Bridge Manual

New York State Department of Transportation

Office of Structures

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# Table of Contents

Foreword  
Acknowledgments  

## 1 INTRODUCTION  
1.1 Purpose ........................................................................................................................ 1-1  
1.2 Applicability ................................................................................................................... 1-1  
1.3 Policy ............................................................................................................................ 1-2  
1.4 Referenced Standards, Manuals and Documents ........................................................ 1-3  

## 2 GEOMETRIC DESIGN POLICY FOR BRIDGES  
2.1 Purpose ........................................................................................................................ 2-1  
2.2 Geometric Design Policy Glossary ............................................................................... 2-1  
2.3 Clear Roadway Width Standards for Bridges ............................................................... 2-5  
2.3.1 General ........................................................................................................... 2-5  
2.3.2 Railroad Bridges ............................................................................................. 2-5  
2.3.3 Miscellaneous Bridge Width Considerations .................................................. 2-7  
2.4 Vertical Clearances ..................................................................................................... 2-10  
2.4.1 Over Highways, for Highway, Pedestrian, and Overhead Sign Structures ... 2-10  
2.4.2 Railroad Grade Separations ......................................................................... 2-12  
2.4.3 Waterways .................................................................................................... 2-12  
2.4.4 Navigable Waterways ................................................................................... 2-13  
2.4.5 Miscellaneous Vertical Clearance Criteria .................................................... 2-13  
2.5 Horizontal Clearances: Under Bridge Features .......................................................... 2-14  
2.5.1 Highway ........................................................................................................ 2-14  
2.5.2 Navigable Waterways ................................................................................... 2-16  
2.5.2.1 Navigation Lights ......................................................................... 2-19  
2.5.2.2 Additional Navigation Aids ........................................................... 2-19  
2.5.3 Railroads ...................................................................................................... 2-22  
2.5.4 Miscellaneous Corridors ............................................................................... 2-27  
2.6 Live Loading Requirements ........................................................................................ 2-28  
2.6.1 New and Replacement Bridges .................................................................... 2-28  
2.6.2 Bridge Rehabilitation .................................................................................... 2-29  
2.6.3 Temporary Bridges ....................................................................................... 2-29  
2.6.4 Pedestrian Bridges ....................................................................................... 2-30  
2.6.5 Railroad Bridges ........................................................................................... 2-30  
2.7 Alignment, Profiles and Superelevation ...................................................................... 2-30  
2.7.1 Horizontal Alignment .................................................................................. 2-30  
2.7.2 Profile ........................................................................................................... 2-30  
2.7.3 Superelevation .............................................................................................. 2-31  

Appendix 2A Bridge Roadway Width Tables  
Appendix 2B One Lane Bridge Policy  
Appendix 2C Vertical Clearance over the NYS Thruway, I-90 and Revised 16' Clearance Network
3 PLANNING NEW AND REPLACEMENT BRIDGE TYPES

3.1 Scoping ................................................................. 3-1
3.2 Preliminary Engineering ........................................... 3-2
3.3 Site Data .............................................................. 3-3
3.4 Hydraulics .............................................................. 3-3
  3.4.1 Hydraulic Design .............................................. 3-3
  3.4.2 Hydraulic Table ................................................ 3-4
  3.4.3 Slope Protection Criteria .................................... 3-5
  3.4.4 Scour Monitoring Devices ................................. 3-5
  3.4.5 Stream Crossing Permit Requirements ............... 3-8
3.5 Structure Selection Process ...................................... 3-8
  3.5.1 Establishing Span Lengths ................................. 3-8
  3.5.2 Bridge Type Based on Span Lengths .................... 3-9
    3.5.2.1 Span Lengths Less than 40 ft ......................... 3-9
    3.5.2.2 Span Lengths Between 40 ft and 100 ft ............ 3-10
    3.5.2.3 Span Lengths Between 100 and 200 ft ............ 3-11
    3.5.2.4 Span Lengths Between 200 ft and 300 ft .......... 3-11
  3.5.3 Multiple Span Arrangements .............................. 3-11
  3.5.4 Spans over 300 ft ............................................ 3-12
  3.5.5 Selection Guidelines ......................................... 3-13
3.6 Substructures ...................................................... 3-14
  3.6.1 Substructure Location ....................................... 3-14
  3.6.2 Foundation Assessment ..................................... 3-15
  3.6.3 Foundation Selection ........................................ 3-15
    3.6.3.1 Water Crossings ......................................... 3-16
    3.6.3.2 Grade Separations ...................................... 3-15
  3.6.4 Orientation, Configuration, and Details ............... 3-16
    3.6.4.1 Skew ....................................................... 3-16
    3.6.4.2 Water Crossings ......................................... 3-16
    3.6.4.3 General Details .......................................... 3-17
3.7 Work Zone Traffic Control ...................................... 3-18
  3.7.1 General .......................................................... 3-18
  3.7.2 Off-Site Detour ............................................... 3-18
  3.7.3 Stage Construction .......................................... 3-18
  3.7.4 On-Site Temporary Bridges ............................... 3-20
  3.7.5 Alternative Alignments ..................................... 3-21
3.8 Alternate Designs .................................................. 3-21
3.9 Hazardous Materials ............................................. 3-22
3.10 Environmental Initiative ........................................ 3-22
  3.10.1 Introduction ................................................... 3-22
  3.10.2 Types of Project Enhancements ......................... 3-23
  3.10.3 When to Identify Enhancements ......................... 3-24
  3.10.4 Summary ...................................................... 3-25
3.11 Final Preliminary Bridge Plan ................................. 3-25
  3.11.1 General ........................................................ 3-25
  3.11.2 Format .......................................................... 3-25
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.12</td>
<td>Structure Justification Report</td>
<td>3-26</td>
</tr>
<tr>
<td>3.13</td>
<td>Hydraulic Justification Report</td>
<td>3-26</td>
</tr>
<tr>
<td>3.14</td>
<td>Accelerated Bridge Construction</td>
<td>3-27</td>
</tr>
<tr>
<td>3.12</td>
<td>Structure Justification Report</td>
<td>3-26</td>
</tr>
<tr>
<td>3.13</td>
<td>Hydraulic Justification Report</td>
<td>3-26</td>
</tr>
<tr>
<td>3.14</td>
<td>Accelerated Bridge Construction</td>
<td>3-27</td>
</tr>
<tr>
<td>Appendix 3A</td>
<td>Bridge Data Sheet Part 1</td>
<td></td>
</tr>
<tr>
<td>Appendix 3B</td>
<td>Bridge Data Sheet Part 2</td>
<td></td>
</tr>
<tr>
<td>Appendix 3C</td>
<td>Project Monitor Sheet</td>
<td></td>
</tr>
<tr>
<td>Appendix 3D</td>
<td>Preliminary Plan Development Procedure for New and Replacement Bridges</td>
<td></td>
</tr>
<tr>
<td>Appendix 3E</td>
<td>Preliminary Bridge Plan Work Process – Structures Division Design</td>
<td></td>
</tr>
<tr>
<td>Appendix 3F</td>
<td>Structures Preliminary Plan Check List</td>
<td></td>
</tr>
<tr>
<td>Appendix 3G</td>
<td>Preliminary Plan Tear Sheet Notes</td>
<td></td>
</tr>
<tr>
<td>Appendix 3H</td>
<td>Structure Justification Report</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>STRUCTURE EXCAVATION, SHEETING, AND COFFERDAMS</td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>General Guidelines for Structure Excavation Protection and Support</td>
<td>4-1</td>
</tr>
<tr>
<td>4.2</td>
<td>Unclassified Excavation and Disposal</td>
<td>4-2</td>
</tr>
<tr>
<td>4.3</td>
<td>Structure, Trench and Culvert, and Conduit Excavation</td>
<td>4-3</td>
</tr>
<tr>
<td>4.4</td>
<td>Removal of Substructures</td>
<td>4-3</td>
</tr>
<tr>
<td>4.5</td>
<td>Excavation Protection System</td>
<td>4-4</td>
</tr>
<tr>
<td>4.6</td>
<td>Interim Sheeting</td>
<td>4-4</td>
</tr>
<tr>
<td>4.6.1</td>
<td>Interim Steel Sheeting</td>
<td>4-4</td>
</tr>
<tr>
<td>4.6.2</td>
<td>Interim Timber Sheeting</td>
<td>4-5</td>
</tr>
<tr>
<td>4.7</td>
<td>Temporary Sheeting</td>
<td>4-6</td>
</tr>
<tr>
<td>4.7.1</td>
<td>Temporary Steel Sheeting</td>
<td>4-6</td>
</tr>
<tr>
<td>4.7.2</td>
<td>Temporary Timber Sheeting</td>
<td>4-6</td>
</tr>
<tr>
<td>4.8</td>
<td>Permanent Sheeting</td>
<td>4-7</td>
</tr>
<tr>
<td>4.8.1</td>
<td>Permanent Steel Sheeting</td>
<td>4-7</td>
</tr>
<tr>
<td>4.8.2</td>
<td>Permanent Timber Sheeting</td>
<td>4-7</td>
</tr>
<tr>
<td>4.9</td>
<td>Cofferdam and Waterway Diversion Guidelines</td>
<td>4-8</td>
</tr>
<tr>
<td>5</td>
<td>BRIDGE DECKS</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>Concrete Deck Slabs</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1.1</td>
<td>Composite Design</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1.2</td>
<td>Monolithic Decks for Spread Girders</td>
<td>5-2</td>
</tr>
<tr>
<td>5.1.2.1</td>
<td>History</td>
<td>5-2</td>
</tr>
<tr>
<td>5.1.2.2</td>
<td>Current Practice</td>
<td>5-2</td>
</tr>
<tr>
<td>5.1.3</td>
<td>Monolithic Decks for Adjacent Concrete Beams</td>
<td>5-3</td>
</tr>
<tr>
<td>5.1.4</td>
<td>Two Course Decks</td>
<td>5-3</td>
</tr>
<tr>
<td>5.1.5</td>
<td>Deck Reinforcement Design</td>
<td>5-4</td>
</tr>
<tr>
<td>5.1.5.1</td>
<td>Isotropic Decks</td>
<td>5-4</td>
</tr>
<tr>
<td>5.1.5.2</td>
<td>Traditional Deck Slab Reinforcement</td>
<td>5-6</td>
</tr>
<tr>
<td>5.1.5.3</td>
<td>Reinforcement of Decks for Adjacent Concrete Beams</td>
<td>5-8</td>
</tr>
<tr>
<td>5.1.5.4</td>
<td>Deck Overhangs</td>
<td>5-8</td>
</tr>
<tr>
<td>5.1.6</td>
<td>Haunches</td>
<td>5-10</td>
</tr>
<tr>
<td>5.1.7</td>
<td>Forming</td>
<td>5-13</td>
</tr>
<tr>
<td>5.1.8</td>
<td>Continuous Structure Deck Slab Placements</td>
<td>5-13</td>
</tr>
<tr>
<td>5.1.9</td>
<td>Stage Construction Deck Slabs</td>
<td>5-18</td>
</tr>
</tbody>
</table>
5.1.9.1 General Considerations ............................................................... 5-18
5.1.9.2 Steel Superstructures .................................................................. 5-19
5.1.9.3 Stage Construction Deflection Calculations for Steel Structures 5-20
5.1.9.4 Prestressed Concrete Superstructures ........................................... 5-20
5.1.10 Deck Sealers ................................................................................. 5-20
5.1.11 Aggregate Requirements for Concrete Decks and Approach Slabs 5-21
5.2 Jointless Decks at Abutments ............................................................ 5-23
5.2.1 Jointless Decks over Conventional Abutments ................................. 5-23
5.2.2 Jointless Decks at Integral and Semi-Integral Abutments ................. 5-24
5.2.3 Jointless Decks at Piers (Link Slab) .................................................. 5-24
5.3 Other Deck Types .............................................................................. 5-25
5.4 Deck Drainage .................................................................................. 5-26
5.5 Deck Expansion Joints ...................................................................... 5-27
5.5.1 Transverse Expansion Joints ............................................................ 5-27
5.5.1.1 Armorless Joint Systems .............................................................. 5-28
5.5.1.2 Armored Joint Systems ............................................................... 5-28
5.5.1.3 Modular Joint Systems ................................................................. 5-29
5.5.2 Longitudinal Joints ......................................................................... 5-29
5.6 Sidewalk and Brush Curb Overlays .................................................... 5-29

