spacing at the end of the bridge is 7 ft-6 in (2.3 m).

\*\*\*A spacing of 8 ft-4 in (2.54 m) or 6 ft-3 in (1.905 m) should be used to match the 25-ft standard lengths of W-beam guardrail.

b) Bend Radius. The following table lists rules of thumb for achievable bend radii for rail elements based upon fabrication criteria. Actual dimensions may vary depending upon a fabricator's equipment and methodology of achieving bends.

<b>Material</b>	<b>Rail System</b>	<b>Min. achievable radius</b>	<b>Comments</b>
Steel	T2, T4	5 ft-0 in (1.5 m)	shop bend for all radii
"	ST	5 ft-0 in (1.5 m)	shop bend for all radii
Aluminum	2-Bar, 3-Bar	30 ft (9.0 m)	shop bend req'd for R < 200 ft
Aluminum	2-Bar, 3-Bar	2.5 ft (750 mm)	hot bend req'd for R < 30 ft.
Steel	Texas T101	5 ft-0 in (1.5 m)	shop bend for all radii

## c) Grades

Bridge rail posts should be constructed normal (perpendicular) to the profile up to and including 5% grades. For grades over 5%, the posts should be plumb. In that case steel posts are cut to the grade and welded to the base plate. For aluminum rail posts a beveled shim is placed under the extruded base plate. The aluminum shim should be a maximum 3/4 in (19 mm) thick at the centerline of the post, which will add to the post height somewhat, and a minimum 3/16 in (5 mm) at the edge.

For protective screening and snow fence, set the screen posts to match the rail posts.

### 642.4.7 Bikeway Rail

The bicycle railing guideline is stated in a memo dated November 10, 1997 from Project Director Robert Greer and approved by Assistant Commissioner Carol Murray:

"On new bridges or on rehabilitated bridges where the rail is being replaced and these bridges are located on a designated bikeway (with signing and/or pavement markings), the height of railing used to protect a bicyclist shall be 54 in measured from the top of the surface on which the bicycle rides to the top of the rail. Smooth rub rails should be attached at handlebar height of 42 in. If a sidewalk separates the bicycle facility from the rail, typical bridge rail shall be used. If bicycles are specifically directed by signage to use the sidewalk, then the 54 in rail shall be used.

On roadways (except for transitions to rail noted herein on bridges) it is not economically reasonable to provide railing heights as noted for bridges. Therefore, unless special conditions exist, typical highway guardrail shall be used on roadways adjacent to designated bikeway shoulders.

On independent bike path facilities where engineering design dictates that a rail is necessary it shall have a height of 54 in measured from the top of the surface on which the bicycle rides to the top of the rail. Smooth rub rails should be attached at handlebar height of 42 inches".

It should be noted that a proposal is before AASHTO to reduce the bike rail height to 43 in (1.1 m).

If special project or site conditions require different treatments than outlined in this section, an exception should be requested from the Chief Engineer.

### 642.4.8 Rail Finishes

Steel bridge rail may be painted over galvanizing to add color for an aesthetic finish. Scratching of the paint (e.g. from snow removal operations) may detract from the finished look. Improperly prepared galvanized surfaces may reduce the adhesion and longevity of the paint. Use of this system should be discouraged.

Aluminum rail may be anodized to add color for an aesthetic finish.

The vertical concrete parapet can be formed with an architectural treatment to improve its aesthetic appearance.

#### 642.4.9 Temporary Concrete Barrier

When temporary concrete (Jersey) barrier is used, the barrier should be attached to the deck if the deck extends transversely less than 4 ft (1.25 m) behind the barrier. The barrier should be attached to the deck two times per 10 ft (3 m) length of barrier with 7/8 in (22 mm) high strength (ASTM A325 or A449) bolts; making sure that the bolts are confined inside the deck reinforcement. In high speed locations, it is recommended that 20 ft (6 m) sections be used on the bridge and attached with four 7/8 in (22 mm) high strength (ASTM A325 or A449) bolts.

When temporary concrete (Jersey) barrier is used and the deck extends transversely more than 4 ft (1.25 m) behind the barrier, the barrier should have approved TL-3 crash tested connections between units.

Temporary concrete barrier should not be placed on new bridge wearing course pavement.

#### 642.4.10 Loads Transmitted to Substructure

For calculating rail crash loads distributed to the substructure (e.g. retaining walls) consult AASHTO LRFD Table A13.2-1. This load is 54 kips (240 kN) for TL-4 distributed over a distance of 3.5 ft (1.07 m).

#### 642.4.11 Curb

Use a 3 in (75 mm) reveal concrete curb with Texas T101 bridge rail, unless approved otherwise by the Design Chief. The concrete curb needs to be protected at the four corners of the bridge if no approach granite curb is used. Concrete curb should not be notched for any membrane application.

## **650 Concrete Decks**

### **650.1 General**

### **650.2 Typical Sections**

- 650.2.1 Cast-In-Place Deck
- 650.2.2 Stay-In-Place Forms

### **650.3 Materials**

- 650.3.1 Concrete
- 650.3.2 Reinforcing Steel

### **650.4 Guidelines for Design**

- 650.4.1 Minimum Deck Thickness
- 650.4.2 Minimum Haunch Thickness for Steel Girders
- 650.4.3 Deck Pouring Sequence
- 650.4.4 Deck Overhang
- 650.4.5 Additional Reinforcing at Rail Post
- 650.4.6 Utilities
- 650.4.7 Reinforcing Bar Layout
- 650.4.8 Main Reinforcing Steel
- 650.4.9 Development and Embedment Lengths
- 650.4.10 Distribution Steel
- 650.4.11 Reinforcing Parameters

### **650.1 General**

Bare decks for bridges should be considered for simple spans without a phased construction joint and with lower traffic volumes; or with stopping conditions or turning movements, which could cause shoving of the pavement wearing surface. Bare decks should have sawn grooves provided per the Special Provisions. Design Chief approval is required for the use of bare decks.

### **650.2 Typical Sections**

#### 650.2.1 Cast-In Place Deck

Minimum deck thickness should be 8 in (200 mm). See Plates 650.4d (650.4e) for deck designs.

#### 650.2.2 Stay-In-Place Forms

##### 650.2.2.1 Precast Deck Panel

Minimum precast deck panel thickness should be 3 1/2 in (90 mm). See 620.5 for more information.

##### 650.2.2.2 Metal Stay-In-Place Forms

Metal stay-in-place forms for concrete bridge decks are not allowed except in special cases as approved by the Design Chief. Rehabilitation projects with a design life of 20 years or less may be a good candidate to permit the use of stay-in-place forms. If stay-in-place forms are permitted for a particular project, it should be so specified in the contract.

## 650.3 Materials

### 650.3.1 Concrete

Concrete decks should be designed for a concrete compressive strength of  $f'c$  4,000 psi (30 MPa). In special cases, the Design Chief may direct the use of other design strengths. Bridge deck concrete should normally be QC/QA.

### 650.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (M 31M Grade 420) epoxy coated.

## 650.4 Guidelines for Design

### 650.4.1 Minimum Deck Thickness

The minimum concrete deck thickness should be determined by using the criteria of clear cover, bar diameters with deformations, and clear space between layers (AASHTO 8.21) with a 0.125 in (5 mm) construction tolerance. For example, 8 in. is the minimum deck thickness using #6 primary bars (0.812 in) and #5 distribution bars (0.687 in), and with clearances of 2.5 in top, 1.25 in bottom and 1 in between layers plus 0.125 in construction tolerance. (For example, 200 mm is the minimum deck thickness using #19 primary bars (20 mm) and #16 distribution bars (18 mm), and with clearances of 65 mm top, 30 mm bottom and 25 mm between layers plus 5 mm construction tolerance)

### 650.4.2 Minimum Haunch Thickness for Steel Girders

Steel girder bridges with concrete decks should be provided with a haunch over each girder monolithic with the deck. The depth of haunch should include camber tolerance (see Plate 630.4e), the cross slope across the top flange, and top flange thickness variations including splice plate thickness, and a minimum haunch thickness of 1 in (25 mm). The haunch thickness is measured from top of flange to the bottom of the deck.

The haunch dimension for designing a composite girder should be assumed to be 0 in (0 mm).

### 650.4.3 Deck Pouring Sequence

A deck pouring sequence should be noted on the plans specifying the deck to be placed in one continuous pour with the concrete remaining plastic until the entire pour is complete. On multiple span bridges the deck should be poured continuously from one end if zero or less than 1,000 lbs (500 kg) uplift occurs at the bearing reaction. Pouring sequences on continuous spans should be developed if uplift occurs. The second pour should be made more than 72 hours after the first pour.

The deck pouring sequence should try to eliminate all, or as many as possible, of the transverse joints. Where decks require multiple pours, pour positive moment areas first with construction joints at dead load inflection points.

The deck pouring sequence should proceed up-grade.

#### 650.4.4 Deck Overhang

The maximum deck overhang (measured from center line of fascia beam to face of coping) for steel girder bridges should be 4 ft (1.25 m) unless approved otherwise by the Design Chief. The typical deck overhang is less than 3 ft (1 m).

The maximum deck overhang (measured from center line of fascia beam to face of coping) for concrete girder bridges should be 1.5 m (5 ft) unless approved otherwise by the Design Chief (this is due to the wider girder flange in concrete girders.). The typical deck overhang is less than 4 ft (1.25 m).

#### 650.4.5 Additional Reinforcing at Rail Post

Additional reinforcing should be required at each bridge rail post base location. Review the appropriate bridge rail Standard Detail sheet for more information.

#### 650.4.6 Utilities

Utility supports should not be attached to the deck since this complicates deck rehabilitation.

#### 650.4.7 Reinforcing Bar Layout

Reinforcing bars may be placed on a skew for bridges with skews less than 25 degrees (Refer to AASHTO LRFD 9.7.1.3). Bars should not be placed on skews if mechanical connectors are to be used.

#### 650.4.8 Main Reinforcing Steel

Main reinforcing steel shall be determined according to AASHTO. Use straight deck bars only. The spacing of the main steel should not be greater than the deck thickness. See Plate 650.4d (650.4e) for more information.

#### 650.4.9 Development and Embedment Lengths

See Plates 601.1b and 601.1c (601.1g and 601.1h) for development and embedment lengths.

#### 650.4.10 Distribution Steel

For concrete decks on simple span steel bridges the maximum spacing for top longitudinal distribution steel should be 12 in (300 mm) (see Plate 650.4a).

For concrete decks on continuous bridges the amount of top and bottom longitudinal steel in the negative moment region should be according to AASHTO (see Plate 650.4a).

The length of the additional top longitudinal reinforcing steel in noncomposite or totally composite bridges, should be  $\frac{3}{4}$  of the length of the negative moment region. These bars should be placed in the top of the deck, sidewalk and curb and should be staggered with their third points over the pier (see Plate 650.4b).

The length of the additional top longitudinal reinforcing steel, composite in the positive moment area only, should be according to AASHTO (see Plate 650.4c). All longitudinal reinforcing steel should be the same size (for constructibility) in order to maintain proper clear cover.

#### 650.4.11 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

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## 651 Approach Slabs

<b>651.1</b>	<b>General</b>
<b>651.2</b>	<b>Typical Sections</b>
<b>651.3</b>	<b>Materials</b>
651.3.1	Concrete
651.3.2	Reinforcing Steel
<b>651.4</b>	<b>Guidelines for Design</b>
651.4.1	Length
651.4.2	Phase Construction
651.4.3	Main Reinforcing Steel
651.4.4	Reinforcing Parameters

### 651.1 General

Approach slabs should be placed at-grade. The at-grade approach slab should eliminate any settlement problems at the backwall. This reduces impact loading at the deck and abutment backwall.

### 651.2 Typical Sections

The approach slab for paved bridges should be placed at-grade minus 2.125 in (53 mm) [2 in (50 mm) of pavement and 0.125 in (3 mm) of barrier membrane].

The approach slab for bare deck bridges should be placed at-grade.

Approach slabs should extend to within 6 in (150 mm) of the curb except for integral bridges.

For integral abutments, the brush curb is cast on top of the approach slab. The approach slab width should match the deck width. Bridge rail should be mounted on top of the brush curb.

### 651.3 Materials

#### 651.3.1 Concrete

Concrete approach slabs should be designed for a compressive concrete strength of  $f'c = 4,000$  psi (30 MPa) (QC/QA Concrete).

#### 651.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31M Grade 420 (M 31 (ASTM A615), Grade 60) epoxy coated.

### 651.4 Guidelines for Design

#### 651.4.1 Length

See 613.4.1 for determining the required approach slab length.

Approach slab designs for 20.0 ft (6 m), 26.0 ft (8 m) and up to our typical maximum length of 30 ft (10 m) are in the following table; Plate 651.4a. Design of the approach slab is based on a span (unsupported length) of  $2/3$  of the length of the slab and an HSm-25 (MSm 22.5) live load.

No additional live load surcharge effects need to be considered for the abutment design when the approach slab length requirements are met.

Maximum length of approach slabs should be 30 ft (10 m) perpendicular to the abutment. Beyond this length the effects of live load surcharge may be neglected.

#### 651.4.2 Phase Construction

Approach slabs at grade, built in phases or for bridge widenings, should be connected to each other.

Buried approach slabs do not have to be connected when there is phase construction or bridge widenings.

#### 651.4.3 Main Reinforcing Steel

See Plate [651.4a](#) for design and reinforcing requirements.

#### 651.4.4 Reinforcing Parameters

See Plate [601.1a](#) for reinforcing parameters.

## **652 Concrete Slabs**

### **652.1 General**

### **652.2 Typical Sections**

652.2.1 Cast-In-Place

652.2.2 Precast

### **652.3 Materials**

652.3.1 Concrete

652.3.2 Reinforcing Steel

### **652.4 Guidelines for Design**

652.4.1 Edge Beam Criteria

652.4.2 Rail Load Distribution Criteria

652.4.3 Main Reinforcing Steel

652.4.4 Development and Embedment Lengths

652.4.5 Reinforcing Parameters

### **652.1 General**

A concrete slab is defined as the primary structural element with the main reinforcing running parallel to traffic. This information only applies to precast concrete slabs that are not prestressed. For prestressed precast concrete slabs see 620.