6 BRIDGE RAILING

6.1 Introduction ....................................................................................... 6-1
6.2 Types of Railing ............................................................................... 6-1
6.3 Railing and Barrier Design for New and Replacement Bridges ............ 6-2
6.3.1 Service Levels ............................................................................... 6-2
6.3.2 Railing/Barrier Design Alternatives .............................................. 6-3
6.3.3 Railing/Barrier Selection ............................................................... 6-5
6.3.3.1 Interstate and Controlled Access Highways ............................... 6-5
6.3.3.2 Other Highways ........................................................................ 6-5
6.3.4 Weathering Steel Bridge Railing .................................................... 6-6
6.3.5 Transitions .................................................................................... 6-7
6.3.6 Modifications ............................................................................... 6-7
6.4 Precast Concrete Barrier ................................................................. 6-7
6.5 Pedestrian Fencing .......................................................................... 6-7
6.6 Permanent Snow Fencing ............................................................... 6-8
6.7 Railing/Parapet Design Dead Loads .................................................. 6-9
6.8 Guidelines for Railing Treatments on Rehabilitation Projects ............. 6-9
6.8.1 Background .................................................................................. 6-9
6.8.2 Purpose ......................................................................................... 6-9
6.8.3 Warrants ....................................................................................... 6-10
6.8.4 Identified Work Strategies ............................................................. 6-11
6.8.4.1 Long Term Work Strategy ......................................................... 6-11
6.8.4.2 Short Term Work Strategy ......................................................... 6-11
6.8.4.3 Monolithic Deck Work .............................................................. 6-12
6.8.5 Actions to be Taken ...................................................................... 6-13
6.8.5.1 Replacing the Bridge Railing/Barrier ......................................... 6-13
6.8.5.2 Upgrading the Bridge/Railing Barrier ........................................ 6-13
6.8.5.3 Retaining the Bridge Railing ...................................................... 6-14
6.8.5.4 Anchorage of Steel Bridge Railing ........................................... 6-15
6.8.6 Responsibilities and Authorities .................................................... 6-16
6.9 Bridge Railing/Transition Shop Drawing Requirements .............................................. 6-16

Appendix 6A  1987 Bridge Railing Crash Test Report
Appendix 6B  Railing Treatments on Rehabilitation Projects

7 UTILITIES

7.1 Criteria for Utility Placement on Bridges ................................................................. 7-1
7.2 Design Information Furnished by Utilities ............................................................... 7-1
7.3 Utility Locations ........................................................................................................... 7-1
7.4 Design Criteria for Utilities and Supports ............................................................... 7-2
7.5 Utility Shares ............................................................................................................. 7-3

8 STRUCTURAL STEEL

8.1 Design ........................................................................................................................ 8-1
8.1.1 Design Methods ................................................................................................. 8-1
8.1.2 Analysis Methods ............................................................................................... 8-2
8.1.3 Design Considerations ....................................................................................... 8-3
8.2 Steel Types ................................................................................................................. 8-3
8.2.1 Unpainted Weathering Steel .............................................................................. 8-3
8.2.2 Drip Bars for Unpainted Weathering Steel ...................................................... 8-4
8.2.3 Painted Steels .................................................................................................... 8-4
8.2.4 HPS Steel .......................................................................................................... 8-5
8.2.5 Other Steels ....................................................................................................... 8-5
8.2.6 Combination of Steel Types .............................................................................. 8-6
8.2.7 Steel Item Numbers ........................................................................................... 8-6
8.3 Redundancy - Fracture Critical Members ................................................................. 8-7
8.3.1 Primary and Secondary Members ...................................................................... 8-7
8.3.2 Redundancy ....................................................................................................... 8-7
8.3.3 Fracture Critical Members ................................................................................ 8-7
8.4 Economical Design .................................................................................................... 8-8
8.4.1 Girder Spacing .................................................................................................... 8-8
8.4.2 Girder Proportioning for Plate Girders ............................................................... 8-9
8.4.2.1 General ........................................................................................................... 8-9
8.4.2.2 Depth ............................................................................................................ 8-9
8.4.2.3 Flanges ......................................................................................................... 8-9
8.4.2.4 Webs ............................................................................................................ 8-10
8.4.2.5 Stability During Erection .............................................................................. 8-11
8.4.3 Rolled Beams ...................................................................................................... 8-11
8.5 Metal Thicknesses ..................................................................................................... 8-13
8.6 Connections .............................................................................................................. 8-13
8.6.1 General ............................................................................................................... 8-13
8.6.2 Bolts ..................................................................................................................... 8-14
8.6.2.1 Bolt Types ..................................................................................................... 8-14
8.6.2.2 Bolt Sizes ...................................................................................................... 8-14
8.6.2.3 Bolt Spacing ................................................................................................. 8-15
8.6.3 Welding ................................................................................................................ 8-15
8.6.3.1 Weld Sizes .................................................................................................... 8-15
8.6.3.2 Weld Detailing ............................................................................................... 8-15
8.18 Pedestrian Bridges .................................................................................................... 8-31
8.16 Railroad Structures .................................................................................................... 8-30
8.14 Trusses ...................................................................................................................... 8-25
8.13 Curved Girders ........................................................................................................... 8-25
8.11 Splices ....................................................................................................................... 8-21
8.9 Camber ...................................................................................................................... 8-19
8.8 Designation of Tension Zones ................................................................................... 8-19
8.7 Stiffeners ................................................................................................................... 8-18
8.6.4 Copes ....................................................................................................................... 8-17
8.6.5 Connection Design ................................................................................................. 8-18
8.5 Stiffeners ................................................................................................................... 8-18
8.7.1 Bearing Stiffeners ................................................................................................ 8-18
8.7.2 Intermediate Stiffeners and Connector Plates ..................................................... 8-18
8.7.3 Longitudinal Stiffeners ......................................................................................... 8-18
8.8 Designation of Tension Zones ................................................................................... 8-19
8.9 Camber ...................................................................................................................... 8-19
8.9.1 Sag Camber .......................................................................................................... 8-20
8.10 Moment, Shear, and Design Load Tables ............................................................... 8-20
8.11 Splices ....................................................................................................................... 8-21
8.11.1 Girder Splices ..................................................................................................... 8-21
8.11.2 Rolled Beam Splices ......................................................................................... 8-24
8.12 Framing Plans ......................................................................................................... 8-24
8.13 Curved Girders ........................................................................................................... 8-25
8.14 Trusses ...................................................................................................................... 8-25
8.14.1 General Considerations ..................................................................................... 8-25
8.14.2 Truss Design Guidelines ..................................................................................... 8-26
8.14.3 Truss Detailing Guidelines ................................................................................. 8-27
8.15 Miscellaneous Details ............................................................................................ 8-27
8.15.1 Bolsters .............................................................................................................. 8-27
8.15.2 Safety Handrail ................................................................................................... 8-29
8.16 Railroad Structures .................................................................................................. 8-30
8.16.1 General Considerations ..................................................................................... 8-30
8.16.2 Design ................................................................................................................ 8-30
8.16.3 Details ................................................................................................................ 8-30
8.17 Movable Bridges ...................................................................................................... 8-30
8.18 Pedestrian Bridges .................................................................................................. 8-31
8.18.1 General .............................................................................................................. 8-31
8.18.2 Design Guidelines ............................................................................................. 8-31
8.18.3 Detailing Guidelines ........................................................................................... 8-32
9 PRESTRESSED CONCRETE
9.1 Introduction .................................................................................................................. 9-1
9.1.1 Pretensioning ........................................................................................................ 9-1
9.1.2 Post-Tensioning ..................................................................................................... 9-1
9.2 Adjacent Prestressed Units ......................................................................................... 9-2
9.2.1 Unit Width ............................................................................................................ 9-2
9.2.2 Unit Depth ........................................................................................................... 9-2
9.2.3 Deck Overhangs ................................................................................................. 9-3
9.2.4 Longitudinal Joints ............................................................................................. 9-3
9.2.5 Skew .................................................................................................................... 9-3
9.2.6 Diaphragms and Transverse Tendons ................................................................. 9-3
9.3 Spread Precast Concrete Beam Superstructures ...................................................... 9-4
9.3.1 Spread Prestressed Box Beams ........................................................................... 9-4
9.3.2 Prestressed I-Girders ........................................................................................... 9-5
9.3.3 Type D NEXT Beams ......................................................................................... 9-5
9.4 Segmental Precast Box Girders ................................................................................ 9-6
9.4.1 Long Multi-Span Bridges ................................................................................... 9-6
9.4.2 Long Span Bridge on High Curvatures ............................................................... 9-6
# Table of Contents

9.4.3 Aesthetics ....................................................................................................... 9-6  
9.4.4 Durability ....................................................................................................... 9-6  
9.5 Bearings for Prestressed Concrete Structures ....................................................... 9-7  
9.6 Concrete Strength ............................................................................................... 9-7  
9.7 Prestressing Strand Type .................................................................................... 9-7  
9.8 Strand Pattern for Pretensioned Elements ............................................................ 9-8  
9.8.1 Precast Box and Slab Units ............................................................................ 9-8  
9.8.2 Precast I-Girders .......................................................................................... 9-8  
9.9 Tensile Stresses Due to Pretensioning ................................................................... 9-8  
9.10 Prestress Losses ............................................................................................... 9-9  
9.11 Allowable Stresses ............................................................................................. 9-10  
9.11.1 Temporary Stresses .................................................................................... 9-10  
9.11.2 Final Stresses .............................................................................................. 9-10  
9.12 Reinforcement ................................................................................................. 9-10  
9.12.1 Shear Stirrups ............................................................................................ 9-10  
9.12.2 Composite Design Reinforcement ................................................................. 9-11  
9.12.3 Anchorage Zone Reinforcement .................................................................. 9-11  
9.13 Camber .............................................................................................................. 9-11  
9.14 Stage Construction Camber Differences .............................................................. 9-11  
9.15 Continuity Design at Interior Supports ................................................................. 9-12  
9.16 Corrosion Inhibitors and Sealers ......................................................................... 9-13  
9.17 Post-Tensioned Spliced Girder Designs ............................................................... 9-13  

10 TIMBER

10.1 Introduction ....................................................................................................... 10-1  
10.2 Characteristics and Properties of Wood as a Construction Material ................. 10-1  
10.3 Types of Construction ....................................................................................... 10-1  
10.4 Selection Criteria ............................................................................................. 10-2  
10.5 Superstructure Components .............................................................................. 10-3  
10.5.1 General ....................................................................................................... 10-3  
10.5.2 Railing ........................................................................................................ 10-3  
10.5.3 Decking and Deck Bridges ......................................................................... 10-3  
10.5.4 Laminated Beam Sections ........................................................................... 10-3  
10.5.5 Special Types - Arches, Frames, and Trusses .............................................. 10-6  
10.5.6 Timber Decks with Steel Beams ................................................................. 10-6  
10.6 Substructures .................................................................................................... 10-6  
10.7 Wearing Surfaces ............................................................................................. 10-7  
10.8 Maintenance and Repairs ................................................................................ 10-7  
10.9 Conclusions ..................................................................................................... 10-7  