### **652.2 Typical Sections**

#### 652.2.1 Cast-In-Place

Minimum thickness should be 12 in (300 mm).

#### 652.2.2 Precast

Minimum precast concrete slab thickness should be 12 in (300 mm).

### **652.3 Materials**

#### 652.3.1 Concrete

Concrete slabs should be designed for a concrete compressive strength of  $f'c$  4,000 psi (30 MPa). In special cases, the Design Chief may direct the use of higher design strengths.

#### 652.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31M Grade 420 (M 31 (ASTM A615), Grade 60) epoxy coated.

## **652.4 Guidelines for Design**

### 652.4.1 Edge Beam Criteria

Longitudinal edges are to be designed per AASHTO 3.24.8

### 652.4.2 Rail Load Distribution Criteria

Rail loads shall be distributed according to AASHTO, Section 3.

### 652.4.3 Main Reinforcing Steel

Main steel will be determined according to AASHTO. Use straight bars only. The spacing of the main steel should not be greater than the 12 in (250 mm).

### 652.4.4 Development and Embedment Lengths

See Plates [601.1b](#) and [601.1c](#) (601.1g and 601.1h) for development and embedment lengths.

### 652.4.5 Reinforcing Parameters

See Plate 601.1a for reinforcing parameters.

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## **653 Timber Structures**

### **653.1 General**

### **653.2 Typical Sections**

- 653.2.1 Timber Beams
- 653.2.2 Timber Longitudinal Deck Superstructure
- 653.2.3 Timber Decks

### **653.3 Materials**

- 653.3.1 Timber
- 653.3.2 Timber Pressure Treatment
- 653.3.3 Membrane and Pavement

### **653.4 Guidelines for Design**

- 653.4.1 Allowable Design Values
- 653.4.2 Camber
- 653.4.3 Maximum Deflection
- 653.4.4 Distribution Beam
- 653.4.5 Diaphragms
- 653.4.6 Bearing Locations

### **653.1 General**

Timber structures may be considered in areas with relatively low volumes of traffic, where lighter weight material may be beneficial, and where aesthetic concerns are raised. A cost analysis should be performed to identify the feasibility of these structures.

### **653.2 Typical Sections**

#### 653.2.1 Timber Beams

1. Glued laminated beams should be specified to be manufactured in accordance with ANSI/AITC standards. Stringers can be manufactured for spans ranging up to 90 ft (26.0 m) in length with thickness ranging from 5 in (125 mm) to 16 in (400 mm) and standard depths up to 54 in (1350 mm)
2. American Standard Lumber sizes (referenced in Wood Handbook: Wood as an Engineered Material) should be specified for sawn timber beams. For material larger than 12 in (300 mm) wide and greater than 16 ft (4.8 m) long, availability of the material should be investigated for the stress grade required.

#### 653.2.2 Timber Longitudinal Deck Superstructure

1. A longitudinal deck superstructure is the main structural element of this bridge type. Timber laminations, placed on end with the wide dimension of the laminate situated vertical, are installed longitudinally between supports. Single spans should be limited to a maximum of 32 ft (10.0 m).
2. Nail laminated or dowel laminated longitudinal deck superstructures consist of sawn lumber, typically 2 in (50 mm) to 4 in (100 mm) wide and 8 in (200 mm) to 16 in (400 mm) deep, nailed or doweled together to form the primary load carrying member. Sawn lumber must be full length between supports.

3. Glued laminated longitudinal deck superstructures consist of panels, typically 42 in (1050 mm) to 54 in (1350 mm) wide and 6.75 in (170 mm) to 14.25 in (360 mm) deep, installed side by side to form a continuous surface.
4. Stress laminated longitudinal deck superstructures consist of sawn lumber, typically 2 in (50 mm) to 4 in (100 mm) wide and 8 in (200 mm) to 16 in (400 mm) deep, transversely post-tensioned together to form one continuous load carrying member. Sawn lumber does not need to be full length between supports. However, butt joints do need to be staggered.

#### 653.2.3 Timber Decks

1. Sawn lumber plank decks consist of planks placed flat and attached to the supporting beams. Lumber planks normally are 3 in (75 mm) to 6 in (150 mm) thick and 10 in (250 mm) to 12 in (300 mm) wide.
2. Nail laminated timber decks consist of the laminations placed edgewise (wide face vertical) and nailed or spiked together to form a continuous surface. Timber laminates are normally 2 in (50 mm) wide and 4 in (100 mm) to 8 in (200 mm) deep.
3. Glued laminated timber decks consist of panels that are normally 3.125 in (80 mm) to 6.75 in (170 mm.) thick and 3 ft (900 mm) to 5 ft (1500 mm) wide. Panels are installed side by side (non-interconnected) or doweled together with steel dowels.

### 653.3 Materials

#### 653.3.1 Timber

The manufacture, grading, and quality control of sawn lumber and structural glued laminated timber should be in accordance with AASHTO and the governing grading agency.

Moisture content should be limited to a maximum of 19% for structural sawn timber. A moisture content of  $12\% \pm 2\%$  should be specified for glued laminated timber to insure proper gluing applications.

#### 653.3.2 Timber Pressure Treatment

Treatment for the timber and lumber should be in accordance with the American Wood Preserver's Association (AWPA) standards.

All timber to be treated should be specified to be accurately and completely fabricated prior to pressure treatment, except as otherwise noted.

Oil-borne preservatives, such as creosote or pentachlorophenol, should be specified for all structural elements where dimensional stability of the members is necessary (i.e. beams, decking, piers, etc.).

Water-borne preservatives, such as chromate copper arsenate (CCA) should be specified for members frequently exposed to human contact (e.g. rail elements).

### 653.3.3 Membrane and Pavement

A liquid membrane, which provides adequate bond between the pavement and timber decking, should be installed when pavement is specified.

## 653.4 Guidelines for Design

### 653.4.1 Allowable Design Values

Allowable design values should be per AASHTO. For species not listed in AASHTO, values should be per the National Design Specification for Wood Construction (NDS).

### 653.4.2 Camber

Camber for the full dead load deflection and one half of the live load deflection should be specified for glue laminated beams, glue laminated longitudinal deck superstructures, and stress laminated longitudinal deck superstructures.

### 653.4.3 Maximum Deflection

Maximum live load deflection for the main load carrying members should be limited to a ratio of  $L/500$ .

### 653.4.4 Distribution Beam

A transverse distribution beam situated at mid-span of the structure should be detailed when longitudinal deck superstructure systems are specified.

### 653.4.5 Diaphragms

For timber beams with deck superstructures, two tie rods, one positioned near the top and the other near the bottom of the diaphragm, should be used in conjunction with timber diaphragms to aid in load distribution and reducing fatigue on the bolted connections.

For spans less than 20 ft (6.0 m), diaphragms near the bearing locations are required. For spans between 20 ft (6.0 m) and 40 ft (12.0 m), diaphragms near the bearing locations and at mid-span are required. For spans between 40 ft (12.0 m) and 90 ft (26.0 m), diaphragms near the bearing locations and at the third points are required.

### 653.4.6 Bearing Locations

Beam bearing locations should be detailed to sit flush on the bearing seat with beam ends sawn vertical after full dead load deflection.

## **654 Waterproofing Membrane**

- 654.1 General**
- 654.2 Typical Sections**
- 654.3 Materials**
- 654.4 Guidelines for Design**
  - 654.4.1 Concrete Decks
  - 654.4.2 Timber Decks
  - 654.4.3 Approach Slabs
  - 654.4.4 Structures under Fill

### **654.1 General**

See 650.1 for guidelines on when to use bare concrete decks.

### **654.2 Typical Sections**

The design thickness is 1/8 in (3 mm). For design purposes the unit weight of the membrane is assumed to be the same as pavement at 150 pcf per cubic meter (2400 kg).

### **654.3 Materials**

Types of membrane materials are Liquid Spray, Welded By Torch, Sheet Membrane, and Liquid.

### **654.4 Guidelines for Design**

#### **654.4.1 Concrete Decks**

These are intended as guidelines and decisions should take into account such things as traffic volume, speed, and design life of structure (e.g. if bridge is going to be replaced in 20 years, do not use Item 538.3). It is recommended that sheet membrane not be used on concrete decks.

Item 538.3, Barrier Membrane, Liquid Spray (F) for bridges on the Interstates and divided highways.

Item 538.5, Barrier Membrane, Welded by Torch (F) for all other bridges.

#### **654.4.2 Timber Decks**

Timber decks should use Item 538.11, Barrier Membrane, Liquid.

#### **654.4.3 Approach Slabs**

Approach slabs should use the same barrier membrane as the concrete deck. This eliminates the need to mobilize another manufacturer.

#### **654.4.4 Structures under Fill**

Structures under fill should have sheet membrane (Item 538.1) installed, including a sand cushion or other means of preventing perforation of the membrane during installation of the base courses. Structures under more than 5 ft (1.5 m) of fill do not need a membrane.

## **660 Rigid Frames and Box Culverts**

### **660.1 General**

### **660.2 Typical Sections**

- 660.2.1 Rigid Frames
- 660.2.2 Box Culverts
- 660.2.3 Precast Wingwall

### **660.3 Materials**

- 660.3.1 Concrete
- 660.3.2 Reinforcing Steel
- 660.3.3 Backfill

### **660.4 Guidelines for Design**

- 660.4.1 Equivalent Fluid Pressure
- 660.4.2 Box Culverts
- 660.4.3 Rigid Frames
- 660.4.4 Knee Reinforcing
- 660.4.5 Minimum Slope for Drainage
- 660.4.6 Reinforcing Parameters

### **660.1 General**

Concrete rigid frames and box culverts are generally used as single span precast or cast-in-place structures (See Plate 450.1a). These structures are most favorably used when they can be placed under fill.

### **660.2 Typical Sections**

#### 660.2.1 Rigid Frames

Cast-In-Place: Prismatic, or non-prismatic (e.g. curved intrados with flat top of slab).  
Precast: Prismatic, non-prismatic (e.g. Con-Span)

#### 660.2.2 Box Culverts

Cast-in-place and precast box culverts should have prismatic sections.

Multiple Cells may be used where there is little concern about beaver dams or debris getting caught at the inlet of the structure.

A 3 in (75 mm.) bevel on the underside of the inlet roof should be provided for box culverts and prismatic frames where high water or debris may contact this surface.

#### 660.2.3 Precast Wingwall

[Blank]

## 660.3 Materials

### 660.3.1 Concrete

Cast-in-place structures under fill should be designed for a concrete compressive strength of  $f'c = 3,000$  psi (25 MPa), Concrete Class A. In special cases, the Design Chief may direct the use of higher design strengths. Cast-in-place structures at grade should be designed for a concrete compressive strength of  $f'c = 4,000$  psi (30 MPa), Concrete Class AA.

Precast structures should be designed for a concrete compressive strength of  $f'c = 5,000$  psi (35 MPa). In special cases, the Design Chief may direct the use of higher design strengths.

### 660.3.2 Reinforcing Steel

All reinforcing steel should be AASHTO M 31 (ASTM A615), Grade 60 (M 31M Grade 420). Structures at grade should have all reinforcing in the top slab epoxy coated.

### 660.3.3 Backfill

Backfill should be Granular Backfill (Bridge) (F), Item 209.201, compacted to 95% of maximum density. Granular backfill should be specified a minimum of 5 ft (1.5 m) outside of the horizontal limits of the structure.

## 660.4 Guidelines for Design

### 660.4.1 Earth Pressure

Use AASHTO 3.20.1 and 3.20.2, for design of reinforced concrete box culverts and rigid frames.

### 660.4.2 Box Culverts

Box Culverts should not be designed to operate under a head condition at the design flood flows.

### 660.4.3 Rigid Frames

Rigid frames should not be used for skews greater than  $30^\circ$  except as allowed by the Design Chief.

### 660.4.4 Knee Reinforcing

Legs for J-bars should be detailed the same length or such that there is a significant difference in length between the legs so that they wouldn't be reversed during construction placement. Leg length for the top slab should be limited to approximately the size of the bar in feet unless shipping requirements restrict this (see 660.4.7).

### 660.4.5 Minimum Slope for Drainage

The top surfaces of structures under fill should be provided with a minimum longitudinal slope of 1% to facilitate drainage off the structure.

### 660.4.6 Reinforcing Parameters

See [Plate 601.1a](#) for reinforcing parameters.

## **670 Structural Plate Structures**

### **670.1 General**

### **670.2 Typical Sections**

### **670.3 Materials**

670.3.1 Steel

670.3.2 Aluminum

670.3.3 Backfill

### **670.4 Guidelines for Design**

670.4.1 Strength Tables

670.4.2 Load Effects on Buried Structures

670.4.3 Design

670.4.4 Additional Plate Thickness

670.4.5 Minimum Thickness

### **670.1 General**

Structural plate structures (pipe-arches, arches and pipes) are an economical solution for use as pedestrian underpasses and recreational use and may be suitable for other applications.

### **670.2 Typical Sections**

Steel structural plate pipe-arches and pipes should not be used for water crossings due to severe corrosion at and below the water line. Structural plate arches may be used for water crossings if the arch seat on the reinforced concrete abutment wall is 1 ft (0.3 m) above the normal high water (Q 2.33).

Aluminum structural plate pipe-arches and pipes should not be used for water crossings if the water carries heavy abrasive loads.

Structural plate pipe-arches and pipes may be used for pedestrian/recreational underpasses. A common size for this application is 12 ft-2 in x 11 ft-0 in (3.71 m x 3.35 m). This accommodates most trail groomers; however, the size needs to be coordinated with the Department of Resources and Economic Development (DRED).

### **670.3 Materials**

#### **670.3.1 Steel**

Steel should meet AASHTO M 167. Bolts should meet AASHTO M-164 (ASTM A-325) 3/4 in (20 mm).

#### **670.3.2 Aluminum**

Aluminum should meet AASHTO M 219. Bolts should meet AASHTO F 468 or galvanized steel (ASTM A-307) 3/4 in (20 mm).

#### **670.3.3 Backfill**

Backfill should be Granular Backfill (Bridge) (F), Item 209.201, compacted to 95% of maximum density. Granular backfill should be specified a minimum of 5 ft (1.5 m) outside of the horizontal limits of the structure.

## 670.4 Guidelines for Design

Design structural plate pipe-arches and structural plate pipes using the strength selection tables. See Plate 670.4a (670.4b) for more information.

### 670.4.1 Strength Tables

See Plate 670.4a (670.4b) for the strength tables which apply to Structural Plate Steel Pipes and Pipe-Arches of bolted construction. The standard available sizes and gages, produced by the industry with a 6 in x 2 in (150 mm x 50 mm) corrugation for steel and 9 in x 2 1/2 in (225 mm x 63 mm) for aluminum are listed.

### 670.4.2 Load Effects on Buried Structures

Load effects on buried structures have been, and still are, the subject of continuous and extensive engineering research. These tables are based on the available results of these efforts. It is most important to recognize in using the tables, that flexible buried conduits are effective only in soil-structure interaction and therefore the final load carrying ability of the finished structure depends as much on the proper method of installation - preparing the bedding and placing the backfill - as on the manufactured strength of the steel pipe or pipe-arch itself.

Strict adherence to the material and construction specifications is extremely important.

### 670.4.3 Design

According to AASHTO Section 12 the following design criteria should be considered:

1. Seam Strength
2. Handling and Installation Strength
3. Buckling Strength of Conduit Wall
4. Supporting Soil Strength

The plate thickness indicated in Plate 670.4a (670.4b) satisfies all design requirements. Additional material thickness was provided to resist future deterioration. Any higher fill cover, other than listed in the tables may be considered only with the approval of the Design Chief.

### 670.4.4 Additional Plate Thickness

If a structural plate structure must be used in a water crossing location, aluminum should be used and the invert should have the required thickness increased .05 in (1.25 mm).

### 670.4.5 Minimum Thickness

Minimum thickness for steel should be 12 gage (0.109 in (2.77 mm)) for span lengths under 10 ft (3 m) and 10 gage (0.138 in (3.51 mm)) for span lengths over 10 ft (3 m).

Minimum thickness for aluminum should be .100 in (2.54 mm) for span lengths under 10 ft (3 m) and .150 in (3.81 mm) for span lengths over 10 ft (3 m).

## **680 Bridge Rehabilitation or Widening**

### **680.1 Design Load**

The design load of a rehabilitated or widened bridge should be clearly marked on the plans. Inventory and Operating ratings should be calculated and recorded on the Bridge Capacity Summary Sheet (see 915).

Non-covered bridges should be designed for HS25 (MS 22.5) and 125% Military load; a design load of HS20 (MS 18) or a legal load equivalent should be the minimum design load. By state law RSA 234.4, when using State Bridge Aid funds a 15 ton carrying capacity is required for rehabilitation of non-covered bridges.

By state law RSA 234.27 Capacity of a Bridge: No funds shall be expended unless such bridge may be rehabilitated to a carrying capacity of at least 6 tons. (Therefore, covered bridges would be designed for H6).

See [Appendix D](#) for legislation.

### **680.2 Rehabilitation or Widening**

The designer of a rehabilitation or a widening of an existing bridge should consider making provisions on the design drawings or in the contract documents for the following:

1. Retrofit, repair and nondestructive testing of fatigue-prone details (e.g. end-of-cover plate welds). If cracks are found see 960 for more information.
2. See 603 for seismic guidelines concerning rehabilitations and widening.
3. See 930 for scour guidelines concerning rehabilitations and widening.

### **680.3 Remaining Fatigue Life**

Remaining fatigue life should be checked during the design of any rehabilitation using the AASHTO Guide Specification for Fatigue Evaluation of Existing Steel Bridges.

## **690 Temporary Bridges**

### **690.1 General**

When a temporary bridge is required the plans should include the notes shown on Plate 690.1a.

### **690.2 Suppliers**

Possible choices for temporary bridges are:

1. Contractor supplied: Acrow and Mabey bridges.
2. State owned temporary bridges, which include Acrow and Bailey bridges stored by the Bureau of Bridge Maintenance and erected by the Contractor. Contact the Bureau of Bridge Maintenance concerning availability of Acrow and Bailey parts. Project Engineers should contact Acrow to obtain design and component list.
3. Contractor supplied and erected temporary bridges not made by a supplier of prefabricated temporary bridges.

### **690.3 Standard Widths**

Standard temporary bridge widths are 24 ft (7.2 m) wide, 2 - 12 ft (2-3.6 m) lanes; 30 ft (9.2 m) wide, 3.6-1.0 (12 - 3) typical and 36 ft (11.0 m) wide 3.6-1.9 (12-6) typical.

Acrow and Mabey have sections called double wide, extra wide, and three lane which are acceptable alternatives to the standard widths.

State owned Bailey bridges are classified as extra wide with a roadway width of 13 ft-9 in (4.2 m) and a total bridge width of 19 ft-11 in (6.07 m).

### **690.4 Riding Surface Treatment**

For high volume traffic, (ADT > 10,000) the typical riding surface treatment is 2 in (50 mm) of pavement, but a non-skid grit surface treatment on the steel deck units may be used if the temporary bridge is not going to be used in the winter.

For low volume traffic (ADT < 10,000) the typical riding surface treatment is a non-skid grit surface treatment on the steel deck units.

For Bailey bridges the typical riding surface is a timber deck.

## **691 Recreational Bridges**

For recreational underpass bridges see [670](#).

### **691.1 Pedestrian**

Pedestrian bridges should meet the requirements of AASHTO and have the required rail height described in [642](#).

### **691.2 Bikeway**

Bikeway bridges should meet the requirements of the AASHTO Bike Guide and have the required rail height described in 642.

### **691.3 Snowmobile**

Snowmobile bridges should be wide and strong enough to accept the local groomer requirements. Contact DRED for more information concerning groomers.

On snowmobile bridges, an agreement should be made with the snowmobile club responsible for maintenance concerning the snow height across the bridge during operation. This information is required to set the rail height and the snow dead load.

### **691.4 Emergency Access**

Consideration should be given to maintenance and emergency vehicle access across the bridge as directed by the Design Chief.

### **691.5 Bridge Capacity Sheet**

A bridge capacity sheet should be filled out for recreational bridges. See [915](#) for more information.

### **691.6 Design**

For pedestrian bridges, the design load should be per AASHTO sidewalk loading or the largest maintenance vehicle which would use the bridge (typical design load would be H10 (M9)). Consideration should also be given to loading from emergency vehicles and verification that the bridge can carry any anticipated loads at less than the Operating Rating.

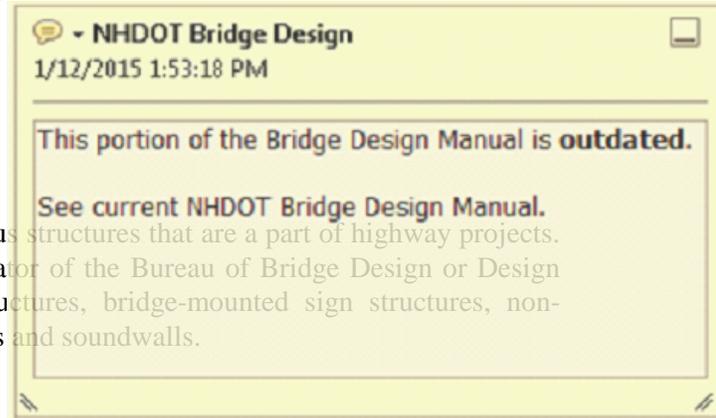
## **692 Railroad Bridges**

### **692.1 General**

See American Railway Engineering and Maintenance of Way Association (AREMA) for the design of railroad bridges.

# SECTION 700 DESIGN OF NON-BRIDGE STRUCTURES

701	<b>General</b>
710	<b>Overhead Sign Structures</b>
710.1	Design Procedure for Sign Footings
720	<a href="#">Bridge-Mounted Signs</a>
730	<a href="#">Traffic Signal Support Structures</a>
740	<a href="#">Retaining Walls</a>
750	<a href="#">Soundwalls</a>



## 701 General

Bridge Design provides design support for miscellaneous structures that are a part of highway projects. Assistance should be requested through the Administrator of the Bureau of Bridge Design or Design Chiefs. These structures include overhead sign structures, bridge-mounted sign structures, non-standard traffic signal support structures, retaining walls and soundwalls.

## 710 Overhead Sign Structures

The Bureau of Bridge Design is responsible for the preliminary and final design of all foundations for overhead sign structures. Consultants may perform these tasks with the approval and guidance of the Bureau. In addition, Bridge Design reviews the design loads and shop drawings for these structures.

### 710.1 Design Procedure for Sign Footings

#### 1. Design Loads

Design wind speed should be 90 mph (145 kph) unless otherwise allowed by the Design Chief. Sign area for design should be increased an additional 30% above the sign areas shown on the plans.

#### 2. Traffic and Highway Design Bureaus' Preliminary Work

- a) Highway Design requests a boring for each footing location.
- b) Highway Design drafts highway cross-section and "stick diagram" at each structure location. The cross-section and stick diagram should show:
  - i) Location of structure including pole base offset(s).
  - ii) Span of structure
  - iii) Size and location of any other attachments to structure.
  - iv) Essential elevations for verifying clearances 17 ft-6 in min (5.34 m).
- c) Cantilever structures should be limited to a 50 ft (15.25 m) maximum overhang. Longer overhangs must be approved by the Design Chief.

3. Highway Design requests preliminary footing size estimate, including estimate of footing quantities from Bridge Design (or Consultant, depending on scope of work in Agreement).
  - a) Bridge Design should be provided copies of the above information (Borings, X-sections, etc.) when this request is made.
  - b) Bridge Design will provide:
    - i) Estimate of approximate footing size.
    - ii) Sketch of proposed footing on X-section. Show top of footing elevation on stick diagram.
    - iii) Estimated quantities for Items 520.21 and 544.
    - iv) Evaluate if cofferdams are required (Item 503.201).
4. Highway Design should transfer information provided by Bridge Design to the Contract Plans (i.e. cross-sections and stick diagrams).
5. After awarding of the Contract, the Fabricator of the structure shall submit shop drawings, a table of loads, and design calculations to Bridge Design (through Construction) for review.
6. Using the design loads from the Fabricator's sign structure design calculations, Bridge Design will design the footing and provide plans and estimate quantities for footing construction. Design procedures shall be based on the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, 1994 with Interims.
7. Bridge Design should review and return shop drawings to the Fabricator (through Construction) for corrections and resubmittal if required. Forward one (1) copy to the Fabrication Engineer for Shop Inspection. The review will be for general conformity with the contract plans and specifications and should include the following: (See also the Sign Footing Design Guidelines Notebook for more detail).
  - a) Check that the plans and calculations are stamped by a NH Licensed Professional Engineer (Plans and calculations will not be reviewed if they are submitted without a PE stamp).
  - b) Check that the appropriate size signs are included on the plans and in the calculations.
  - c) Check the orientation of the signs (correct side: front or back).
  - d) Check the clearances of the signs based on the information provided on the "Overhead Sign Detail Sheets" and cross-sections in the Contract Plans.
  - e) Check the structure span and the correct location of the signs on the shop drawings and in the calculations.
  - f) Check that the correct wind speed is utilized in the calculations and that the overhead sign structure design includes an additional 30% sign surface area as required by the specifications.

- g) Check that walkways are provided when required by the specification [exterior walkways if lighted signs are included, interior walkways if truss structures are greater than 6 ft-0 in (1.83 m) high].
- h) Check that material properties noted on the shop drawing conform to the calculations provided and that the materials conform with NHDOT specification requirements.
- i) Provide a cursory check that connections for components shipped independently to the field match between field sections.
- j) A detailed check of every dimension is not required (it is anticipated that errors in detailing individual components will become apparent during fabrication).
- k) A detailed review of the design is not required (design input will be checked as noted above).

8. Bridge Design will send 4 full size copies of footing plans to Construction.

- a) File original in tubs (File 77-5). Sign structure footings prior to 1999 may be found in File 45-4.
- b) Include half-size copy of footing plans in project folder along with the Footing Design.
- c) File Calculations and Fabrication Shop Drawings and Design Calculations of Structure in project folder.
- d) All sign structure project folders should be filed together (Consultant Section).
- e) List footing sizes in Database. (Filename: S:\MISC\SIGNFTG\CHART)

Sign Structure Manufacturers:

Brookfield Fabricating Corp.	Brookfield, MO	816-258-2214
L.B. Foster Co.	Pittsburgh, PA	717-788-5027
Walpar, Inc.	Birmingham, AL	205-925-4990

- 9. The Project Engineer should make sure that money for shop inspection is included in the estimate. (\$2000 for first structure and \$1500 for each subsequent structure.)

## 720 Bridge-Mounted Signs

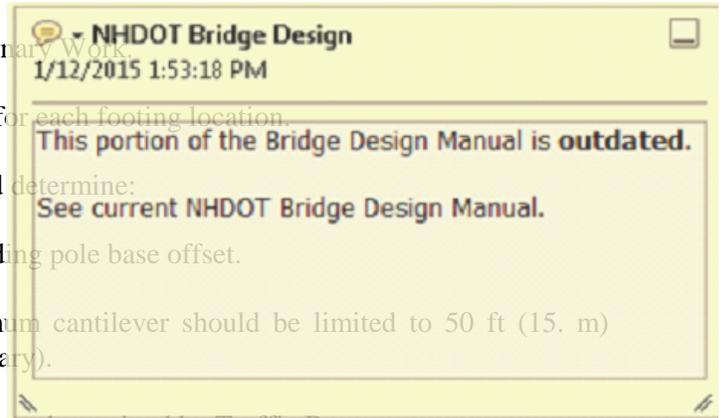
Bridge mounted sign structures should be provided per the standard detail sheets. (The size of the signs should be added to the plan notes.)