11 SUBSTRUCTURES

11.1 Foundations .................................................................................................... 11-1  
11.1.1 General ...................................................................................................... 11-1  
11.1.2 Spread Footings on Soil ............................................................................ 11-1  
11.1.3 Spread Footings on Rock .......................................................................... 11-1  
11.1.4 Pile Foundations ....................................................................................... 11-2  
11.1.4.1 Pile Types ............................................................................................. 11-2  
11.1.4.2 Pile Spacing and Placement Details ..................................................... 11-2
11.7 Bridge Piers .............................................................................................................. 11-25
11.6 Abutments ................................................................................................................ 11-14
11.5 Retaining Walls ......................................................................................................... 11-8
11.4 Concrete for Substructures ........................................................................................ 11-8
11.3 Forming Considerations ............................................................................................ 11-6
11.2 Substructure Joints .................................................................................................... 11-7
11.1.11 Footing Thickness ............................................................................................. 11-6
11.1.10 Tremie Seals ....................................................................................................... 11-5
11.1.9 Stepped Footings .................................................................................................. 11-7
11.1.8 Footing Depth ....................................................................................................... 11-4
11.1.7 Design Footing Pressures and Pile Capacities ....................................................... 11-4
11.1.6 Pilasters .................................................................................................................. 11-4
11.1.5 Drilled Shafts ....................................................................................................... 11-4
11.1.4 Concrete for Substructures .................................................................................... 11-6
11.1.3 Construction Joints ............................................................................................... 11-7
11.1.2 Forming Considerations ........................................................................................ 11-6
11.1.1 Footing Depth ....................................................................................................... 11-4
11.1.10 Tremie Seals ....................................................................................................... 11-5
11.1.9 Stepped Footings .................................................................................................. 11-7
11.1.8 Footing Depth ....................................................................................................... 11-4
11.1.7 Design Footing Pressures and Pile Capacities ....................................................... 11-4
11.1.6 Pilasters .................................................................................................................. 11-4
11.1.5 Drilled Shafts ....................................................................................................... 11-4
11.1.4 Concrete for Substructures .................................................................................... 11-6
11.1.3 Construction Joints ............................................................................................... 11-7
11.1.2 Forming Considerations ........................................................................................ 11-6
11.1.1 Footing Depth ....................................................................................................... 11-4
11.6 Abutments .................................................................................................................. 11-14
11.5.1 Retaining Wall Types ............................................................................................ 11-9
11.5.1.1 Cantilevered Retaining Wall .......................................................................... 11-10
11.5.1.2 Counterfort Retaining Wall .......................................................................... 11-10
11.5.1.3 Buttressed Retaining Wall ........................................................................... 11-10
11.5.1.4 Crib Wall ....................................................................................................... 11-10
11.5.1.5 Gabions .......................................................................................................... 11-10
11.5.1.6 Gravity Retaining Wall .................................................................................. 11-11
11.5.1.7 Semi-Gravity Retaining Wall ........................................................................ 11-11
11.5.1.8 M.S.E.S. Retaining Walls ............................................................................. 11-11
11.5.1.9 Cantilevered Sheet Pile Retaining Wall .......................................................... 11-11
11.5.1.10 Tied Back Sheet Pile Retaining Wall ............................................................ 11-12
11.5.1.11 Soldier Pile and Lagging Retaining Wall ....................................................... 11-12
11.5.1.12 Tied Back Soldier Pile and Lagging Retaining Wall ..................................... 11-12
11.5.2 Proportioning of Cantilevered Retaining Walls .................................................... 11-12
11.5.3 Wingwall Type and Considerations ...................................................................... 11-13
11.6 Abutments .................................................................................................................. 11-14
11.6.1 Abutment Type and Considerations ...................................................................... 11-15
11.6.1.1 Cantilevered Abutment ................................................................................. 11-16
11.6.1.2 Isolated Pedestal Stub Abutment ................................................................... 11-16
11.6.1.3 Spill Through Abutment .............................................................................. 11-16
11.6.1.4 M.S.E.S. Abutments ...................................................................................... 11-17
11.6.1.5 Gravity Abutments ....................................................................................... 11-18
11.6.1.6 Integral Abutments ......................................................................................... 11-18
11.6.1.7 Semi-Integral Abutments ............................................................................. 11-22
11.6.2 Abutment and Wall Details ................................................................................... 11-23
11.6.2.1 Stem Thickness .............................................................................................. 11-23
11.6.2.2 Pedestal Dimensions ..................................................................................... 11-24
11.6.2.3 Drainage ......................................................................................................... 11-24
11.7 Bridge Piers ................................................................................................................ 11-25
11.7.1 Pier Types .............................................................................................................. 11-25
11.7.1.1 Solid Pier ........................................................................................................ 11-26
11.7.1.2 Hammerhead Pier ....................................................................................... 11-26
11.7.1.3 Multi-Column Pier ....................................................................................... 11-26
11.7.1.4 Pile Bents ...................................................................................................... 11-27
11.7.2 Pier Protection ....................................................................................................... 11-27
12  BRIDGE BEARINGS

12.1 Bearing Types ............................................................................................................. 12-1
   12.1.1 Steel Rocker Bearings (Type S.R.) .............................................................. 12-1
   12.1.2 Steel Sliding Bearings (Type S.S.) ............................................................... 12-1
   12.1.3 Elastomeric Bearings .................................................................................... 12-1
      12.1.3.1 Plain Elastomeric Bearings (Type E.P.) ....................................... 12-2
      12.1.3.2 Steel Laminated Elastomeric Bearings (Type E.L.) ..................... 12-2
      12.1.3.3 Steel Laminated Elastomeric Bearings with Sole Plate (Type E.B.) 12-2
   12.1.4 Multi-Rotational Bearings (Type M.R.) .......................................................... 12-3

12.2 General Design Considerations .................................................................................. 12-3
   12.2.1 Design Method ............................................................................................. 12-3
   12.2.2 Live Load on Bearings .................................................................................. 12-3
   12.2.3 Minimum Loads on Bearings ........................................................................ 12-3
   12.2.4 Uplift ............................................................................................................. 12-4
   12.2.5 Bearings for Curved Girders ......................................................................... 12-4

12.3 Bearing Selection Criteria ........................................................................................... 12-4

12.4 Painting of Bearings .................................................................................................... 12-5

12.5 Standard Bearing Designs .......................................................................................... 12-5

12.6 Rehabilitation Projects ................................................................................................ 12-7
   12.6.1 New Bearings on Existing Pedestals ............................................................ 12-7
   12.6.2 New Bearings on New Pedestals ................................................................... 12-7

13  APPROACH DETAILS

13.1 Approach Slabs .......................................................................................................... 13-1
   13.1.1 Purpose ........................................................................................................ 13-1
   13.1.2 Length Determination ................................................................................... 13-1
   13.1.3 Width Determination ..................................................................................... 13-1
   13.1.4 Skewed Approach Slabs .............................................................................. 13-2
   13.1.5 End of Approach Slab Details ....................................................................... 13-2

13.2 Approach Drainage Details ......................................................................................... 13-3
   13.2.1 Purpose ........................................................................................................ 13-3
   13.2.2 Superstructures with Curbs or Barriers ........................................................ 13-3
   13.2.3 Superstructures without Curbs or Barriers .................................................... 13-3

14  BRIDGE PLAN STANDARDS AND COMMUNICATION OF DESIGN

14.1 Overview ..................................................................................................................... 14-1

14.2 Bridge Model Management ......................................................................................... 14-1
   14.2.1 General ........................................................................................................ 14-1
      14.2.1.1 CADD Files Types ........................................................................ 14-2
      14.2.1.2 Workflow Continuity ..................................................................... 14-2
   14.2.2 Bridge Design File ........................................................................................ 14-2
   14.2.3 Bridge Front File ......................................................................................... 14-4
   14.2.4 Personal Work Files ..................................................................................... 14-4
   14.2.5 Bridge Drawing Files .................................................................................... 14-5
      14.2.5.1 Drawings With Hidden Lines ....................................................... 14-6
14.2.5.2 Drawings With Cross Sections .................................................... 14-6
14.2.6 Bridge Estimate File ........................................................................ 14-6
14.2.7 Three-Dimensional Solid Models .................................................... 14-7
  14.2.7.1 2-D vs. 3-D ........................................................................ 14-7
  14.2.7.2 Substructures ........................................................................ 14-8
  14.2.7.3 Superstructures ....................................................................... 14-9
  14.2.7.4 Earthwork ............................................................................. 14-9
14.3 Detailing Standards ............................................................................. 14-11
  14.3.1 Bridge Detail (BD) Sheets ............................................................ 14-11
  14.3.2 Title Blocks ............................................................................. 14-11
  14.3.3 Scales and Scale Bars ................................................................. 14-11
  14.3.4 Dimension and Table Value Rounding ......................................... 14-12
14.4 Bridge Plan Organization .................................................................. 14-13
14.5 Bridge Plan Content .......................................................................... 14-14
14.6 Professional Engineer Stamping ......................................................... 14-18
14.7 Resolving Reviewer Comments ......................................................... 14-18
14.8 Amendments and Field Revisions ...................................................... 14-18
14.9 Design Phase Record Plan Sets ........................................................... 14-19
14.10 Project Archiving ............................................................................ 14-19
14.11 Electronic Data Transfer ................................................................... 14-20

Appendix 14A Contract Plan Review Checklist
Appendix 14B Checklist for Constructability Review

15 CONCRETE REINFORCEMENT

15.1 Introduction ..................................................................................... 15-1
15.2 Spacing .......................................................................................... 15-1
15.3 Cover ............................................................................................. 15-1
15.4 Reinforcing Bar Guidelines ............................................................... 15-2
  15.4.1 Maximum Bar Length ................................................................. 15-2
      15.4.1.1 Deck Slab Bars ................................................................. 15-2
      15.4.1.2 Abutment and Pier Bars ................................................... 15-3
  15.4.2 Reinforcement Splicing ............................................................... 15-3
      15.4.2.1 General Splicing Guidelines ............................................. 15-3
      15.4.2.2 Splicing Vertical Reinforcement in Walls ......................... 15-3
15.5 Minimum Anchorage, Lap and Embedment ...................................... 15-3
  15.5.1 Basic Development Length for Bars ......................................... 15-4
  15.5.2 Length of Splices for Tension Bars ........................................... 15-7
  15.5.3 Length of Splices for Compression Bars .................................... 15-11
15.6 Marking of Bars for Bar Lists ........................................................... 15-12
15.7 Footing Reinforcement ..................................................................... 15-13
15.8 Abutment Reinforcement ................................................................. 15-13
15.9 Column Reinforcement .................................................................... 15-13
15.10 Pier Cap Reinforcement .................................................................. 15-14
15.11 Temperature and Shrinkage Reinforcement .................................... 15-14
Table of Contents

15.12 Protecting Reinforcement from Corrosion ............................................................... 15-15
  15.12.1 Epoxy-Coated Reinforcement ......................................................................... 15-17
  15.12.2 Galvanized Reinforcement .............................................................................. 15-17
  15.12.3 Stainless Steel Clad Reinforcement ............................................................... 15-19
  15.12.4 Solid Stainless Steel Reinforcement ............................................................. 15-19
  15.12.5 Protection of Reinforcement in Substructures ............................................... 15-19

15.13 Reinforcing Bar Lists ............................................................................................ 15-20

15.14 Drilling and Grouting ........................................................................................... 15-20

16 ESTIMATE OF QUANTITIES

16.1 General ................................................................................................................. 16-1
16.2 Precision Versus Practicality .................................................................................... 16-1
16.3 Utility Share of Bridge Estimate ............................................................................. 16-2
16.4 Lump Sum Price Analysis ....................................................................................... 16-2
16.5 Alternate Bid Procedure ......................................................................................... 16-3

17 STANDARD NOTES

17.1 Introduction .............................................................................................................. 17-1
17.2 Standard Proposal Notes ......................................................................................... 17-1
17.3 General Notes Sheet/Superstructure Slab Sheet .................................................... 17-23