## 730 Traffic Signal Support Structures

Bridge Design has developed standard detail sheets for signal foundations for signal mast arm structures up to a 50 ft (15.0 m) maximum overhang.

Bridge Design should provide footing designs for signal structures which do not fall within the parameters shown on the signal footing standard detail sheets.

1. Traffic and Highway Design Bureaus' Preliminary Work.
  - a) Highway Design requests a boring for each footing location.
  - b) Highway Design and Traffic should determine:
    - i) Location of structure including pole base offset.
    - ii) Span of structure. (Maximum cantilever should be limited to 50 ft (15. m) except as absolutely necessary).
    - iii) Size and location of signals as determined by Traffic Bureau.
    - iv) Size and location of any other attachments to structure.
2. Highway Design or Traffic compares the proposed structure to the charts developed for Standard Footings for Traffic Signal Foundations.
  - a) If the proposed structure meets the requirements of one of the standards (Type 1A, B, C or Type 2 24R, 30R, 36R-A, 36R-B) then the appropriate standard footing size is referenced on the plans and the standard detail sheet is included in the Contract Plans.
  - b) If the proposed structure does not fall within the requirements of one of the standards, then Highway Design or Traffic should request a footing design from Bridge Design. The following information should be included with the request:
    - i) Soils report and/or boring logs for each proposed footing location.
    - ii) Information as outlined in 1b above. Minimum and actual highway clearances should also be shown.
    - iii) Specific requirements as to maximum footing size or type of footing.
    - iv) Highway plan view and cross-section(s) at or near the proposed location of the structure.
3. Bridge Design should design the footing and draft plans including quantities, for inclusion in the Contract Plans. Design procedures shall be based on the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, 1994 with Interims.
4. Include half-size copy of footing plans in the Bridge Design project folder along with footing design information and calculations.



## 740 Retaining Walls

See 614 for information concerning Retaining Walls.

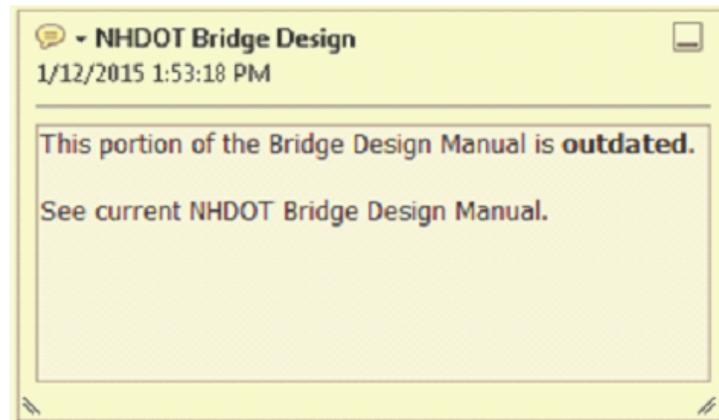
## 750 Soundwalls

Bridge Design assisted in the development of standard detail drawings for timber soundwalls, including their foundations. These details should be useable for all timber soundwalls less than 20 ft (6.1 m) in height with minor plan changes; Bridge Design assistance should be required only in those instances where the standard details cannot be used.

Concrete soundwalls filled with soil and planted to create an aesthetic "green" wall have also been used. A source of water is a necessity if these walls are to be successful in getting plantings to grow. Several systems have been used (i.e. Evergreen and Ecco Walls).

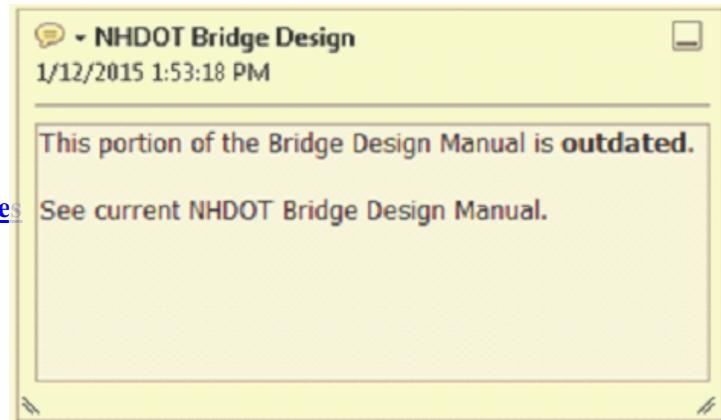
Borings should be requested by Highway Design for these structures.

Per Bureau of Materials & Research, the required vertical requirements is 1% minimum of the cross-sectional area of the footing. For the typical 36 in diameter footing, use 18-#7 (18-#22M) bars regardless of the depth of the footing.



## SECTION 800 - CONSTRUCTION REVIEW

<b>801</b>	<b>General</b>
<b>810</b>	<b><u><a href="#">Falsework Plans</a></u></b>
810.1	General
810.2	Submittals for Documentation
<b>820</b>	<b><u><a href="#">Shop Plans</a></u></b>
820.1	General
820.2	Review
820.3	Copies for Approval
<b>830</b>	<b><u><a href="#">Temporary Retaining Structures</a></u></b>
<b>840</b>	<b><u><a href="#">Welding and Fabrication</a></u></b>
840.1	General



### 801 General

Plans should be reviewed for conformance with the provisions of Section 105. The following guide may be used: "Shop Detail Drawing Review/Approval Guidelines" AASHTO/NSBA Steel Bridge Collaboration, G1.1-9.

### 810 Falsework Plans

#### 810.1 General

Falsework plans are normally required for CIP Box Culverts, Rigid Frames, Superstructures (Concrete Deck) and special cases, such as pier formwork and hammerhead support. A PE stamp shall be required on all plans and calculations.

#### 810.2 Submittals for Documentation

Submittals for Documentation should include, but not be limited to, the following:

1. Stresses in the following components: plywood, stringers, walers, braces, etc., using the AASHTO loadings with appropriate load factors.
2. Dimensions to fit the structure according to plan.
3. Deflections in order to limit distortion of finished concrete surface [1/8 in (3 mm) is maximum allowed]. Deflection criteria is only per dead load, not construction live load.
4. Superstructure cantilever falsework utilizing ready-made brackets, such as Fleming Brackets, shall normally be spaced at 4 ft (1.25 m)  $\pm$ . Holes for falsework brackets in exterior girders should not be permitted where the exterior girders will be seen by the travelling public. If holes are allowed, they shall be filled with HS bolts after usage.
  - a) Bracket holes drilled in the web shall be no closer than 3 in (75 mm), preferably 6 in (150 mm) to a transverse stiffener or connection plate.

- b) Shop plans and falsework plans shall specify that the bracket holes will be filled with ASTM A325 Type 3 HS bolts (for unpainted or painted weathering steel bridges) or ASTM A325 Type 1 HS bolts (for painted bridges) after the brackets are removed.
5. The Contractor should furnish five (5) copies of the plans to the Bureau of Construction. The Bureau of Construction will then transmit two (2) copies to the Bureau of Bridge Design. The Project Engineer should review the submittal for documentation and keep one (1) copy of the plans in the Bridge Design project folder. If there are obvious errors/omissions in the submittal, the Project Engineer should return one (1) copy to the Bureau of Construction to insure coordination.

## **820 Shop Plans**

### **820.1 General**

Shop Plans for review from Fabricators are normally received for pier noses, voided slabs, box beams, rolled beams or girders (steel and concrete), cross frames, lateral bracing, bearings, scuppers, expansion joints, bridge rail and bridge approach rail.

### **820.2 Review**

Shop Plans should be checked for general conformity with the Contract plans (i.e., the structural size of a member should be checked, but not its detailed length, hole layout dimensions, etc. unless required or directed).

The Steel Fabrication Inspection Engineer should review shop plan submittals for conformity with current welding and fabrication code requirements and practice.

The Concrete Fabrication Inspection Engineer should review the concrete mix for concrete members.

### **820.3 Copies for Approval**

For the first submission, the Fabricator shall furnish four (4) copies to the Contractor who shall submit them to the Bureau of Construction, and, upon receipt of a set of plans stamped "Approved" or "Approved as Noted" shall submit the number of copies of the final Shop Plans as required by Section 105.02 of the NHDOT Standard Specifications.

For the final submission, one (1) copy should be checked, stamped "Approved" and noted as the "Office Copy", and placed in the pigeon hole or project folder.

Three copies should be stamped, signed and transmitted with a letter to the Bureau of Construction for their use. One (1) of these copies should be stamped, signed and returned to the Fabricator by the Bureau of Construction.

On structural steel jobs to be shop inspected, one (1) stamped and signed copy should be given to the Steel Fabrication Inspection Engineer at the Bureau of Bridge Design for transmittal to the Shop Inspection Agency.

On concrete jobs to be shop inspected, two (2) stamped and signed copies should be given to the Concrete Inspection Engineer at the Bureau of Materials & Research for transmittal to the Shop Inspection Agency.

Reproducible shop drawings shall be submitted upon approval of structural steel and concrete members as required by the Standard Specifications or Special Provisions.

## **830 Temporary Retaining Structures**

For the review of steel sheet pile cofferdam design, use the reference publication, Steel Sheet Piling Design Manual, US Steel.

For more information concerning cofferdams, see 612.

## **840 Welding and Fabrication**

### **840.1 General**

Shop and field welding shall conform to ANSI/AASHTO/AWS D1.5 Bridge Welding Code, AASHTO Standard Specifications for Highway Bridges, Division II, Section 10 Fabrication of Steel Structures and NHDOT Standard Specifications.

## SECTION 900 - EXISTING BRIDGES

<b>901</b>	<b>General</b>
<b>905</b>	<b><u><a href="#">Bridge Inspection</a></u></b>
<b>906</b>	<b><u><a href="#">Sign Structure Inspection</a></u></b>
<b>910</b>	<b><u><a href="#">Bridge Structural Condition Reports</a></u></b>
<b>915</b>	<b><u><a href="#">Bridge Rating</a></u></b>
<b>920</b>	<b><u><a href="#">Bridge Management System</a></u></b>
<b>925</b>	<b><u><a href="#">Overload Permits</a></u></b>
925.1	Application
925.2	Vehicle Load
925.3	Research the Bridges on the Route
925.4	Compare Vehicle Load to Bridge Capacity
925.5	Determine Permit Requirements
<b>930</b>	<b><u><a href="#">Scour Determinations</a></u></b>
<b>940</b>	<b><u><a href="#">Bridge-Mounted Utilities</a></u></b>
940.1	Requests
940.2	Design Criteria
940.3	Approval Procedures
<b>950</b>	<b><u><a href="#">Bridge Painting Contracts</a></u></b>
<b>960</b>	<b><u><a href="#">Structural Repairs</a></u></b>
960.1	Steel
960.2	Concrete
<b>970</b>	<b><u><a href="#">Historic Bridges</a></u></b>
<b>975</b>	<b><u><a href="#">Covered Bridges</a></u></b>

### 901 General

Section 900 deals with existing "in-service" bridges. Bridge Design keeps bridge plans on file, performs condition inspections, rates bridge capacity, and directs or recommends repair and painting procedures when necessary, in coordination with the Department's Bridge Management System and the Bridge Maintenance Bureau.

### 905 Bridge Inspection

All public highway bridges are inspected at least once every two years by Bridge Design inspectors. State bridges on the "red list" are inspected every six months. Town-owned red-list bridges are inspected once every year. All inventoried pedestrian bridges and railroad bridges are inspected under these same guidelines.

A "Red List" bridge is a bridge requiring interim inspections due to known deficiencies, poor condition, weight restriction or type of construction.

### 906 Sign Structure Inspection

All state-owned sign structures are inspected periodically by Bridge Design inspectors. Structural condition reports are prepared and kept on file for all inspected sign structures.

[Top of the Document](#)

## 910 Bridge Structural Condition Reports

Bridge structural condition reports are prepared and kept on file for all inspected bridges. The bridge component condition ratings conform to the FHWA Recording & Coding Guide for the Structural Inventory & Appraisal of the Nation's Bridges (a.k.a. the National Bridge Inventory).

## 915 Bridge Rating

Ratings should be performed for every bridge project by the Design Engineer in terms of HS loadings without the military load, in accordance with the National Bridge Inspection Standards.

SPECIAL INSTRUCTIONS FOR NHDOT FORM 4 - BRIDGE CAPACITY SUMMARY (see Plate 915.1 and also Appendix E).

1. Exterior stringers should be given separate ratings if their capacity is less than interior stringers.
2. Effective Span Lengths are measured longitudinally, and can be considered as the longitudinal length of bridge loading seen by the member as follows:
  - a) Bridge Decks: Use an effective span length equal to 1 ft-0 in (300 mm).
  - b) Simple Spans: The effective span length is taken as center-to-center of bearings.
  - c) Continuous Spans:  
For midspan capacities, rate the same as simple spans.  
For capacities over pier, use each of the adjacent spans as simple spans.
  - d) Intermediate Floorbeams: Use the sum of the adjacent floorbeam spacing and take the Equivalent Moment Rating for the effective span.
  - e) End Floorbeams: Use the floorbeam spacing, and use the Equivalent Shear Rating for the effective span.
  - f) Trusses: Use the loaded length of span to produce the controlling capacity at each member. (This may result in different effective span lengths for the same member during stress reversals).

## 920 Bridge Management System

Bridge Design provides bridge condition data which is used in prioritizing the repair or replacement of bridges. The entire bridge management system includes regional planning commissions, the Department, the NH Legislature, and Governor & Council in a review and evaluation process that results in what is called the "Ten-Year Transportation Improvement Program". See 310.

## 925 Overload Permits

The overload permit application process is described as follows:

### 925.1 Application

Process application information received from the Permit Supervisor, Bureau of Highway Maintenance.