Appendix 17A Bridge Removal

18 SPECIAL SPECIFICATIONS

18.1 Introduction .............................................................................................................. 18-1

19 BRIDGE REHABILITATION PROJECTS

19.1 Introduction .............................................................................................................. 19-1
  19.1.1 Project Scoping ................................................................................................. 19-1
  19.1.2 Preliminary Design ......................................................................................... 19-3
  19.1.3 Final Design .................................................................................................... 19-4
19.2 Existing Structure Evaluation ................................................................................... 19-6
  19.2.1 In Depth Inspections ....................................................................................... 19-6
  19.2.2 Bridge Rehabilitation vs Replacement Selection Guidelines .......................... 19-7
19.3 Concrete Rehabilitation .......................................................................................... 19-13
  19.3.1 Concrete Scaling ............................................................................................. 19-13
  19.3.2 Concrete Spalling ......................................................................................... 19-14
  19.3.3 Concrete Cracking ......................................................................................... 19-14
  19.3.4 Concrete Sealers ............................................................................................ 19-15
NYSDOT Bridge Manual

19.4 Steel Rehabilitations ................................................................. 19-16
  19.4.1 Deck Replacements ......................................................... 19-16
  19.4.2 Structure Widening/Stage Construction ......................... 19-16
  19.4.3 Painted vs. Unpainted ..................................................... 19-17
  19.4.4 Fracture Critical Member (FCM) Work ............................. 19-17
  19.4.5 Rehabilitation of Riveted Structures ................................. 19-17
  19.4.6 A7 Steel Retrofits or Replacement ................................... 19-20
  19.4.7 Fatigue .............................................................................. 19-21
  19.4.8 Partial Length Coverplate Retrofits ................................. 19-21
19.5 Continuity Retrofit ................................................................. 19-21
  19.5.1 Feasibility ......................................................................... 19-21
  19.5.2 General Design Considerations ...................................... 19-22
    19.5.2.1 Full Continuity vs Continuous for Live Load ............ 19-22
    19.5.2.2 Fatigue Considerations ............................................ 19-24
    19.5.2.3 Detail Verification .................................................... 19-24
  19.5.3 Design Guidelines ......................................................... 19-25
19.6 Truss Rehabilitation ............................................................... 19-27
19.7 Seismic Rehabilitation ............................................................ 19-28

Appendix 19A Rehabilitation Preliminary Checklist

20 QUALITY

20.1 Introduction ......................................................................... 20-1
20.2 Technical Quality Actions .................................................. 20-1
  20.2.1 Quality Control ............................................................. 20-1
  20.2.2 Technical Progress Reviews ......................................... 20-2
  20.2.3 Quality Assurance Monitoring Reviews ....................... 20-6

21 COMPUTER PROGRAMS

21.1 Guidelines on Use ............................................................... 21-1
21.2 Hydraulics Programs ......................................................... 21-1
21.3 Structures Programs .......................................................... 21-2
  21.3.1 In-House Programs ...................................................... 21-2
  21.3.2 Commercial Programs .................................................. 21-5

22 MAINTENANCE

22.1 Introduction ......................................................................... 22-1
22.2 Geometrics ......................................................................... 22-1
22.3 Deck Joints and Drainage ................................................... 22-1
22.4 Approach Drainage ............................................................ 22-2
22.5 Superstructure .................................................................... 22-2
  22.5.1 Material Type ............................................................... 22-2
  22.5.2 Steel Details ................................................................. 22-3
22.6 Bridge Inspection and Maintenance Access Considerations ...... 22-3
22.7 Movable Bridges ............................................................... 22-3
# Table of Contents

## AESTHETICS

### 23.1 Appearance in Design

- Location and Surroundings .................................................. 23-2
- Horizontal and Vertical Geometry .......................................... 23-3
- Superstructure Type and Shape ............................................. 23-3
- Pier Shape and Placement .................................................... 23-8
- Abutment Shape and Placement ........................................... 23-16
  - Skew .................................................................................. 23-17
  - Wingwalls and Curtainwalls .............................................. 23-18
- Parapet and Railing Details .................................................. 23-19
- Colors ................................................................................ 23-21
- Textures ............................................................................. 23-22
- Ornamentation .................................................................... 23-24

Appendix 23A

Glossary
# List of Figures

<table>
<thead>
<tr>
<th>Figure Number</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Curved Alignment Layout</td>
<td>2-9</td>
</tr>
<tr>
<td>2.2</td>
<td>Schematic of a Median Berm</td>
<td>2-15</td>
</tr>
<tr>
<td>2.3</td>
<td>Typical Canal Channel Sections</td>
<td>2-20</td>
</tr>
<tr>
<td>2.4</td>
<td>Canal Pier Details</td>
<td>2-21</td>
</tr>
<tr>
<td>2.5</td>
<td>Railroad Clearance Diagram</td>
<td>2-23</td>
</tr>
<tr>
<td>2.6</td>
<td>Track on Embankment</td>
<td>2-24</td>
</tr>
<tr>
<td>2.7</td>
<td>Track in Cut</td>
<td>2-25</td>
</tr>
<tr>
<td>2.8</td>
<td>Typical Railroad Rock Cut Section</td>
<td>2-26</td>
</tr>
<tr>
<td>2.9</td>
<td>Typical Thru Girder Railroad Bridge</td>
<td>2-28</td>
</tr>
<tr>
<td>2.10</td>
<td>Banking Simple Curve</td>
<td>2-33</td>
</tr>
<tr>
<td>2.11</td>
<td>Banking Spiral Curve</td>
<td>2-34</td>
</tr>
<tr>
<td>2.12</td>
<td>Banking Details for Bridge Decks</td>
<td>2-35</td>
</tr>
<tr>
<td>2A.1</td>
<td>Usable Shoulder Details</td>
<td>2A-5</td>
</tr>
<tr>
<td>3.1</td>
<td>Shoulder Break Area</td>
<td>3-8</td>
</tr>
<tr>
<td>5.1</td>
<td>Overhang Form Bracing</td>
<td>5-9</td>
</tr>
<tr>
<td>5.2</td>
<td>Haunch Table</td>
<td>5-12</td>
</tr>
<tr>
<td>5.3</td>
<td>Haunch Detail (Cracking Problem)</td>
<td>5-12</td>
</tr>
<tr>
<td>5.4</td>
<td>Slab Placement Sequence - A</td>
<td>5-17</td>
</tr>
<tr>
<td>5.5</td>
<td>Slab Placement Sequence - B</td>
<td>5-17</td>
</tr>
<tr>
<td>5.6</td>
<td>Slab Placement Sequence - C</td>
<td>5-17</td>
</tr>
<tr>
<td>8.1</td>
<td>Cover Plate Connections</td>
<td>8-12</td>
</tr>
<tr>
<td>8.2</td>
<td>Reinforced Cope Detail</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3</td>
<td>Blocked Flange Detail</td>
<td>8-17</td>
</tr>
<tr>
<td>8.4</td>
<td>Low Bolster Detail</td>
<td>8-28</td>
</tr>
<tr>
<td>8.5</td>
<td>High Bolster Detail</td>
<td>8-29</td>
</tr>
<tr>
<td>10.1</td>
<td>Longitudinal Stress Laminated Deck</td>
<td>10-4</td>
</tr>
<tr>
<td>10.2</td>
<td>Parallel Chord Truss</td>
<td>10-4</td>
</tr>
<tr>
<td>10.3</td>
<td>'T' Section Bridge</td>
<td>10-5</td>
</tr>
<tr>
<td>10.4</td>
<td>Box Section Bridge</td>
<td>10-5</td>
</tr>
<tr>
<td>11.1</td>
<td>Typical Retaining Wall Types</td>
<td>11-9</td>
</tr>
<tr>
<td>11.2</td>
<td>Suggested Proportions of Retaining Wall</td>
<td>11-13</td>
</tr>
<tr>
<td>11.3</td>
<td>Typical Abutment Types</td>
<td>11-15</td>
</tr>
<tr>
<td>11.4</td>
<td>Bridge Seat Width</td>
<td>11-23</td>
</tr>
<tr>
<td>11.5</td>
<td>Typical Pier Types</td>
<td>11-25</td>
</tr>
<tr>
<td>Figure Number</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>---------------</td>
<td>------------------------------------------------------------------------------------------------</td>
<td>----------</td>
</tr>
<tr>
<td>15.1</td>
<td>Hooked Dowel</td>
<td>15-4</td>
</tr>
<tr>
<td>19.1</td>
<td>Typical Retrofit Details</td>
<td>19-23</td>
</tr>
<tr>
<td>23.1</td>
<td>Beam Depth Comparison</td>
<td>23-4</td>
</tr>
<tr>
<td>23.2</td>
<td>Visual Effect of Slenderness Ratios</td>
<td>23-4</td>
</tr>
<tr>
<td>23.3</td>
<td>Slender Superstructures</td>
<td>23-5</td>
</tr>
<tr>
<td>23.4</td>
<td>Continuous Girder Depth</td>
<td>23-5</td>
</tr>
<tr>
<td>23.5</td>
<td>Overhang Shadowing</td>
<td>23-6</td>
</tr>
<tr>
<td>23.6</td>
<td>Avoid Stiffeners on the Exposed Side of the Fascia Girders</td>
<td>23-6</td>
</tr>
<tr>
<td>23.7</td>
<td>Haunched Girders</td>
<td>23-7</td>
</tr>
<tr>
<td>23.8</td>
<td>Haunch Details</td>
<td>23-7</td>
</tr>
<tr>
<td>23.9</td>
<td>Fishbellied Girders</td>
<td>23-8</td>
</tr>
<tr>
<td>23.10</td>
<td>Pier Height</td>
<td>23-8</td>
</tr>
<tr>
<td>23.11</td>
<td>Pier/Column Thickness</td>
<td>23-9</td>
</tr>
<tr>
<td>23.12</td>
<td>Alternate Column Treatments</td>
<td>23-10</td>
</tr>
<tr>
<td>23.13</td>
<td>Pier Layout Details</td>
<td>23-11</td>
</tr>
<tr>
<td>23.14</td>
<td>End View of Capbeam</td>
<td>23-12</td>
</tr>
<tr>
<td>23.15</td>
<td>Overhang Alternatives</td>
<td>23-13</td>
</tr>
<tr>
<td>23.16</td>
<td>Solid Pier Shapes</td>
<td>23-14</td>
</tr>
<tr>
<td>23.17</td>
<td>Battered Solid Piers</td>
<td>23-15</td>
</tr>
<tr>
<td>23.18</td>
<td>Tall Pier Configurations</td>
<td>23-15</td>
</tr>
<tr>
<td>23.19</td>
<td>Pier Groupings</td>
<td>23-16</td>
</tr>
<tr>
<td>23.20</td>
<td>Abutment Details</td>
<td>23-17</td>
</tr>
<tr>
<td>23.21</td>
<td>Abutments on a Skew</td>
<td>23-18</td>
</tr>
<tr>
<td>23.22</td>
<td>Wingwall Configuration</td>
<td>23-19</td>
</tr>
<tr>
<td>23.23</td>
<td>End of Barrier Detail</td>
<td>23-20</td>
</tr>
<tr>
<td>23.24</td>
<td>Concrete Barrier Treatments</td>
<td>23-20</td>
</tr>
<tr>
<td>23.25</td>
<td>Fencing Alternatives</td>
<td>23-21</td>
</tr>
<tr>
<td>23.26</td>
<td>Wingwall Stone/Brick Treatment</td>
<td>23-23</td>
</tr>
</tbody>
</table>
List of Tables

<table>
<thead>
<tr>
<th>Table Number</th>
<th>Description</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1</td>
<td>Clear Bridge Roadway Width Standards</td>
<td>2-6</td>
</tr>
<tr>
<td>2-2</td>
<td>Vertical Clearance over Highways (Travel Lane and Paved Shoulders)</td>
<td>2-11</td>
</tr>
<tr>
<td>2-3</td>
<td>Lateral Offset from Centerline of Tracks</td>
<td>2-27</td>
</tr>
<tr>
<td>R</td>
<td>Minimum roadway Widths for and Replacement</td>
<td>2A-2</td>
</tr>
<tr>
<td>N</td>
<td>Maximum width of Traveled Way and Shoulder</td>
<td>2A-3</td>
</tr>
<tr>
<td>X</td>
<td>Minimum Roadway for Rehabilitations</td>
<td>2A-4</td>
</tr>
<tr>
<td>3-1</td>
<td>Hydraulic Data Table</td>
<td>3-4</td>
</tr>
<tr>
<td>3-2</td>
<td>Multiple Span Arrangement Ratios</td>
<td>3-12</td>
</tr>
<tr>
<td>4-1</td>
<td>Soil Design Parameters</td>
<td>4-5</td>
</tr>
<tr>
<td>4-2</td>
<td>Excavation Requirements</td>
<td>4-11</td>
</tr>
<tr>
<td>4-3</td>
<td>Support and Protection System Requirements</td>
<td>4-12</td>
</tr>
<tr>
<td>4-4</td>
<td>Cofferdam Requirements</td>
<td>4-13</td>
</tr>
<tr>
<td>5-1</td>
<td>Deck Thickness Requirements</td>
<td>5-1</td>
</tr>
<tr>
<td>5-2</td>
<td>Traditional Transverse Deck Slab Reinforcement</td>
<td>5-7</td>
</tr>
<tr>
<td>5-3</td>
<td>Design Haunch Depth Table</td>
<td>5-11</td>
</tr>
<tr>
<td>5-4</td>
<td>Aggregate Type Selection</td>
<td>5-23</td>
</tr>
<tr>
<td>6-1</td>
<td>Railing and Barrier Selection</td>
<td>6-4</td>
</tr>
<tr>
<td>6-2</td>
<td>Railing/Barrier Design Dead Loads</td>
<td>6-9</td>
</tr>
<tr>
<td>8-1</td>
<td>Steel Plate Thicknesses</td>
<td>8-13</td>
</tr>
<tr>
<td>12-1</td>
<td>Bearing Nomenclature</td>
<td>12-5</td>
</tr>
<tr>
<td>12-2</td>
<td>Bearing Design – Standard Type EL Elastomeric</td>
<td>12-6</td>
</tr>
<tr>
<td>12-3</td>
<td>Bearing Design – Standard Type EB Elastomeric</td>
<td>12-6</td>
</tr>
<tr>
<td>14-1</td>
<td>Suggested Sheet Scales</td>
<td>14-12</td>
</tr>
<tr>
<td>14-2</td>
<td>Dimension Rounding Guidelines</td>
<td>14-12</td>
</tr>
<tr>
<td>15-1</td>
<td>Minimum Reinforcement Cover</td>
<td>15-1</td>
</tr>
<tr>
<td>A</td>
<td>Standard Reinforcing Bar Properties</td>
<td>15-2</td>
</tr>
<tr>
<td>B</td>
<td>Basic Development Length (BDL) for Compression Bars</td>
<td>15-4</td>
</tr>
<tr>
<td>C</td>
<td>BDL of Hooked Dowels in Tension</td>
<td>15-4</td>
</tr>
<tr>
<td>D</td>
<td>BDL for Straight Uncoated Dowels &amp; Tension Bars (Not Top Bars)</td>
<td>15-5</td>
</tr>
<tr>
<td>E</td>
<td>BDL for Straight Uncoated Dowels &amp; Tension Bars (Top Bars)</td>
<td>15-5</td>
</tr>
<tr>
<td>F</td>
<td>BDL for Straight Epoxy Coated Dowels &amp; Tension Bars (Not Top Bars)</td>
<td>15-5</td>
</tr>
<tr>
<td>G</td>
<td>BDL for Straight Epoxy Coated Dowels &amp; Tension Bars (Top Bars)</td>
<td>15-5</td>
</tr>
<tr>
<td>H</td>
<td>BDL Reduction Requirements</td>
<td>15-6</td>
</tr>
<tr>
<td>I</td>
<td>Length of Splices for Tension Bars</td>
<td>15-7</td>
</tr>
<tr>
<td>Table Number</td>
<td>Description</td>
<td>Page No.</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>15-2</td>
<td>Lap Splice Selection Guidelines</td>
<td>15-8</td>
</tr>
<tr>
<td>J</td>
<td>Class B Splice – Uncoated (Not Top Bars)</td>
<td>15-9</td>
</tr>
<tr>
<td>K</td>
<td>Class B Splice – Uncoated (Top Bars)</td>
<td>15-9</td>
</tr>
<tr>
<td>L</td>
<td>Class C Splice – Uncoated (Not Top Bars)</td>
<td>15-9</td>
</tr>
<tr>
<td>M</td>
<td>Class C Splice – Uncoated (Top Bars)</td>
<td>15-10</td>
</tr>
<tr>
<td>N</td>
<td>Class B Splice – Epoxy Coated (Not Top Bars)</td>
<td>15-10</td>
</tr>
<tr>
<td>O</td>
<td>Class B Splice – Epoxy Coated (Top Bars)</td>
<td>15-10</td>
</tr>
<tr>
<td>P</td>
<td>Class C Splice – Epoxy Coated (Not Top Bars)</td>
<td>15-11</td>
</tr>
<tr>
<td>Q</td>
<td>Class C Splice – Epoxy Coated (Top Bars)</td>
<td>15-11</td>
</tr>
<tr>
<td>R</td>
<td>Length of Splices for Compression Bars</td>
<td>15-11</td>
</tr>
<tr>
<td>15-3</td>
<td>Approximate Reinforcement Cost Comparison</td>
<td>15-16</td>
</tr>
<tr>
<td>15-4</td>
<td>Expected Service Life</td>
<td>15-17</td>
</tr>
<tr>
<td>15-5</td>
<td>Hooks for Galvanized Bars</td>
<td>15-18</td>
</tr>
<tr>
<td>16-1</td>
<td>Precision for Estimate of Quantities</td>
<td>16-2</td>
</tr>
<tr>
<td>19-1</td>
<td>Current Bridge Standards</td>
<td>19-9</td>
</tr>
<tr>
<td>19-2</td>
<td>Bridge Rehabilitation vs. Replacement Worksheet</td>
<td>19-12</td>
</tr>
<tr>
<td>19-3</td>
<td>Concrete Cracking Treatments</td>
<td>19-15</td>
</tr>
<tr>
<td>20-1</td>
<td>Bridge Plan Technical Progress Reviews</td>
<td>20-4</td>
</tr>
</tbody>
</table>
Foreword

This Bridge Manual is intended to serve as an aid to those planning and designing bridges in New York State. It is an accompaniment to the NYSDOT Standard Specifications for Highway Bridges and NYSDOT LRFD Bridge Design Specifications. It is hoped that it will serve as a guide to good bridge engineering practice.

George A. Christian Jr., P.E.
Deputy Chief Engineer (Structures)
Acknowledgments

This manual started as an update of the old Standard Details for Highway Bridges. However, it became evident that a number of new topics needed to be included and a greater commentary on both detailing and design practice for bridges was needed. An attempt has also been made to compile and incorporate as many of the outstanding Structures Division’s Engineering Instructions as possible. It is hoped that this will assist engineers and drafters by having a concise reference source. The result of this effort is the NYSDOT Bridge Manual.

This manual is the product of the work of many contributors over the last 10 years without whose efforts this project would not have been possible. My thanks to all who dealt with compiling and sifting mountains of information, writing text, resolving numerous comments and completing endless rewrites.

Appreciation is also given to the many individuals in the regions and main office that reviewed drafts of this manual. Their many insightful comments have done much to improve its content.

James H. Flynn III, P.E.
Editor

January 2008
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Section 1
Introduction

1.1 Purpose

This Bridge Manual has been prepared to provide policies, guidance and procedures for bridge project development and design for the New York State Department of Transportation. This manual provides guidance for decisions in the bridge project process, documents or references policies and standards that need to be considered, and provides a commentary discussing good bridge engineering practice.

One of the primary goals of this manual is to provide assistance to designers to ensure that “quality” bridges are constructed. “Quality” bridges are durable, economical, aesthetically pleasing, safe, and environmentally sound.

Although this manual provides guidance on design procedure, many subjects presented only highlight criteria and practice. A complete analysis and design to produce a safe, economical and maintainable structure is the responsibility of the designer.

1.2 Applicability

This manual applies to all bridges constructed under contracts with the New York State Department of Transportation. Designers are required to consult the manual for policies, guidance, details and interpretation of the design specifications. In addition, its use is encouraged for all bridges in New York State.

Highway and pedestrian bridge design are governed by the design specifications contained in the most recent issuance of the NYSDOT LRFD Bridge Design Specifications or the New York State Department of Transportation Standard Specifications for Highway Bridges. This manual does not replace the provisions of these specifications. It is intended to supplement the design specifications in areas that are not addressed or fully covered. Additional information on the design of facilities for pedestrians, bicycles, and persons with disabilities may be found in Chapters 17 and 18 of the Highway Design Manual.

Major long span bridges are special cases for bridge design. They typically need special design criteria which go beyond the provisions of the NYSDOT LRFD Bridge Design Specifications. The NYSDOT LRFD Bridge Design Specifications do not have an explicit span limitation, however, the commentary states that spans in excess of 600 feet were not considered in its development.
Major long span bridges should have specific bridge design criteria developed once the bridge type has been selected and before final design begins. If during preliminary development it is determined that the NYSDOT LRFD Bridge Design Specifications do not cover all aspects of the structure design appropriate supplemental design criteria should be developed by researching design criteria for similar structures in the US and Canada.

1.3 Policy

NYSDOT has officially adopted the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications for use in New York State. The AASHTO LRFD Bridge Design Specifications and the “NYSDOT LRFD Blue Pages” constitute the NYSDOT LRFD Bridge Design Specifications. The adoption of these specifications continues a process in which NYSDOT has transitioned from the NYSDOT Standard Specifications for Highway Bridges to full adoption of the LRFD specifications. The NYSDOT Standard Specifications for Highway Bridges consists of the 17th edition of the AASHTO Standard Specifications for Highway Bridges together with the New York State “Blue Pages.” The NYSDOT LRFD Bridge Design Specification is mandatory for the design of all new and replacement bridges by NYSDOT and Consultant designers. The FHWA mandated a full implementation date on October 1, 2007 for all State-initiated Federal-aid funded projects. For locally administered Federal-Aid projects see Chapter 9 of the NYSDOT Procedures for Locally Administered Federal-Aid Projects Manual.

The existing NYSDOT Standard Specifications for Highway Bridges are now archived and used when necessary for the repair and rehabilitation of structures. The design specifications that may be used for rehabilitation and repair projects include the LRFD Specifications, the Standard Specifications or the specifications used in the original design.

Load Ratings – Currently, NYSDOT overload permitting and bridge posting policies require that new and replacement bridges be load rated using the Load Factor Design (LFD) or Allowable Stress Design (ASD) methods. For this reason, load ratings will continue to be computed by the LFD or ASD method. The load ratings for all new or replacement bridges will also be computed by the Load and Resistance Factor Rating (LRFR) method. Load rating for both methods shall be shown on the Contract Plans. LRFR ratings shall be shown at the inventory and operating levels as rating factors of the AASHTO HL-93 load. Once overload permitting and bridge posting policies are revised to accommodate LRFR, load ratings using LFD and ASD methods will be discontinued.

Buried Structures – Buried structures include box culverts, three-sided frames, and pipes. Any buried structure designs begun in October 2010 or later shall be designed by LRFD. For designs begun before October 2010 designers may continue to use either the NYSDOT Standard Specifications for Highway Bridges or the LRFD specifications for the design of buried structures.
1.4 Referenced Standards, Manuals and Documents

The following references contain material that is relevant to bridge project development and design. They contain provisions that pertain to a particular type of bridge or part of the bridge project process. Instead of reproducing them in full in this manual, they are incorporated by reference. Bridge designers need to consider their provisions where applicable.