- Proposed route - written description and/or map
- Proposed vehicle configuration
  - Width (including number of tires and tire spacing)
  - Axle spacings
  - Vehicle height
  - Proposed vehicle loading - gross axle loadings

### 925.2 Vehicle Load

Determine the equivalent H- and HS- loading for the overload vehicle using the PC program "SPECTRUM" or "DEPLOYAD".

### 925.3 Research the Bridges on the Route

- Locate and identify all bridges to be crossed
  - Bridge location map book
  - Route flat file cards
- List the following for each bridge:
  - Location
  - Bridge Number
  - Design Load
  - Dimensions - length, width, spans, etc.
  - Bridge type
- Determine current condition of each bridge
  - Bridge Sufficiency Rating from Inventory Listing
  - Bridge Inspection Reports
- Determine the available capacity of each bridge
  - Bridge Capacity Summary Sheet - giving Inventory and Operating Ratings
  - Correct ratings if required to reflect current conditions
  - A permit overload vehicle should not be allowed over a bridge that does not have a Bridge Capacity Summary sheet on file. If a summary sheet is not on file, one should be prepared, and the original given to the Bridge Inspection Engineer.

## **925.4 Compare Vehicle Load to Bridge Capacity**

For single-trip permits, the available bridge capacity (Operating Rating) may be increased to allow for improved weight distribution (e.g. by centering the vehicle on the bridge) or for reduced impact (e.g. by restricting the vehicle speed).

- For example, if by restricting the vehicle speed to 10 mph and using an impact factor of 10% rather than the normal 25%, the available capacity may be increased by  $1.25/1.10 = 1.14$ , or 14%.

## **925.5 Determine Permit Requirements**

- Maximum vehicle speed while crossing bridges;
- Lateral positioning on bridge while crossing;
- Restriction of other vehicles from bridge.
- A written response including any specific requirements/restrictions should be sent to the Permit Supervisor. The number of hours required to review the request should also be included.

## **930 Scour Determinations**

FHWA requires that all existing bridges over water be evaluated for a 500-year flood event. The NHDOT program to evaluate some existing bridges over water was conducted by a consultant and the results are located in the office of the Chief of Existing Bridge Section.

## **940 Bridge-Mounted Utilities**

This section refers to placement of utilities on existing bridges and the process and criteria for design review.

See NHDOT Utility Accommodation Manual for more information.

### **940.1 Requests**

1. Requests for placing a utility on a structure should come through the Highway Maintenance District Office.
2. If a request comes to Bridge Design directly, send a copy of the transmittal to the District and, if necessary, to the Design Services Engineer.
3. Often it is a good idea to talk with someone at District to see if they know about the utility and have any objections to the proposal. Practical field experience is valuable.

### **940.2 Design Criteria**

1. If the roadway pavement is in good or new condition, investigate whether there is a sidewalk or an unpaved area available as an alternate location for the utility.
2. Do not attach a utility to the exterior of a structure except as a last resort.
3. Do not reduce the clearance over a stream or a roadway.

4. Do not anchor utility brackets into a deck slab. (If there is a redecking project, the utility will not have any support.)
5. Holes in backwalls should be core drilled. Seal around the utility with non-shrink grout.
6. Anchor bolts in concrete should be stainless steel.
7. Do not allow welding to main members of the bridge; use clamping or bolted connections.
8. Do not allow drilling of holes in flanges of girders or stringers; use mechanical means (e.g. clamps).
9. Any holes permitted in the steel should be drilled and never flame cut.
10. Cross frames and diaphragms may be adjusted or altered, if necessary, to support the utility.
11. Brackets should be galvanized after fabrication. Some fittings have rust resistant coatings.
12. Painting of new steel should be performed in accordance with current specifications and be compatible with the existing paint. New steel and damaged areas should be painted.

### **940.3 Approval Procedures**

1. If the bridge is Town or City-owned, the review is at the request of the Owner; the Department does not automatically review the plans for approval. The approval is a statement that the work is in accordance with standards, which would be used if it were a State-owned bridge.
2. "Approval" or "Approved as Noted" is stamped on PRINTS.
3. Projects involving work within the right of way of an Interstate highway must have FHWA approval. This is taken care of by the Department's Design Services Engineer. Utilities DO NOT run parallel with the centerline of an Interstate highway without special permission from FHWA.
4. Do not be too specific with minute details. It is better to be specific on the end result desired: a job that is satisfactory to all concerned and which does not harm the bridge. All work shall be performed in such a manner as not to harm the bridge.
5. Do not get involved with blasting procedures. This is where geotechnical advice should come in. The blasting, if done, should be done "in such manner as not to harm the bridge or the property of others". Any work done which does not accomplish this has not been done according to instructions on our part. That responsibility should rest with the Contractor, not personnel in Bridge Design.
6. Bridge Design is not in the utility business. It should not be our responsibility to dictate to the utility designer what the properties of his materials are. It is their responsibility to know at what spacing they should provide for supports, guides, etc. Similarly, spacing between utilities is not our decision to make in all cases.

7. If we have original drawings, then provide prints at no charge to the designer to locate the proposal on the reproducibles so that as many sets as required can be submitted through the Division. (The number of submittal copies depends on how many offices are involved with the work.) It is to our advantage for the designer to have the most complete information available. If grade is critical, bearing elevations are important to provide.
8. Bridge Maintenance or Construction personnel (sometimes District) are to inspect work under construction: a set of plans should be provided to them. The transmittal letter to the utility company or utility designer should state that they have an obligation to notify Bridge Maintenance at least two weeks prior to the work.
9. Check utility files in Bridge Design for other utilities approved for the area. If one utility has been approved for the area, they have a right to the spot approved for the area, even if work has not been done yet. (It has happened that two people at different times have requested to put utilities in the same location.) Make certain where they are to be located.
10. Not all utilities are covered by "Dig Safe", and not all utilities are located where they were shown on the plans. This has happened even when locations were drawn by the Utility.
11. It is not always desirable to put utility locations on plans as soon as approved. Some literally take years to be built. (Some approvals ten years old have not been put on the bridge.)
12. When an attachment must be in an area usually not approved, this is not to be considered a precedent for future attachments.
13. Sheeting, if needed, should be specified as "Appropriate" which will cover compliance with any and all requirements including those of OSHA. The Utility Company should provide a design as required.
14. Not all utilities with a town name are municipally owned. (e.g. Hampton Water Works is privately owned.)

## **950 Bridge Painting Contracts**

The first step is to determine the condition of the paint on the existing bridge by requesting that the Bureau of Materials & Research perform an assessment of bridge coating (ABC survey). On large bridges an outside consultant does the assessment if funding is available. The second step is to calculate the area of steel to be painted (for trusses use the weight of steel). From this information a repainting estimate and specification can be prepared. The coatings on almost all bridges painted before the mid-1980's contain lead or chromates. As a result, contract specifications must specify stringent EPA pollution control and OSHA worker safety requirements. The NH Department of Environmental Services Air Resources Division must be notified prior to job startup. On any job that involves painting, an EPA ID No. should be obtained from the Department of Environmental Services (DES).

## **960 Structural Repairs**

### **960.1 Steel**

Damage to structural steel may occur due to overheight vehicle damage, construction errors, saw cutting during concrete deck removal, fatigue cracking, or other reasons. Whatever the damage, its nature and location should be carefully documented and photographed. Knowing the bridge materials and characteristics from existing plans, a repair procedure can be developed. The Bureau has prepared several general details and procedures for repairs as well as Section 550 in the NH Standard Specifications. Especially helpful are NCHRP 271 Guidelines for Evaluation and Repair of Damaged Steel Bridge Members, and NCHRP 321 Welded Repair of Cracks in Steel Bridge Members.

### **960.2 Concrete**

The Washington State DOT concrete repair manual is a good guide for prestressed concrete repair.

## **970 Historic Bridges**

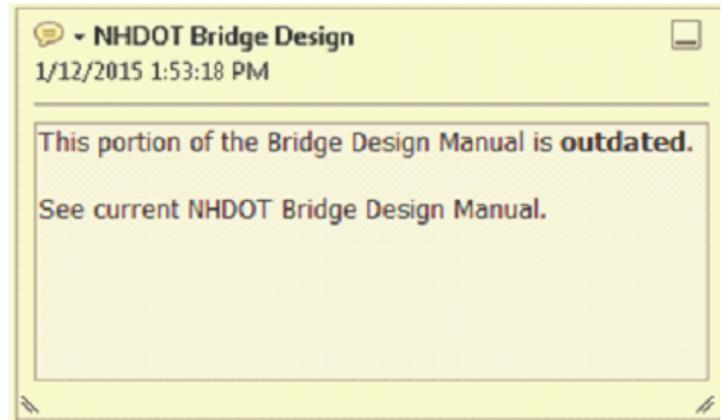
The Historic Bridge Committee assesses bridges over fifty years old for eligibility on the National Register of Historic Places and the NH Historic Bridge Preservation Plan. The NH State Historic Preservation Office (SHPO) works closely with the DOT Bureau of Environment and Bridge Design on historic bridge issues. Federal rules outline the definition and consideration of historic bridges. The eligibility of a bridge for historic preservation can be determined by contacting the Bureau of Environment or from the latest inspection report. The Orford-Fairlee arch is the State's only steel bridge on the National Register of Historic Places. The Cornish-Windsor bridge is the longest covered bridge in the United States.

## **975 Covered Bridges**

The NH Division of Resources and Economic Development is responsible for numbering the covered bridges in NH and prepares literature and maps in the interest of promoting tourism. The State owns a number of covered bridges and the rest are Town or privately owned. The DOT inspects the State and Town-owned covered bridges. Most covered bridges open to traffic have a "red list" status. Maintenance is the responsibility of the owner. Fire prevention methods currently used include heat-detection cables, water sprinklers, and fire-resistant coatings.

# SECTION 1000 - COMPUTER PROGRAMS

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1070.2	Microsoft Office



## 1001 General

Many applications exist to automate the tasks a bridge engineer performs. This section is intended to provide a procedure for the use and development of these applications.

## 1010 Application Development

### Documentation

When an application is developed, it is imperative that the results are authenticated with hand calculations. To assure that the application has been accurately developed and is being properly used by others, an Applications Notebook should be kept which contains an index of existing applications, the program's author, directions for its use, location on the network disk drive, and hand calculations organized in a manner that will allow a user to verify the version of the application which they intend to use. This also allows users of the application to examine the hand calculations in order to learn the method that the application employs.

The index is located in S:\DESIGN\PROGRAMS\INDEX.DOC. The program developers shall list their contributions to the index and then print the index. The new index shall replace the existing index in the Applications Notebook. See Plate 1010.1.

Each application shall be separated from adjacent applications by a cover sheet which is located in S:\DESIGN\PROGRAMS\COVERSHT.DOC. The cover sheet shall include the same information as the index as well as any information necessary to successfully use the program. See Plate 1010.2.

Users should use the program version from the network drive and not create duplicate copies on individual computers. This will ensure that the program includes all updates to the original version.

### Revisions

Users who want to revise an existing application shall, if possible, confirm the need for revision with the author of the application. Revised applications shall include a comment line which includes the date of the revision, the purpose of the revision, and the person responsible for the revision. (It may be necessary to insert this information at the bottom of a file rather than at the top of the file, to maintain page formatting of the application or the integrity of a macro.) Also, supporting calculations shall be included in the Application Notebook to verify the accuracy of the revisions. Alterations to a program which would render previous data sets useless, (or worse, would cause previous data sets to produce incorrect results) should be avoided. Instead, consider the creation of a new program.

If a new program is created, place the old program in the application's \ARCHIVE subdirectory and rename the application to include the date the file was archived (e.g. DeckJul00.xls).

## **1020 Mainframe Design Programs**

### **1020.1 CBEAM**

Continuous Beam Analysis Program for Steel Structures and Concrete Structures - **Program Limitation - Working Stress Design**

### **1020.2 GEOMBR**

Georgia Geometry Program - Bridge geometry emphasizing elevation control. **Program Limitation - Only runs on VAX (not on NODE HAL1).**

### **1020.3 COGO**

Coordinate Geometry Program - **Program Limitation - Only runs on VAX (not on NODE HAL1).**

## **1030 Network Design Programs**

### **1030.1 RETWAL2**

Retaining Wall Design and Abutment Design - Program Limitation - Working Stress Design.

### **1030.2 FRAME**

Concrete Rigid Frame Analysis Program

### **1030.3 SIGN**

Determines loads on an overhead sign footing based on reactions from the sign manufacturer's calculations.

## **1040 PC Design Programs**

### **1040.1 BRASS Girder**

### **1040.2 MERLIN**

Located on Pentium 100 MHz next to Laser Jet 4MV printer on In-House Design section.

### **1040.3 SAP2000**

A structural analysis program which requires Windows 95.

### **1040.4 SPAN**

A simple-span prestressed beam design program produced by LEAP Software, Inc.

## **1040.5 H56A**

Multiple Cell Box Culvert Design & Analysis Program Uses Finite Element Method. Located in: C:\BRDESIGN\BOXCULD.

## **1040.6 WEAP86**

This is typically run by the Soils Section.

Wave Equation Analysis of Piles Determines proper driving parameters for a given pile driving system. Results will be used by Bureau of Construction. Located in: C:\BRDESIGN\PILED

## **1040.7 SIMON**

Girder design program developed by AISC.

## **1040.8 SPECTRUM**

Determines Simple Span Shear & Moment Equivalents for any Special Truck Configuration. See the Bridge Inspection Engineer for this program.

## **1040.9 ARCH and ARCHLF**

Arch Analysis Programs Located in: C:\BRDESIGN\ARCHD

## **1040.10 COM 624P**

Laterally loaded pile head. Source disks on file.

## **1040.11 PONTIS**

Bridge inspections and condition assessment.

## **1040.12 Others**

Others not listed here. Check with current listing of PC programs in the Applications Notebook.

## **1050 Excel Spreadsheets**

### **1050.1 REBAR**

Generates reinforcing summary plan sheets for either English or metric projects. Located in S:\DESIGN\REBAR\ENGSUM2K.XLS (English) or METSUM2K.XLS (metric).