The NYSDOT references can be found at [https://www.dot.ny.gov/publications](https://www.dot.ny.gov/publications).

The Bridge Detail (BD) Sheets referenced below contain standard details and, occasionally, instructions to designers on material that is to be incorporated into the Contract Plans.

- American Railway Engineering & Maintenance of Way Association Manual for Railway Engineering (AREMA)
- NYSDOT Bridge Deck Evaluation Manual
- NYSDOT Bridge Detail (BD) Sheets
- NYSDOT Bridge Inspection Manual
- NYSDOT Bridge Inventory Manual
- NYSDOT Bridge Safety Assurance Vulnerability Manuals
- NYSDOT CADD Standards and Procedure Manual
- NYSDOT Structures Division Cell Library
- NYSDOT Project Development Manual
- NYSDOT Environmental Procedures Manual/The Environmental Manual
- NYSDOT Highway Design Manual
- NYSDOT Manual of Uniform Traffic Control Devices
- NYSDOT Prestressed Concrete Construction Manual (PCCM)
- NYSDOT Standard Specifications for Construction and Materials
- NYSDOT LRFD Bridge Design Specifications
- NYSDOT Steel Construction Manual (SCM)
- NYSDOT Procedures for Locally Administered Federal Aid Projects
- NYSDOT Survey Manual
- FHWA Seismic Retrofitting Manual for Highway Bridges
- AASHTO Guide for the Planning, Design, and Operation of Pedestrian Facilities
- AASHTO LRFD Movable Highway Bridge Design Specifications
- AASHTO Maintenance and Management of Roadways and Bridges Manual
- AASHTO Manual for Bridge Evaluation
- AASHTO Roadside Design Guide
- AASHTO Guide for the Development of Bicycle Facilities
- AASHTO Guide Specification and Commentary for Vessel Collision of Highway Bridges
- AASHTO Manual for Assessing Safety Hardware
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| **Bridge Deck Repair** | That type of work that is intended to return the structural deck of an existing bridge to a condition of suitable ride quality and/or safe wheel load capacity. The deck may be composed of concrete, steel or other material, and the type of construction may include monolithic decks as well as separate wearing surfaces over a slab. The restorative work may include overlay or separate wearing surfaces (with or without a waterproof membrane) over the whole deck area of the bridge or over substantial areas. For purposes of this policy, a complete bridge deck replacement should be classified as a bridge rehabilitation. Under this policy, bridge deck repair done in conjunction with other superstructure or substructure restoration work also should be classified as a bridge rehabilitation. A bridge deck repair project may include some incidental structure repair work that is related to the deck repair work (e.g., header or backwall repair). |
| **Clear Bridge Roadway Width** | The clear distance between inside faces of bridge railing, or the clear distance between faces of curbs, whichever is less. The typical Department 5-inch wide brush curb (introduced at the bridge only) shall not be considered to reduce the rail-to-rail dimension. |
| **Design Speed** | A speed used to determine the physical features of a highway. It may or may not be equal to the statewide limit or to the posted speed limit at the bridge site. The design speed is determined according to Chapter 2 of the *Highway Design Manual*. |
| **Federal-Aid Project** | A bridge or highway project that is to be funded, either entirely or partially, with Federal-aid funds. |
| **Highway Project** | A construction project whose primary objective is to construct a new highway, or to reconstruct, or to restore and preserve, an existing highway. The project may include bridge work of any type that is incidental to the primary objective. |
| **Narrow Bridge** | A bridge carrying two-way traffic, but less than 18 ft in clear width between railing or curbs, or a one-way ramp less than 12 ft. wide. |
| **National Highway System (NHS)** | A network of major roads that were designated by the Federal Highway Administration in consultation with the individual states and signed into law in November 1995. |

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2 A list of designated NHS Highways is contained in the “National Highway System Route Listing” and is maintained by the Highway Data Services Bureau of the Office of Technical Services.
New Bridge
A bridge constructed to serve a new or relocated highway that is not intended to serve as a substitute for an existing bridge being removed as part of the same project. It shall be considered a new bridge when a bridge is constructed to ultimately become a substitute for an existing bridge which will be removed in a subsequent project.

One Lane Bridge
A particular type of narrow bridge, carrying two-way traffic but less than 16 ft. in clear width between railing or curbs.

Pedestrian/Bicycle Bridge
A structure provided specifically for the travel of bicyclists and pedestrians, frequently as part of a shared use path facility.

Planned Improvements
Improvements to the roadway width projected within a 20-year planning horizon. They do not necessarily need to be programmed. These are, however, documented plans the Department or local municipality hopes to accomplish when funding becomes available and when it fits into the Region's or local agency's capital program. Whether or not there are planned improvements shall be addressed in the scoping documentation used to establish the project design criteria. Refer to the Project Development Manual for requirements on addressing planned improvements in project scoping and development.

Roadway
That portion of a highway, including all through traffic lanes, auxiliary lanes, and shoulders, suitable for vehicular use. Also referred to as "surfacing" or "pavement."

Shoulder
That portion of the roadway, graded but not necessarily paved or surfaced, for accommodation of stopped vehicles, for emergency use and for lateral support of subcourses and surface courses. For purposes of this policy, the shoulder shall refer to the usable shoulder (see Appendix 2A for illustrations of shoulders). For applying this policy, the existing approach shoulders should be measured no closer than 100 ft. from the ends of existing bridges. If the approach shoulder width varies, a determination must be made of what the most typical shoulder width is for that section of highway. Be aware that providing the typical width may cause the project limits to be extended slightly to widen the varying shoulder.

Sidewalks
Space provided on a structure exclusively for the use of pedestrian travel, generally separated from the roadway by a raised curb. See Chapter 18 of the Highway Design Manual.

Surfaced Shoulder
A roadway shoulder that is paved, or stabilized and maintained with a bituminous or other similar surface treatment.

Traveled Way
That portion of the roadway exclusive of shoulders, designed for the movement of vehicles.
2.3  Clear Roadway Width Standards for Bridges

2.3.1  General

Unless specifically noted in the provisions, the geometric design standards provided in this section shall apply to all projects, whether or not the project is a Federal-Aid Project. For purposes of this policy the "AASHTO Policy" shall refer to the AASHTO A Policy on Geometric Design of Highways and Streets, 2011.

Bridge Widths and Bridge Approach Widths:

Bridge widths shall be established consistent with Table 2-1, Clear Bridge Roadway Width Standards. Bridge widths that do not meet the requirements of Table 2-1 Clear Bridge Roadway Width Standards shall be documented as a nonstandard feature and approval shall be requested from the grantor of Design Approval, as outlined in Chapter 4 of the Project Development Manual.

For bridge replacements or rehabilitations that are not part of a highway project, the bridge widths determined from this policy shall also be used for the widths of any highway reconstruction work necessary on the approaches to the bridge.

Bridge approach widths for bridges that are part of a highway project shall be determined according to Chapter 2 of the Highway Design Manual.

Policy Exceptions: Unless there is a clear safety issue involved, bridge widths greater than the minimums described below should not be used, except where extenuating circumstances exist. The use of bridge widths for a particular project that do not meet the requirements of this policy shall be documented as a nonstandard feature. Approval shall be requested from the grantor responsible for Design Approval, as noted in Chapter 4 of the Project Development Manual.

Bridges with adjacent prefabricated concrete superstructures may have a greater width because of economic considerations discussed in Section 9.2.1.

2.3.2  Railroad Bridges

Each individual railroad will be responsible for providing a trackage section showing horizontal offsets and clearance diagrams for the bridge. The distance between the centers of multiple tracks shall also be set by the railroad. The Rail Agreements Section in the Design Quality Assurance Bureau should be contacted to assist in obtaining these design parameters. Also, see Section 2.5.3 for more details.
<table>
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<th>Type of Bridge Work</th>
<th>No Planned Improvement</th>
<th>Planned Improvement</th>
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<tr>
<td></td>
<td>Rehab</td>
<td>Approach roadway width. Highway Design Manual (HDM) Chapter 2 roadway widths are desirable (12 ft. min. lanes, 10 ft. min. right shoulder and 4 ft. min. left shoulder).</td>
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<td>Replace</td>
<td>Undivided Arterial: Wider of approach roadway width OR existing traveled way plus 4 ft. min. shoulders. Divided Arterial: Wider of approach roadway width OR existing traveled way plus 4 ft. right shoulders and match existing left shoulders. (A 4 ft. min. left shoulder is desirable.)</td>
<td>HDM Chapter 2 roadway widths shall be met.</td>
</tr>
<tr>
<td></td>
<td>Rehab</td>
<td>Undivided Arterial: Wider of approach roadway width OR existing traveled way plus 2 ft. shoulders. (4 ft. min. shoulders are desirable.) Divided Arterial: Wider of approach roadway width OR existing traveled way plus 2 ft. right shoulder and match existing left shoulder. (4 ft. min. shoulders are desirable.)</td>
<td></td>
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<tr>
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<td>New</td>
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<td>HDM Chapter 2 roadway widths shall be met.</td>
</tr>
<tr>
<td></td>
<td>Replace</td>
<td>Approach roadway width but not less than HDM Chapter 2 roadway widths where bicycles are a consideration.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rehab</td>
<td>Approach roadway width. (4 ft. min. shoulders are desirable).</td>
<td></td>
</tr>
<tr>
<td>Rural or Urban Collectors and Local Roads/Streets</td>
<td>New/Replace</td>
<td>Approach roadway width but not less than Table R of Appendix 2A or greater than Table N of Appendix 2A.</td>
<td>Full width of planned roadway but not less than Table R of Appendix 2A or greater than Table N of Appendix 2A.</td>
</tr>
<tr>
<td></td>
<td>Rehab</td>
<td>Approach roadway width but not less than Table X of Appendix 2A or greater than Table N of Appendix 2A. (4 ft. min. shoulders are desirable).</td>
<td>Full width of planned roadway but not less than Table X of Appendix 2A or greater than Table N of Appendix 2A.</td>
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</tr>
<tr>
<td>Shared-Use</td>
<td>All</td>
<td>The minimum clear width shall be 12 ft.</td>
<td></td>
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Notes:
1. The accident history and other operational conditions shall be analyzed prior to determining there are no planned improvements or that the existing roadway width can be retained. See HDM Section 5.3 for guidance.
2. Parking lanes on the approaches are not included in the approach roadway width. However, they may be considered on bridges less than 50 ft. in length.
3. If sidewalks are proposed at both ends of the bridge, they shall be carried across the bridge. For discussion of minimum width of sidewalk see Section 2.3.3 Miscellaneous Bridge Width Considerations Curbs.
4. When establishing the width of a bridge that has different approach roadway widths, select a width that provides the most safety, consistency and economy in consultation with the Structures Design Quality Assurance Bureau’s Project Development Unit and the Design Quality Assurance Bureau’s Project Development Section.
5. See Appendix 2B One-Lane Bridge Policy for one-lane bridge replacement.
6. A reduction in shoulder widths may be considered for long structures. For guidance see Section 2.3.3 Miscellaneous Bridge Width Considerations.

Table 2-1
Clear Bridge Roadway Width Standards
Figure 2.1
Curved Alignment Layout
Miscellaneous: A reduction in shoulder widths may be considered for long structures. For these structures consideration may be given to reducing the 10 ft. or 8 ft. right shoulder to a minimum of 6 ft. The possibility of vehicle breakdowns should be accommodated with minimum shoulder widths of 4 ft. left and 6 ft. right.

In urban areas, parking lanes are not normally carried across bridges and shall only be considered for bridges less than 50 ft.

In urban areas sidewalk widths greater than the minimum may be carried across the structure.