### **1050.2 DECKLFD**

Calculates required deck reinforcement using load factor design. Located in S:\DESIGN\PROGRAMS\DECK\ DELFDENG.XLS (English) or DKLFDMET.XLS (metric).

### **1050.3 AISIBRGS**

Calculates required elastomeric bearing pad size. Located in S:\DESIGN\PROGRAMS\BR\_SHOES\AISIBRGS.XLS.

### **1050.4 STEEL-MET or STEEL-ENG**

Analyzes steel bridge shoes. Located in S:\DESIGN\PROGRAMS\BR\_SHOES

### **1050.5 SPLICE**

Analyzes bolted splices in steel beams. Located in S:\DESIGN\PROGRAMS\STEEL\_BEAM\SPLICE

## **1060 CAD/D Programs**

### **1060.1 GDS**

Graphics Design System

### **1060.2 MOSS**

3-Dimensional Surface Modeling System

### **1060.3 ORACLE**

Relational Data Base Management System

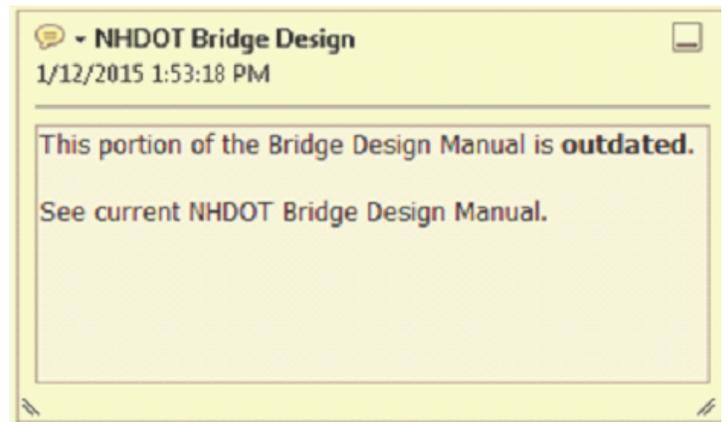
## **1070 General Computer Software**

### **1070.1 Personal Computer**

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## **1101 General**

The administrative procedures of the Bureau of Bridge Design are subject to the Department's *Policies & Procedures*, (a copy of which is on file with the Administrative Secretary) and are generally described herein.

### **1101.1 Office Safety**

See the NHDOT Employee Health and Safety Program booklet (7/99)

The Safety Committee representative from Bridge Design should conduct an annual safety review of the office and field crews per Administrative Procedure 1.39.

### **1101.2 Bridge Design Project Bid Record**

## **1110 Correspondence**

### **1110.1 Transmittals**

A transmittal letter should accompany the transmittal of documents (e.g. plans, drawings, reports, etc.) to other Bureaus, Consultants, agencies, and a blue copy filed with the permanent project records.

### **1110.2 Adjacent States**

Coordination by the Department with officials of adjacent States is required for bridges crossing state lines. (See 1150.3)

### **1110.3 Public**

The public's right-to-know is addressed in DOT Policy #1.28.

## **1120 Electronic Records**

### **1120.1 General**

All project-related documents should be filed by project on the Bureau's S: drive. Working documents should be labeled draft. Confidential documents may be kept on individual J: drives.

### **1120.2 Pontis**

*Pontis* is the name of a Bridge Management Software program, which includes a detailed database of the condition of all the bridges for which the Department is responsible. *Pontis* is instrumental in predicting maintenance and improvement needs and is also used to support the National Bridge Inventory, the FHWA's collection of inventory data from all the States. The Bureau's Existing Bridge Section is responsible for the data collection and maintenance of the *Pontis* program.

### **1120.3 Internet**

Guidelines for using the Internet are established by the Bureau of Information Technology Services.

### **1120.4 Computer Bridge Inventory**

It is the responsibility of the Chief Bridge Inspector to update the Computer Bridge Inventory files when a new bridge is completed.

### **1120.5 Procedures for Updating CAD/D Standard Details**

Proposed changes to standard details should be submitted to the In-House Design Chief showing the proposed change. The change may be circulated for comments, as appropriate, and then the final change(s) made with approval by the Design Chief. Changes should then be circulated as necessary through the office. A record of the change(s) should be kept.

## **1130 Paper Records**

### **1130.1 Documentation**

Telephone calls, verbal decisions, meeting minutes, and information pertinent to projects should be recorded in writing and filed with the permanent project records.

### **1130.2 Correspondence File**

The correspondence file should contain all pertinent correspondence between and among parties in the development of a project, as well as the project estimates. Original documents should be filed if possible. Transmittal copies may be made on blue paper.

### **1130.3 Project Folders**

The project folder should contain all pertinent information relating to the design of a project. The file constitutes the permanent, legal, design record of the project, and as such, should contain all necessary (final) design materials, including but not limited to, the hydraulic report, boring request, boring logs, design calculations, check calculations, quantity calculations, geotechnical report, falsework plans, utility plans, construction submittals, hearing plans and report, and estimates.

### **1130.4 Pigeon Hole Files**

The pigeon hole files constitute storage space for oversize working plans that cannot be kept in the project folder. These plans should be kept for reference until the project is built and finalized by Engineering Audit. Examples of plans would include but not be limited to the original detail plot, original contour plot, preliminary plan, hearing plan, final plans with quantities, and approved shop drawings for structural steel, concrete girders, expansion joints, bridge rail, bridge approach rail, and shop inspection reports.

### **1130.5 Project Card File**

A project card should be started for every bridge project as soon as project authorization is received. (The project card is yellow). The project card should be finalized immediately after bids are awarded. It is the responsibility of the Project Engineer to complete a card for every bridge project. Project bid estimates are now being kept on file by the clerical staff.

### **1130.6 Flat Card File**

When a contract is awarded, a flat file card (by route) and a Bridge Capacity Summary Sheet (Form 4) shall be filled out by the Project Engineer.

### **1130.7 Survey Field Books**

The Highway Design Records Section is responsible for the record-keeping of all survey books. Document survey books received by or leaving Bridge Design with Records Section using a Survey Book Routing Form.

### **1130.8 Archives**

Archive-ready project folders should be kept in the Bureau until Engineering Audit has closed out the project at which time they should be released to Archives. A record of archival documents is kept with the Administrative Secretary.

### **1130.9 Filing Cabinets**

The Bureau has extensive technical publications, catalogues, and reports which are helpful in the development of bridge design and related issues. A brief summary of subjects and locations is listed in Table 1130.9.

### **1130.10 Plan File Tubs**

Contract final/as-built bridge plans are kept in permanent tub files in the Bureau (Preliminary Plans may be discarded after completion of project.). A brief summary of locations is listed in Table 1130.10.

### **1130.11 Library**

The Bureau has extensive technical books and publications related to the development of bridge designs and related issues. A brief summary of subjects and locations is listed in Table 1130.11.

**Table 1130.9 - Bureau Filing Cabinets**

<b>File Cabinet #</b>	<b>Location</b>	<b>Subject</b>
	By Main Door	AASHTO Files
	Basement	Archive-Ready Project Files
	By Main Door	Bureau Administrative Forms
	Under Pigeon Holes	Bureau-Designed Project Folders (In House)
	In SCL's Office	Consultant Contracts & Estimates
	In SCL's Office	Consultant-Designed Project Folders
	By Copier	Correspondence Project Files
	By Main Door	FHWA Files
	On Drafting Table	Flat Card Files
	File alley	In-Service Bridge Inspection Reports - State owned
	File alley	In-Service Bridge Inspection Reports - Town owned
	File alley	Insurance Flood Maps
	File alley	Overhead Sign Foundations
	By Main Door	Paint Catalogues
	By Back Door	Product Catalogues
	By New Pigeon Holes	Project Card Files
	In SCL's Office	Active Municipal Managed Project Files

**Table 1130.10-Plan File Tubs**

<b>Plan File Tub #</b>	<b>Location</b>	<b>Subject</b>	<b>Years (start)</b>	<b>Years (end)</b>	<b>File #s (start)</b>	<b>File #s (end)</b>
0	Storage	Storrs Files	1914	1921	A1	B47
0	Storage	NH Highway Dept	1907	1936	B48	W1
0	Storage	Town Tub Bridges	1985	1997	TT 1-1-1	
1	Storage		1915	1922	1-1-1-1	1-18-5-1
2	Storage		1938	1942	2-1-1-1	2-15-3-20
3	Storage		1950	1971	3-1-1-1	3-15-1-15
3	Storage		1954	1956	D1	E11
4	Storage		1979	1981	4-1-1-1	4-15-4-2
5	Storage		1981	1993	5-1-1-1	5-13-4-4
7	Back Door	1 - 15	1953	1957	A1	C12
7	Back Door	1 - 15	1961	1961	4-1-1	15-4-4
8	Back Door	16 - 30	1963	1978	16-1-1	30-4-4
9	Consultant end	31 - 45	1971	1989	31-1-3	45-3-1
10	Consultant end	46 - 60	1989	1995	46-1-1	60-4-3
11	Consultant end	61 - 71	1995	1996	61-1-1	71-4-1
12	Consultant end	72 - 85 -	1998	--	72-1-1	85-1-1
13		86 - 94			86-1-1	94-1-

**Library - Table 1130.11**

<b>Book Shelf #</b>	<b>Location</b>	<b>Subject</b>
	Dave Powelson	AASHTO Standard Specifications - Past Issues
	Dave Powelson	AISC Construction Manuals - Past Issues
	Dave Powelson	AREMA Specifications
	Dave Powelson	Aluminum Specifications
	Jerry Zoller	Bridge Rail Test Reports
	Aaron Janssen	CADD Manuals
		Civil Engineering Handbooks
	Dave Powelson	CRSI Handbook (Old)
	Gen Bookcase	FHWA Research Reports
	David Scott	High Performance Concrete
	Dave Powelson	Historical Bridge Classifications
	Jerry Zoller	Paint Specifications
	David Scott	PCI Specifications
	David Scott	Seismic Documents
	Jerry Zoller	Steel Repairs
	Jerry Zoller	Welding Specifications
		Timber Bridge Information

**1130.12 Bridge Design Manual**

All changes to the Bridge Design Manual are to be submitted to the In-House Design Chief for review and distribution.

## 1140 Bureau Organization

### 1140.1 Organizational Chart

See Organizational Chart on file with the Administrative Secretary.

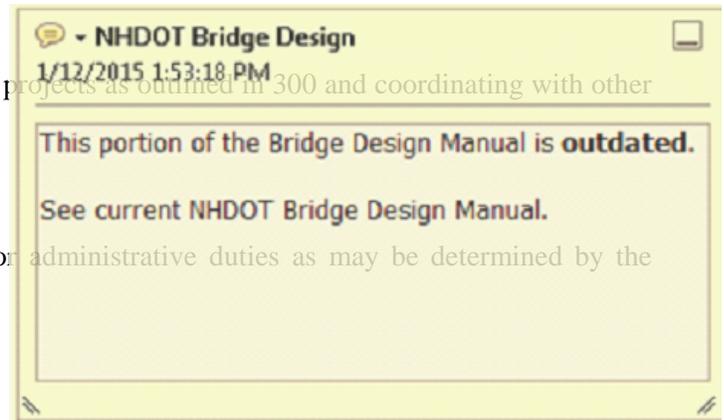
### 1140.2 Design Chief

The Design Chief is responsible for developing projects as outlined in 300 and coordinating with other agencies and Bureaus as necessary.

### 1140.3 Administrative Secretary

The Administrative Secretary is responsible for administrative duties as may be determined by the Administrator, such as the following:

- Oversee and assign work of clerical staff
- Purchase office supplies and equipment
- Process contracts and agreements
- Schedule meetings
- Keep track of employee leave time.



### 1140.4 Trainee / Mentor Program

A typical two- or four-month Engineer trainee orientation program may cover the following subjects and assignments:

- Orientation in Bridge Design
- Deck Design (HS20 & HS25)
- Draft Deck reinforcement (Plan & Section)
- Computer Programs - Deck and Girder
- Girder Design
- Overview of CAD/D
- Boring Layout
- Deck Falsework Check
- Superstructure Design Check
- Field Trip to Fabrication Plant
- Deck Falsework Check
- Bridge Inspection
- Design Abutment & Wings
- Compute Quantities
- Project Related Design Work

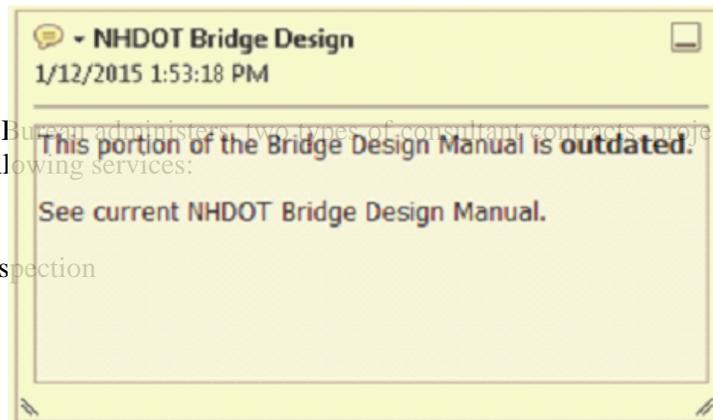
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## 1150 Contract Procedures

### 1150.1 Consultants

The Department normally uses, and the Bureau administers, two types of consultant contracts, project-specific and statewide, to provide the following services:

1. Bridge Design
2. Bridge Painting Consulting and Inspection
3. Structural Steel Inspection
4. Underwater Diving



### 1150.2 Consultant Agreements

To secure the consultant services listed above as (1) Bridge Design and (2) Bridge Painting Consulting and Inspection, the Bureau follows procedures outlined in the Department's *Consultant Selection and Service Agreement Procedures* (the "yellow book" on file with the Chief of Consultant Design).

To secure the consultant services listed above as (3) Structural Steel Inspection, and (4) Underwater Diving, the Bureau follows procedures outlined in the Department's *Consultant Selection and Service Agreement Procedures for Prequalified Low Bid Technical Service Statewide Contracts*, dated February 26, 1998 (on file in the consultant file cabinet).

### 1150.3 Interstate Agreements

The Department shall enter into an Interstate Agreement on projects involving both New Hampshire and a neighboring state, whether Maine, Massachusetts, or Vermont. The Interstate Agreement must obtain Commissioner's Office approval and signature by officials of both states before obtaining approval by the NH Governor and Council. Such an Agreement should be obtained for the design and construction phases (one agreement to cover both phases) of a project before preliminary engineering money can be spent and before the project can be advertised for bids.

The Interstate Agreement establishes the percentage that each state will pay in the cost of the project. For bridges over water between NH and ME, the cost is split 50%-50%. The NH-VT state line runs along the Vermont 1938 shoreline and the percentage for PE is based on its location. The percentage for construction is based on the actual amount of construction in each State. The project engineering is based on the existing bridge location, while the construction engineering is based on the proposed bridge location.

### 1150.4 Administration

Normally, contracts or Interstate Agreements are initiated by a Design Chief but the operational aspects are handled by the Bureau's Administrative Secretary.

## 1160 Consultants

### 1160.1 Evaluation Form

The Bureau evaluates the work of Bridge Design Consulting Engineers with a "Past Performance Report". This report is initiated in the Consultant Design Section and is processed through the Bureau

Administrator to the Commissioner's Office and then on to the Consultant Firm. The report is confidential and the original is kept on file in the consultant master file in Highway Design. No other copies are to be made of this report.

## **1160.2 Distribution**

A list of active bridge design consultants is maintained in the Bureau and used for the distribution of any information or materials affecting their work.

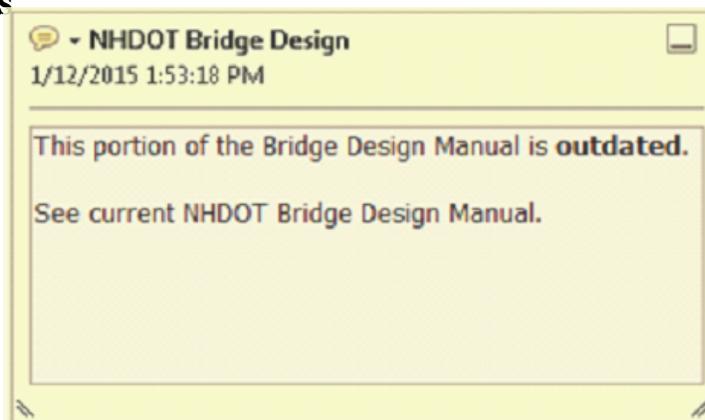
## **1180 Procedures for Public Meetings**

### **1180.1 Notification of Public Officials**

See 480 for information.

### **1180.2 ROW Coordination & Notification**

See 480 for information.



## **1190 Miscellaneous**

### **1190.1 Transportation Enhancement Projects**

Transportation Enhancement Projects are Federally-funded projects aimed at improving non-vehicular aspects of transportation, such as the rehabilitation or construction of sidewalks, bicycle paths, pedestrian trails, etc.

### **1190.2 CMAQ Projects**

CMAQ (Congestion Mitigation and Air Quality) Projects are Federally-funded projects aimed at improving air quality by modifying aspects of vehicular transportation, such as the construction of Park 'n Rides, etc.

### **1190.3 Railroads**

Projects involving railroad companies should have an agreement arranged by the Utilities Section.

### **1190.4 Research Advisory Council**

The Research Advisory Council is made up of Bureau Administrators and approves and selects research topics for the spending of federal State Planning and Research (SPR) funds. The research projects are thereafter administered by the Bureau of Materials & Research.

### **1190.5 Committees**

For informational purposes, see Table 1190.5 for committees served by members of the Bureau.

### **1190.6 List of Experimental Projects / Features / Products**

See Table 1190.6

**Table 1190.5 - Committees**

<b>Member</b>	<b>Agency</b>	<b>Committee</b>
Joe Adams	NH DOT	<ul style="list-style-type: none"> <li>• CAD Selection / Evaluation Committee</li> </ul>
Bob Aubrey	PCI - NE	<ul style="list-style-type: none"> <li>• PCI Technical Committee</li> </ul>
Dean Bennett	NH DOT	<ul style="list-style-type: none"> <li>• Access Users Group</li> <li>• Safety Committee (backup member)</li> </ul>
	NH DOT NH DOT	<ul style="list-style-type: none"> <li>• Bridge Design Manual Committee</li> <li>• Total Quality Management</li> </ul>
Bob Juliano	PCI - NE	<ul style="list-style-type: none"> <li>• PCI Technical Committee</li> </ul>
	NETC	<ul style="list-style-type: none"> <li>• NETC Technical Committee on Project 98-6 "Removal of Concrete Adjacent to Concrete to Remain"</li> </ul>
Mark W. Richardson	NH DOT AASHTO SHRP HITEC	<ul style="list-style-type: none"> <li>• Consultant Selection Committee</li> <li>• Highway Subcommittee on Bridges and Structures</li> <li>• Lead State High Performance Concrete – Chairman</li> <li>• Evaluation of Earth Retaining Systems</li> </ul>
David Powelson	NH DOT NH DOT NETC  AASHTO AASHTO	<ul style="list-style-type: none"> <li>• Access Users Group</li> <li>• USGS Scour Project</li> <li>• NETC Technical Committee on Project 98-2 "Guidelines for Scour Monitoring Systems"</li> <li>• Pontis Technical Advisory Group</li> <li>• Pontis Users Group</li> </ul>
David Scott	NH DOT NH DOT	<ul style="list-style-type: none"> <li>• Bridge Design Manual Committee</li> <li>• CADD Selection / Evaluation Committee</li> </ul>
Peter Stamnas	PCI - NE	<ul style="list-style-type: none"> <li>• PCI Technical Committee</li> </ul>
David Scott	NH DOT NETC  SHRP HPC NH DOT	<ul style="list-style-type: none"> <li>• Bridge Design Manual Committee</li> <li>• NETC Technical Committees 95-5 "Concrete Br. Jts." 98-3 "Evaluation of Asphalt Expansion Joints"</li> <li>• SHRP Products Review Committee</li> <li>• High Performance Concrete Technical Committee</li> <li>• Specification Committee</li> </ul>
Jerry Zoller	NH DOT NH DOT NEPCOAT NETC  NSBA	<ul style="list-style-type: none"> <li>• Bridge Design Manual Committee</li> <li>• Safety Committee</li> <li>• North East Protective Coating Committee (secretary)</li> <li>• NETC Technical Committee on Project 98-1 "Bridge Rail Transitions"</li> <li>• Quality Council</li> </ul>

## SECTION 1200 MUNICIPAL BRIDGE PROGRAM

<b>1201</b>	<b><u>General</u></b>
1201.1	Design Guidelines
<b>1210</b>	<b><u>Municipal Bridge Estimates</u></b>
1210.1	General
1210.2	Preliminary Information Gathering
1210.3	Field Trip/Site Evaluation
1210.4	Project Estimate
<b>1220</b>	<b><u>Municipally Managed Bridge Process</u></b>
1220.1	General
<b>1230</b>	<b><u>Municipally Managed Design Guidelines</u></b>
1230.1	General
1230.2	Engineering Study
1230.3	Preliminary Plans
1230.4	Finalized Plans and Contract Proposal
1230.5	Bridge Load Rating Analysis
1230.6	Review Guidelines

### 1201 General

The Bureau of Bridge Design (through the Bureau of Municipal Highways) assists communities with their bridges in several different ways:

1. Developing Bridge Aid estimates for rehabilitation, removal, or replacement of Town-owned bridges.
2. At the Town's request, coordinating the entire project development process leading to the preparation of plans and contract documents for project advertising and construction.
3. Municipally managed projects - review of projects developed by Town-hired Consultants and advertised by the Town.

#### 1201.1 Design Guidelines

Municipal bridges shall be designed in accordance with AASHTO. Exceptions must be approved in writing by the Town.

The NHDOT Bridge Design Manual should be used as an additional guideline for design.

## **1210 Municipal Bridge Estimates**

### **1210.1 General**

Bridge Design prepares estimates for Town bridges upon receiving an "Application for Preliminary Estimate" for Bridge Aid. This application is signed by the Town and processed through Municipal Highways.

### **1210.2 Preliminary Information Gathering**

In preparation for a site visit, a project folder for each bridge should be started. This folder will contain all the information necessary to prepare an estimate. Folders should be kept in the Town Bridge Advisory Files located in the back of the In-House Bridge Design Section.

Information to be gathered should include copies of a location map, latest inspection report, traffic counts (from Bridge Mini List unless better data is available), flat card (if available), historical status, and hydraulic support data.

If an estimate for the bridge has been previously developed, this part of the process will probably involve simply making a copy of the latest inspection report prior to conducting a site visit.

### **1210.3 Field Trip/Site Evaluation**

A site visit is a necessary and extremely important step in preparing a good estimate. There are several purposes/tasks that should be accomplished during this field trip.

1. Take pictures of the structure and roadway approaches; they are extremely helpful for future reference.
2. Evaluate bridge condition.
3. Evaluate the site with an eye towards roadway deficiencies and safety issues that should be corrected; possible impacts to environmental or cultural resources, right-of-way impacts and hydraulic issues (waterway opening, stream alignment, flow velocities, dams or other structures, flood relief, evidences of past high water marks). Are traffic counts in the ballpark? What are traffic control possibilities? What structure types might be considered? Likely soil types/bedrock?
4. Drive any reasonable detours recording their length and any concerns about width, road conditions, etc.

Outline the most viable concept(s) for rehabilitation or replacement, taking all factors into consideration; note recommendations.

## **1210.4 Project Estimate**

An estimate for the bridge rehabilitation or replacement concept should be prepared. Forms are available in the Advisory File for documenting the costs.

Use the estimating guidelines in 475.

A narrower bridge/roadway may be considered if the Town wishes to reduce impacts on a lightly traveled road. A statement should be provided that, in this case, the Town assumes all liabilities associated with a reduced standard.

Once the preliminary estimate is completed, a letter summarizing the concept and estimated costs should be drafted to Municipal Highways for the Bridge Administrator's signature. The blue copy shall be filed in the project advisory folder.

## **1220 Municipally Managed Bridge Process**

### **1220.1 General**

A municipality may manage the design and construction of a bridge rehabilitation or replacement project and receive Bridge Aid under the applicable provisions of RSA 234. The Municipality shall comply with the requirements outlined below. A guide is provided in the Appendix. The municipality shall apply for a preliminary estimate of the total cost to rehabilitate or replace a bridge and the Department should investigate the site and prepare an estimate as outlined in 1210. The process shall then proceed as follows:

1. After receipt of the Department's Preliminary Estimate, the Town shall notify the Department's Administrator, Bureau of Municipal Highways of their intent to conduct the project themselves and indicate whether they will perform the work with municipal forces or Consultant engagement and contract construction.
2. Should a Consultant be engaged, the Municipality shall provide complete documentation of the selection process and subsequent fee negotiation.
3. Process the design in accordance with "Design Procedures for Bridge Aid - Municipally Managed Project".
4. Upon approval of finalized plans and specifications by the Department, the Municipality may proceed by force account method to advertise the work for bid and send a copy of the advertisement to the Bureau of Municipal Highways.
5. Upon receipt of bids, the municipality shall submit a tabulation of bids received to the Bureau of Municipal Highways for Department approval of award to the low bidder.
6. The Municipality should provide on-site construction inspection by a NH Licensed Professional Engineer.

## **1230 Municipally Managed Design Guidelines**

### **1230.1 General**

These instructions cover administrative procedures applicable to all bridge projects designed by NH Licensed Professional Engineers for municipally managed projects. All studies and plans shall be submitted through the Municipality to the NHDOT Administrator, Bureau of Municipal Highways, for review and approval. Active project files for projects under design should be kept in the Municipal Bridge files in the Consultant Design Chief's office, Bureau of Bridge Design.

### **1230.2 Engineering Study**

The Municipal Engineer or Consultant shall prepare an Engineering Study for the project to include the following:

1. Existing Conditions

This section shall contain a description of the existing bridge and roadway to include bridge width and length; type of bridge superstructure and substructure; and general alignment of the approach roadway including any significant geometric or topographical conditions.

2. Design Criteria

This section shall contain a listing of the relevant design criteria and manuals to be used, including bridge loading and design speed.

3. Proposed Roadway Alignment

This section shall include a description of the methodology and reasoning used to determine the proposed roadway alignment. The discussion shall include horizontal and vertical curves; travel way and shoulder width; and impacts of the proposed roadway alignment, to include wetlands, utilities, other existing structures, and private property.

4. Bridge Type Studies and Recommendations

Bridge types studied shall be indicated in narrative and the recommended bridge type shown in plan, elevation and typical section along with the requisite reasoning therefor. The typical section shall portray the components of the substructure and superstructure, materials of construction, beam spacing, and dimensions of pavement, curbs, etc.

5. Boring Layout and Logs

Borings shall be taken if determined necessary by the Municipal Engineer or Consultant. The number and content of the boring logs shall be sufficient to present a reasonably accurate picture of subsurface conditions.

6. Hydrologic and Hydraulic Studies

The hydrologic and hydraulic parameters at the bridge site shall be determined using accepted engineering methods.

## 7. Cost Estimate

An itemized cost estimate shall be furnished for the proposed bridge type and roadway alignment. Item numbers, names and units shall conform to the Item Description Master File as furnished by the Department.