2.4 Vertical Clearances

2.4.1 Over Highways for Highway, Pedestrian, and Overhead Sign Structures

Minimum vertical clearance requirements over highways help accommodate the movement of large vehicles for maintenance operations, utility work, and the transport of people, products, construction equipment, military equipment for national defense, etc. To facilitate the movement of large vehicles, the Federal government established a 16 ft. vertical clearance network that consists of the National Highway System (NHS), with a few exceptions. The NHS includes:

- All routes on the Interstate System.
- The Strategic Highway Corridor Network (STRAHNET) and its highway connectors to major military installations. The STRAHNET includes highways important to the United States strategic defense policy and which provide defense access, continuity, and emergency capabilities for the movement of personnel, materials, and equipment in both peace time and war time.
- Other major routes, as established by the 1995 NHS Act.

The following portions of the NHS are exempted from the 16 ft. vertical clearance route:

- Parkways.
- Portions of the New York State Thruway, I-90, and I-190 (See Appendix 2C.)
- All NHS routes within an urban area which has a federally approved 16 ft. vertical clearance routing (For a list of the routes contact the appropriate Regional Planning and Program Manager.) Note that portions of the STRAHNET within the urban area must still have a 16 ft. vertical clearance.

The Regional Planning and Program Management Group should be contacted to determine if the route is part of the 16 ft. vertical clearance network.

Vertical clearances shall be established consistent with Table 2-2 Vertical Clearance Over Highways (Travel Lane and Paved Shoulder). If the minimum vertical clearance cannot be met, a nonstandard feature justification, prepared in accordance with the Highway Design Manual, Chapter 2, Section 2.8, is required. Appendix 2C of the Bridge Manual describes the substitute 16 ft. network for the New York State Thruway for which no exception to the 16 ft. vertical clearance can be entertained. Appendix 2D contains the special procedures for nonstandard vertical clearances over the Interstate System.
2.4.4 **Navigable Waterways**

The only waterway in New York State that has prescribed requirements for vertical clearances is the New York State Barge Canal System. The minimum requirements are as follows:

- Champlain Canal, Cayuga-Seneca Canal, and Erie Canal (west of Three Rivers) have a minimum vertical clearance of 15'-6" above maximum navigable pool elevation. The channel depth shall be no less than 12 ft. from normal pool elevation.

- Oswego Canal and Erie Canal (from Waterford west to Three Rivers) have a minimum vertical clearance of 20 ft. above maximum navigable pool elevation. The channel depth shall be no less than 14 ft. from normal pool elevation.

**NOTE:** Variances for reductions will not be granted for channel depth or vertical clearance standards.

Bridges undergoing replacement or major rehabilitation that do not currently provide these minimum requirements shall be designed to comply with the prescribed vertical clearances. In some instances, the existing bridge exceeds the minimum clearances. This does not always mean that a replacement or rehabilitation project may reduce the existing vertical clearance. Coordination with the N.Y.S. Canal Corporation in early project development is required to determine the acceptable vertical clearance.

Other navigable waterways such as the Hudson River (south of Albany), St. Lawrence River/Seaway, etc., may fall under the jurisdiction of other local, state and federal agencies, commissions, and/or authorities. These agencies may have their own requirements for vertical clearance to be provided or may desire to increase or decrease the existing vertical clearance. In instances that involve a state owned bridge, coordination between all the interested parties is necessary to achieve the most appropriate vertical clearance.

Vertical clearance for other navigable waterways may be determined in many ways; i.e. existing, upstream and downstream clearances, type and size of vessels utilizing the waterway, etc. This information is also valuable in considering the need to provide pier protection (refer to Section 2.5 - Horizontal Clearances: Under-Bridge Features). Ordinary High-Water elevation for nontidal or Mean High Water for tidal areas will be used when determining minimum vertical clearance. Water depth will be determined from Normal Pool Elevation in nontidal waters or Mean Sea Level in tidal areas.

2.4.5 **Miscellaneous Vertical Clearance Criteria**

**Thru-Truss** - The end portals of all newly designed highway trusses shall allow for 16 ft. of vertical clearance plus an additional 6 inches to accommodate oversize vehicles and future overlays.

**Flood Control Project** - Where a bridge project crosses an established or proposed flood control project, the responsible agency (e.g., U.S. Army Corps of Engineers) will establish the desired vertical clearance over the Floodway Project Design Elevation. The Hydraulics Unit of the Office of Structures will provide assistance in obtaining the criteria.
Trails/Shared-Use Paths - Structures crossing over existing or proposed recreational trails/shared-use paths shall provide a minimum of 10 ft. vertical clearance with 12 ft. preferred. The minimum vertical clearance over a pedestrian only path is 8 ft. with 10 ft. preferred. Consideration should be given to providing vertical clearance for authorized/maintenance vehicles when necessary.

Canal Trails - Along all sections of the canal system, access corridors are being established. This system of trails on the banks parallel to the canal should also provide, when possible, 10 ft. of vertical clearance. At locations with a trail on each side, a vertical clearance of at least 13 ft. should be provided, if possible, on at least one side. This will allow access for maintenance equipment such as small cranes and dump trucks. Early coordination with the Canal Corporation is recommended.

Extended Berm (Bench) - In places where an abutment has a larger than standard berm in front of the bridge seat a minimum clearance of 3 ft. is desired between the bottom of the low beam elevation and the top of the bench. This provides access for inspection of the underside of the superstructure.

Parkways - Table 2-1 shall be followed for vertical clearance requirements. However, many structures crossing parkways are required to be of certain configuration, i.e., arches, frames, etc. These configurations can significantly affect horizontal and vertical clearances. If there are considerable constraints on profile adjustments and if the required minimum vertical clearance is 14 ft., it shall be provided over at least one lane. The remaining lanes may provide a lower minimum of 12’-6”.

Up to an additional 6 inches should be added to the vertical clearance for future resurfacing. Where the under roadway has previously been overlaid, some relief in the amount of vertical profile adjustment can be obtained by considering a reduction in the future overlay allowance. Existing pavement elevations near the bridge should be compared to the record plans and an existing thickness of overlay should be determined. This value should be compared to the normal 6 inch overlay allowance, and appropriate reduction in the future allowance be considered. Pavement overlay projects will require milling or removal of the existing overlay once the thickness approaches 6 inches.

If the existing vertical clearance is nonstandard, the need for improvement in the vertical clearances should be investigated during major rehabilitation (as defined in Section 19.1) or replacement projects involving the existing highways and structures.

2.5 Horizontal Clearances: Under-Bridge Features

2.5.1 Highway

Whenever possible, a substructure unit should be located to minimize the potential of vehicle impact as well as to lessen the effects of a hostile environment such as salt laden road spray and snow. The desired roadside horizontal clearances to fixed objects and recommended roadside clear areas shall be provided in accordance with the current AASHTO Roadside Design Guide and Chapter 10 of the Highway Design Manual. Piers located in narrow medians should be made parallel to the roadway whenever possible to allow for the possible future
widening of the under roadway. In wider medians, a graded earth berm treatment should be used in the pier area. (See Figure 2.2 for details.)

Figure 2.2
Schematic of a Median Berm
In urban areas, a minimum setback of 10 ft. from the face of curb to the face of any substructure unit should be provided. This corridor allows for sidewalk and utility placement independent of the roadway. Design speeds and class of highway may require greater setback distances. Refer to the *Highway Design Manual* for the recommended clear zone.

Stopping Sight Distance is also a critical design element. See Chapter 2 and Chapter 5 of the *Highway Design Manual* for more information.

### 2.5.2 Navigable Waterways

Waterways in New York State vary in type from intermittent streams to large lakes and rivers which can support navigation involved in interstate or foreign commerce. Actual navigation on these waterways may be nonexistent, strictly recreational (rafts/canoes) or mixed recreational and commercial. Jurisdiction for approval of work in these waterways may rest with the New York State Department of Environmental Conservation, the U. S. Coast Guard, U. S. Army Corps of Engineers, New York State Department of State, Adirondack Park Agency, Office of Parks and Recreation and Historic Preservation, United States Fish and Wildlife Service, National Marine Fisheries, National Park Service, or New York City Department of Environmental Protection.

In the early phases of project development, all projects involving a waterway should be evaluated by the NYS Department of Transportation’s Regional Environmental Coordinator. Procedures to be followed for locally administered projects can be found in Chapter 8 of the *Procedures for Locally Administered Federal-Aid Projects Manual (PLAFAP)*. Table BTA-1, Appendix 8-2 of the PLAFAP manual indicates the need to include a Coast Guard Jurisdiction Checklist. A copy of the Coast Guard Jurisdiction Checklist can be found in Appendix 2E of this manual.

Bridge projects that require fill and/or excavation in or adjacent to surface waters, including wetlands and special aquatic sites, or that impact state and federal rare, threatened or endangered species require early coordination with the Regional Environmental Contact. Regulatory permit conditions may influence the type of work performed. For example, replacing an existing single span with a precast reinforced concrete box requires prior approval from the Department of Environmental Conservation and the Corps of Engineers. For further information on permitting issues relating directly to the disturbances of surface waters and associated riparian areas, please refer to Chapter 4 of the *Environmental Procedures Manual* and Chapter 8 of the *Highway Design Manual*.

Waterways that support commercial navigational traffic typically require a formal Coast Guard Permit. The Coast Guard Compliance Unit of the Office of Structures will help determine the need, and normally prepare the paperwork, for a Coast Guard permit for state administered projects. For locally administered projects, it shall be the responsibility of the project sponsor or his designee to assemble the necessary permit documents and submit them to the appropriate Coast Guard District for their action. Access to the Coast Guard Bridge permit Application Guide is provided on the Internet through the Bridge Administration Web Page (http://www.uscg.mil/hq/cg5/cg551/)
Rivers that are designated for inclusion in the State or Federal Wild, Scenic and Recreational Rivers systems may have restrictions on the placement of piers within the banks of the river. Contact should be made with the appropriate Regional Environmental Coordinator prior to establishing span lengths.

The location of piers and pier protection systems for structures in the New York City/Long Island Region, the Lower Hudson River area, the Great Lakes Region, and the St. Lawrence River/Seaway should be handled on a case by case basis. Coordination with the appropriate Coast Guard District is required.

Early attention should be paid in determining the various types of permits needed and required supporting documentation. If identified too late, the permit process can become the critical path for a project.

The only waterway in New York State that has prescribed requirements for horizontal clearances is the New York State Barge Canal System. The following guidelines should be considered binding in designing new or replacement bridges over the canal system. Minor variances to the stated criteria may be granted on a case by case basis. Final decisions on variance requests will rest with the N.Y.S. Canal Corporation and N.Y.S. Dept. of Transportation.

1. Horizontal Clearance: Consideration should be given to hydraulic/hydrologic factors, canal curvature and local navigation conditions. Adverse site conditions which may merit an increase in horizontal clearance standards should be identified early in project development and all subsequent design reports. Adequate documentation must be provided (accident records, groundings, etc.) for considerations that will increase project cost due to required increases in the minimum stated criteria.

2. Access Trails: The lands adjacent to the Barge Canal System are being developed for recreational use by the public. Where appropriate, the placement of a new substructure shall accommodate an access trail beneath the structure. The elevation of this trail should be kept above ordinary high water whenever possible. Adequate vertical clearances shall also be provided (See Miscellaneous Vertical Clearance Criteria, Canal Trails). Minimum trail widths can be found in AASHTO’s Guide for the Development of Bicycle Facilities.