### **1230.3 Preliminary Plans**

Preliminary plans, which may later be incorporated into the final plans, shall include, but not be limited to:

1. Location plan (small scale, such as 1" per mile) (to be removed from contract drawings).
2. General plan and elevation of the bridge.
3. Cross-section of the approach roadway adjacent to the bridge.
4. Cross-section of the lower roadway or water course through the bridge area.
5. Cross-section of the bridge.
6. General notes including design loading, foundation type, allowable foundation loads, minimum frost cover, superstructure type and seismic design category.
7. Hydrologic and hydraulic data, including drainage area and design flood volume, velocity and elevation.
8. Profiles of all roadways affected by the project.
9. Boring locations and logs if available.
10. Existing ground contours and proposed finished channel contours including proposed channel and slope protection.
11. Roadway plan and critical cross-sections.

#### **1230.4 Finalized Plans and Contract Proposal**

Following review of the Preliminary Plans by the Department, finalized plans and the contract proposal, including specifications, shall be prepared and submitted to the Department for review and approval. This submission shall include an updated quantity list and cost estimate.

Before finalized plans and contract proposal are submitted, they shall be independently checked in detail by a structural and highway designer, other than the original designer, and reviewed by the Consultant's supervising engineer in responsible charge of the project.

The Municipal Engineer's or Consultant's Licensed Professional Engineer stamp for the State of New Hampshire shall appear on the plans and contract proposal to be advertised. The stamp shall be that of the professional engineer who prepared the plans and contract proposal or under whose direct supervisory control they were prepared.

#### **1230.5 Bridge Load Rating Analysis**

The Consultant shall perform a load rating analysis for the bridge using the AASHTO Strength Design Method (Load Factor Design), to be submitted on a Form 4 as provided by the Department. See 915.

#### **1230.6 Review Guidelines**

Consultant submittals should be reviewed for consistency and to make sure there isn't significant departure from Department design/plan guidelines. Estimated time requirements are 2-3 hours for each submittal. Comments from the reviews should be listed in a letter to the Administrator, Bureau of Municipal Highways, indicating acceptance/rejection of the submittal. All materials should be retained in the Active Municipal Managed Project Files.

## Appendix B

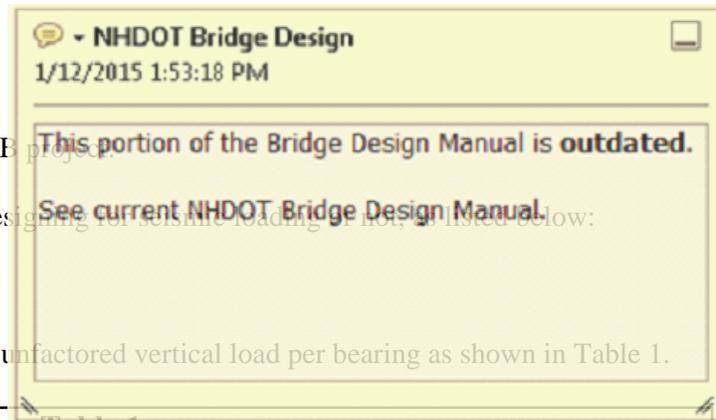
### Seismic Isolation Bearings Worksheet

The values shown are for the Derry 12158-B project.

The first two steps are the same whether designing for seismic loading or not, as listed below:

#### Step 1

Determine the total maximum non-seismic unfactored vertical load per bearing as shown in Table 1.



**Table 1**

<b>Maximum Non-Seismic Vertical Loads Per Bearing</b>			
<i>LOCATION</i>	<i>DEAD LOAD (KIPS)</i>	<i>LIVE LOAD (KIPS)</i>	<i>TOTAL LOAD (KIPS)</i>
ABUT A	19	60	79
PIER 1	57	72	129
PIER 2	57	72	129
ABUT B	19	60	79

#### Step 2

Determine the maximum non-seismic unfactored lateral loads per substructure unit as shown in Table 2. These loads are oriented parallel (longitudinal) and perpendicular (transverse) to the superstructure centerline.

**Table 2**

<b>Maximum Non-Seismic Lateral Loads Per Substructure</b>			
<i>WIND LOAD</i>		<i>WIND ON LL</i>	
LONG.	TRANS.	LONG.	TRANS.
6 KIPS	25 KIPS	5 KIPS	12.5 KIPS

**LONGITUDINAL BRAKING FORCE = 45 KIPS ON COMPLETE SUPERSTRUCTURE**

The third step assumes the use of seismic isolation bearings in determining the maximum superstructure movement due to temperature.

### Step 3

Determine the maximum superstructure movement due to temperature as shown in Table 3. (The superstructure expands about the center of the structure and there are no fixed supports in seismic isolation bearing design.)

<i>LOCATION</i>	<i>D (IN)</i>
ABUT A	0.36
PIER 1	0.16
PIER 2	0.16
ABUT B	0.36

The Project Engineer next has to determine the capacity of the existing or proposed substructure to determine the economics of using seismic isolation bearings.

#### **Rehabilitation Bridge Project**

For a rehabilitation bridge project, this is a straightforward review of the existing substructure. Once the substructure capacities are determined, the appropriate loading cases are used to determine if the existing substructure can handle the required loading.

#### **New Bridge Project**

For a new bridge, the Designer needs to perform two separate designs: one without and one with seismic isolation bearings. For the case with seismic isolation bearings, the Designer may assume a seismic loading of 0.12 x deadload reactions for each bearing along the superstructure. This should give the Designer two designs for comparison (assume the isolation bearings cost \$2500/bearing unless experience proves otherwise).

If the use of isolation bearings appears to provide an economical design, then proceed to step 4.

#### Step 4

Determine the acceleration coefficient A and limit displacements to a typical amount of 2 in. (Watch for bridge elements that will limit the displacements, such as finger joints, pilasters, or cheek walls.). Determine unfactored seismic lateral loads as shown in Table 4.

**Table 4**

**Maximum Forces And Maximum Deflections Per Substructure  
Allowed Due To Earthquake  
(A=0.17g, Soil Type I)**

<i>LOCATION</i>	<i>LONGITUDINAL</i>		<i>TRANSVERSE</i>	
	<i>F (KIPS)</i>	<i>DEFL. (IN)</i>	<i>F (KIPS)</i>	<i>DEFL. (IN)</i>
ABUT A	34.2	2.00	34.2	1.00
PIER 1	46.17	2.00	46.17	1.00
PIER 2	46.17	2.00	46.17	1.00
ABUT B	34.2	2.00	34.2	1.00

**9 Bearings Per Substructure**

These loads are oriented parallel (longitudinal) and perpendicular (transverse) to the superstructure centerline and an R factor of 1.0 is assumed.

#### Step 5

Send Tables 1, 2, 3, 4 and a set of Preliminary Plans to the approved Suppliers for comments to ensure that they can meet the specification requirements.

#### Step 6

All approved Suppliers' requirements should be incorporated before design of substructure begins.

The abutments should still be designed for lateral earth pressure using the Mononobe-Okabe analysis method given in AASHTO Division I-A, Section 6.

# Appendix C

## STATE OF NEW HAMPSHIRE

DEPARTMENT OF TRANSPORTATION  
LOCATION SECTION

### BRIDGE REPORT

Project: \_\_\_\_\_ Project No. \_\_\_\_\_

Bridge No. \_\_\_\_\_ Roadway \_\_\_\_\_ Inventory No. \_\_\_\_\_

Description: \_\_\_\_\_

1. Questions to be asked of Division Engineer or Patrolman:

- a. Extent of trouble caused by ice or drifting \_\_\_\_\_
- b. Does all high water pass through existing structure? \_\_\_\_\_
- c. Do all existing approaches supply relief for high water? \_\_\_\_\_
- d. Additional information by Division Engineer or Patrolman

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

2. Additional information by local residents

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

3. High water information at proposed site:

- a. Elevation of highest water \_\_\_\_\_ Date \_\_\_\_\_
- b. Elevation of average high water \_\_\_\_\_
- c. Location \_\_\_\_\_

4. Additional remarks

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

5. Location of nearest existing structure over same waterway

\_\_\_\_\_

- a. Type of Structure \_\_\_\_\_
- b. Clear Span \_\_\_\_\_ c. Clear Height \_\_\_\_\_
- d. Is waterway adequate? \_\_\_\_\_
- e. Additional remarks \_\_\_\_\_

Submitted by \_\_\_\_\_ Date \_\_\_\_\_

(Copy of Bridge Division; Hydraulics Section; Original for Location Section File)

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**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Duty to Construct and Maintain Bridges***

**SECTION 234:2**

**§ 234:2 Bridge Defined.** – The word bridge when used herein shall mean a structure, having a clear span of 10 feet or more measured along the center line of the roadway at the elevation of the bridge seats, spanning a water course or other opening or obstruction, on a public highway to carry the traffic across, and shall include the substructure, superstructure and approaches thereto.

**Source.** 1921, 155:1. PL 85:1. 1935, 26:2. RL 101:1. 1945, 188:1, part 14:1. RSA 242:1. 1981, 87:1, eff. April 20, 1981.

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-2.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Bridge Inspection***

**SECTION 234:23**

**§ 234:23 Bridges on Class IV and V Highways and Municipally Maintained Bridges on Class II Highways.** – The town or city official in charge of highways, or the selectmen of a town, shall make a biennial inspection of bridges on class IV or V highways and town or city maintained bridges on class II highways. Records of said inspection shall be kept by the town or city. Such town or city officials, or the selectmen of a town, may employ such qualified assistants, engineers or other services as may be necessary to carry out the provisions of this section. Evidence of compliance with this section shall be a prerequisite to application for state bridge aid funds. Inspection reports shall be of a standard form in current use by the department of transportation.

**Source.** RSA 242-A:3. 1969, 222:1. 1973, 554:1. 1981, 87:1. 1985, 402:6, I(a)(5).

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-23.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Bridge Inspection***

**SECTION 234:24**

**§ 234:24 Inspection and Report.** – As a further prerequisite to application for state bridge aid funds an inspection and report shall be made by, or under the supervision of, a registered professional engineer experienced in bridge design and acceptable to the commissioner of transportation.

**Source.** RSA 242-A:3-a. 1973, 554:2. 1981, 87:1. 1985, 402:6, I(b)(5).

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-24.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Bridge Inspection***

**SECTION 234:25**

**§ 234:25 Assistance to Towns and Cities.** – The commissioner of transportation shall, upon request of any town or city, inspect a bridge or bridges in said town or city and supply a copy of the record of said inspection at no expense to the town or city, provided that sufficient qualified personnel are available to make such inspections.

**Source.** RSA 242-A:4. 1969, 222:1. 1979, 482:3. 1981, 87:1. 1985, 402:6, I(b)(5).

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-25.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Rehabilitation of Covered Wooden Bridges***

**SECTION 234:26**

**§ 234:26 Department of Transportation; Authorization.** – The department of transportation is hereby authorized to assist in the rehabilitation of existing wooden covered bridges upon the state secondary and town road systems in the proportions set forth under RSA 234:10 and 11, for the following purposes:

- (a) Replacing of floor beams and reflooring.
- (b) Reroofing.
- (c) Repair or replacement of truss members or wooden arch members.
- (d) Replacement or repair of piers, abutments and wing walls.

**Source.** 1953, 184:1. RSA 243:1. 1981, 87:1. 1985, 402:6, I(a)(5).

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-26.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Rehabilitation of Covered Wooden Bridges***

**SECTION 234:27**

**§ 234:27 Carrying Capacity of Bridge.** – No funds shall be expended unless such bridge may be rehabilitated to a carrying capacity of at least 6 tons.

**Source.** 1953, 184:2. RSA 243:2. 1981, 87:1, eff. April 20, 1981.

<http://sudoc.nhsl.lib.nh.us/rsa/20/234-27.HTM>

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Rehabilitation of Covered Wooden Bridges***

**SECTION 234:28**

**§ 234:28 Limitation on Expenditures.** – I. The total amount that may be expended on any bridge under this subdivision for the above purpose shall in no instance exceed the estimated sum that might be necessary for the construction or reconstruction of a bridge under RSA 234:4, 10 and 11.

II. The commissioner of transportation may waive the requirement for a new covered bridge that replaces a covered bridge which was destroyed within the previous 5 years of application under RSA 234:5.

**Source.** 1953, 184:3. RSA 243:3. 1981, 87:1. 1993, 284:4, eff. July 1, 1993.

**TITLE 20**  
**Transportation**

**CHAPTER 234**  
***Bridges And Bridge Aid***

***Duty to Construct and Maintain Bridges***

**SECTION 234:4**

**§ 234:4 Capacity of Bridge.** – All bridges constructed with bridge aid funds shall have a carrying capacity of at least the legal load as stipulated in RSA 266. All bridges reconstructed with bridge aid funds shall have a carrying capacity of at least 15 tons.

**Source.** 1921, 155:12. PL 85:16. 1929, 131:2. RL 101:16. 1945, 188:1, part 14:3. RSA 242:3. 1981, 87:1. 1985, 78:3. 1994, 317:1, eff. Aug. 7, 1994.

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<http://sudoc.nhsl.lib.nh.us/rsa/20/234-4.HTM>

Date	<b>SECTION 1 Changes</b>
10-1-01	<i>Removed</i> Plates <b>620.5b, 620.5c, 620.5d, 620.5e, and 630.4h</b>
7-1-02	<i>Changed</i> plates <b>650.4d &amp; e</b> Didn't send out – Only to staff
1-2-03	Plates <i>changed</i> : <b>320.3, 320.6, 475.1a, 540.1c, 650.4a, 650.4d, 650.4e, 660.1b, 1160.1,</b> Changes in Item Numbers: <b>610.4.14 Footing Seal</b> <b>613.5.2 Design Guidelines</b> <b>620.4.11 Addition of Defection</b> <b>620.5 Precast Deck Panels <i>whole item</i></b> <b>630.2.5 Shear Connectors</b> <b>630.3.2 #4</b> <b>630.3.2.1 Painting for Weathering Steel</b> <b>642.4.4 Protective Screening &amp; snow fence Addition of</b> <b>642.4.5 Deleted item</b> <b>642.4.6 Bridge rail system</b> <b>642.4.8 Rail Finishes</b> <b>Bridge Capacity Summary ~ Available Capacity</b>