3. Channel: The edge of channel is defined as the outside edge of the theoretical bottom angle. Therefore, in a typical earth section of 75 ft., the channel is 75 ft. wide. Figures 2.3 and 2.4 show typical channel sections and minimum requirements for the location of a pier and pier protection system. All substructures, including cofferdams and fender systems, shall be placed a minimum of 5 ft. outside of channel limits. Encroachment upon earth or rock section channel limits will not be allowed. Please note that typical sections are subject to transition areas which will vary from the stated widths.
4. Pier Protection: Where barge traffic exists, all new or replacement substructures located in water depths exceeding 2 ft. shall have an impact attenuator system around the pier(s). A typical system shall consist of a permanent steel sheeting cofferdam with a tremie seal and filled with screened gravel (a heavy-duty galvanized gabion cover in river sections is required). The minimum gravel fill requirement is 5 ft. from face of pier to inside edge of sheeting. Steel sheeting will extend to 3 ft. above maximum navigable pool elevation. A rubber dock-fender system will be installed on the channel sides of sheeting and wrap around the face of the pier so that it extends at least 3 ft. beyond the point at which the sheeting is parallel with the pier. The centerline of the rubber dock fender shall be located 18 inches above normal pool elevation. Should normal pool elevation and maximum navigable elevation differ by more than 2 ft., a second fender shall be placed at an elevation of 18 inches above maximum navigation elevation. In all cases the minimum horizontal clearance from centerline of pier to edge of channel shall be 16 ft.

5. Where the potential for barge traffic exists, and construction of a pier does not require the use of a sheet piling cofferdam (i.e., areas that can be dewatered), any proposed bridge project shall consider using the same guidelines as above. This approach would allow the option of constructing an impact attenuator system at a future date and not encroach on channel limits. The minimum horizontal clearance of 16 ft. from centerline of pier to the edge of channel should be used.

6. Column pier configurations are not typically recommended for use on canal bridge projects. If column piers are chosen their use shall be limited to areas outside of the designated channel and shall be placed on a solid pier plinth that extends no less than 3 ft. above maximum navigable pool elevation. In instances where an impact attenuator system is not required at this time, a rubber dock fender system is necessary to protect both vessel and structure from damage. Therefore, all substructures located in water depths exceeding 2 ft. of depth (from normal pool) will have a rubber dock fender system installed. Installation requirements are the same here as they are for the impact attenuator system.

7. Rehabilitation Projects: Rubber dock fenders and/or an impact attenuator system for substructures located in the navigable portion of the canal should be considered on an individual basis and practicality of such an installation. It is also important to note that any rehabilitation work which will change the width of the superstructure, skew angle or alter existing horizontal and/or vertical clearances over the canal will require a U. S. Coast Guard bridge permit before construction may commence. When this occurs, navigation lights not previously required may become mandatory. Questions should be directed to the Office of Structures, Coast Guard Compliance Unit.

8. Permits: All bridges (permanent or temporary) constructed over the canal require a Section 9 bridge permit before construction may commence. The Office of Structures, Coast Guard Compliance Unit or the bridge owner or his designee is responsible for obtaining the bridge permit and coordinating with the U.S. Coast Guard.
2.5.2.1 Navigation Lights

The U.S. Coast Guard is the sole authority in determining the requirements for navigation lights. The Office of Structures, bridge owner, or the bridge owner’s designee is responsible for securing Coast Guard approval. Once approval of the lighting system is obtained, modifications cannot be made without additional Coast Guard review.

For fixed bridges required to have navigation lighting, each fascia of the superstructure shall indicate channel limits of passage through the structure for nighttime traffic. The edge of channel will be marked by a red channel margin light which shall show through a horizontal arc of 180 degrees. The center of channel will be marked by a green navigation light showing through a horizontal arc of 360 degrees. The focal plane (center of lens) of all navigational lights shall never be less than 6 inches below “Low Steel”. Navigation lights are not considered an encroachment on vertical clearances and should be placed over actual channel limits whenever possible.

Due to the variety of structure types and navigable conditions, some bridge locations may be exempted from displaying navigation lighting. The Office of Structures or the bridge owner or a designee will coordinate with the U. S. Coast Guard for proper lighting requirements.

2.5.2.2 Additional Navigation Aids

The U.S. Coast Guard is the sole authority in determining the requirements for numerous other aids to navigation. Ordinarily, they do not mandate such items but the possibility does exist. The Office of Structures, bridge owner, or the bridge owner’s designee is responsible for coordination with the Coast Guard. Possible items that may be required to be installed to aid navigation are retroreflective panels, pier lights, daymarks, radar reflectors, racons, painting of the bridge piers, and vertical clearance indicators.
Figure 2.3 – Typical Canal Channel Sections

Erie Canal - Waterford to Three Rivers
Oswego Canal - Three Rivers to Oswego

Champlain, Cayuga and Seneca Canals
Erie Canal - Three Rivers to Tonawanda

Figure 2.3 – Typical Canal Channel Sections
2.6.2 Bridge Rehabilitation

Existing highway bridges should be rehabilitated to carry the HS 20 live load, unless economically unjustified.

Bridges whose superstructures are completely replaced while retaining all or part of the substructure will be designed in accordance with NYSDOT LRFD Bridge Design Specifications. Existing substructures to remain shall not be upgraded solely to accommodate the higher live loads or LRFD Specifications.

Where the HS 20 loading cannot be economically justified, bridges should be rehabilitated to support an H 20 live load. In some cases, locally owned bridges or State-owned bridges carrying local roads may be rehabilitated to a lesser loading provided that heavy loads are anticipated to be rare. The minimum acceptable loading for a rehabilitated structure is H 15. Rehabilitation of any structure to a live loading less than HS 20 must be expressly approved by the Regional Director.

2.6.3 Temporary Bridges

Temporary structures carrying vehicular traffic shall be designed for an HL-93 or an HS 20 live load. While an HS 20 design live load is sufficient for all current legal loads, it is recognized that in some situations, the design live load for temporary structures should be the HL-93 design live load. HL-93 shall be used for the following types of projects:

- Interstate or equivalent highways with very high Average Daily Truck Traffic (ADTT). Very high ADTT can generally be taken to be over 10,000.
- Interstate or equivalent highways where it is anticipated that the temporary structures will be in service longer than one year.
- Other locations that may have unique situations in regard to very heavy industrial truck traffic, anticipated very heavy permit vehicles or access to railroad yards and port facilities.

It is also recognized that some locations may not require a HS 20 design live load for temporary structures. This would most often be the case for structures on parkways or in rural areas. However, locations in rural areas should be treated with caution since many low volume roads frequently carry heavy vehicles such as logging trucks, milk tankers and heavy farm machinery. Structures on parkways that will be in use over a winter season should also be treated with caution because snow removal equipment may approximate HS 20 loading.

All uses of temporary structures with design live load less than HS 20 need to receive approval from the Regional Structures Engineer. In certain circumstances, temporary structures designed for a live load less than HS 20 will require posting. In no case will approval be granted for a design live load less than H 15. In no case shall a temporary bridge on an NHS designated route be designed for less than HS 20.

Place Standard Note #7 from Section 17.3 on the plans for all projects containing temporary structures.
2.6.4 Pedestrian Bridges

All pedestrian bridges will be designed in accordance with NYSDOT LRFD Bridge Design Specifications and the AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges, December 2009. The owner may waive the fracture critical member requirements for design of tubular members. See Section 8.18 for additional guidance.

2.6.5 Railroad Bridges

All structures carrying railroads will be designed for Cooper E-80 loading (U.S. Units), unless noted otherwise.

2.7 Alignment, Profile and Superelevation

2.7.1 Horizontal Alignment

The alignment of a bridge can be controlled by a highway realignment project or be set by the standards that are to be used for a bridge only replacement project. Three factors normally dictate the chosen alignment: class of highway, design speed and traffic volume. The requirements of each individual project should be reviewed prior to establishing the necessary horizontal and vertical control standards. If possible, the highway designer should avoid placing spiral alignments and compound curve alignments on structures. Conventional highway treatments such as spiral alignments, reverse curves and superelevation banking transitions, when used on a bridge, can complicate the design, increase cost and make construction difficult.

Severely skewed alignments can cause uplift, seismic design and maintenance problems, and may result in a structure that is considerably longer than the existing structure.

2.7.2 Profile

When selecting project standards, such as maximum grades and stopping sight distances, the highway designer should avoid placing a sag curve at the bridge location. If this is not possible, the bridge designer should avoid placing the beam itself on a sag and fabricating it with negative camber. The placement of a level (0%) grade on the bridge should be avoided. If possible, steel beams shall use haunches for sag correction with the top and bottom flanges remaining parallel on a vertical tangent. (See Section 8.9.1, for further discussion on sag cambers for steel bridges.)

Prestressed units shall not be subjected to negative camber. The only corrective measure which can be used for adjacent units is to vary the thickness of the wearing surface. If this procedure cannot accommodate the geometry of the curve in a reasonable manner, the use of the adjacent slab or box units is not recommended. Prestressed I-beam or spread box/slab units can use varying haunches to accommodate some sag vertical curvature.
Appendix 2A
Bridge Roadway Width Tables

The tables included in the following two pages have been derived from Chapters V and VI of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2011.

Table R, “Minimum Clear Bridge Roadway Widths”, applies to new and replacement bridges on Rural and Urban Collectors and on Local Roads and Streets.

Table N, “Maximum Width of Traveled Way and Shoulder”, applies to new, replacement and rehabilitated bridges on Rural and Urban Collectors and on Local Roads and Streets.

Table X, “Minimum Clear Bridge Roadway Widths for Bridge Rehabilitations”, applies to certain bridge rehabilitations on local and collector roads, see Table 2-1.

See Section 2.3 and Table 2-1 for additional discussion on bridge roadway widths.

Additional clarifications:

1. All traffic is two-way.
2. The average daily traffic (ADT) in vehicles per day is always the design year traffic.
3. Refer to Project Development Manual (PDM) Appendix 5 for the design year for bridge work.
4. "Traveled way" is the portion of the roadway for the movement of vehicles, exclusive of shoulders.
### Table R

Minimum Clear Bridge Roadway Widths For New and Replacement Bridges (Local and Collector Roads)

<table>
<thead>
<tr>
<th>Design Volume (veh/day)</th>
<th>Minimum Roadway Width of Bridge&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under 400</td>
<td>Width of traveled way plus 2 ft. each side</td>
</tr>
<tr>
<td>400 - 1500</td>
<td>Width of traveled way plus 3 ft. each side</td>
</tr>
<tr>
<td>1500 – 2000</td>
<td>Width of traveled way plus 4 ft. each side&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>Over 2000</td>
<td>Approach roadway width&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

(Ref. AASHTO’s *A Policy on Geometric Design of Highways and Streets*, 2011, Table 6-6)

**Notes:**

<sup>a</sup> Where the approach roadway width (traveled way plus shoulders) is surfaced, that surface width should be carried across the structures.

<sup>b</sup> For bridges in excess of 100 ft. in length the minimum width of traveled way plus 3 ft. on each side is acceptable.
### Table N

**Maximum Width of Traveled Way and Shoulder**  
*(Local and Collector Roads)*

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Width of Traveled Way (ft)&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Design Volume (veh/day)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Under 400</td>
<td>400 to 1500</td>
<td>1500 to 2000</td>
<td>Over 2000</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>20</td>
<td>22</td>
<td>24</td>
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<td>65</td>
<td>22</td>
<td>22</td>
<td>24</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Width of Shoulder on Each Side of Road (ft)</td>
<td>All Speeds</td>
<td>2.0&lt;sup&gt;a&lt;/sup&gt;</td>
<td>5.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

(Ref. AASHTO’s *A Policy on Geometric Design of Highways and Streets*, 2011, Table 6-5)

**Notes:**

- <sup>a</sup> Per HDM Chapter 2, Exhibit 2-5, a 4.0 ft. minimum shoulder is required where roadside barrier is utilized.
- <sup>b</sup> exclusive of auxiliary lanes
### Table X

**Minimum Clear Bridge Roadway Widths For Bridge Rehabilitations**  
*(Local and Collector Roads – Two Lanes)*

<table>
<thead>
<tr>
<th>Design Volume (veh/day)</th>
<th>Minimum Clear Bridge Roadway Width (ft) a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under 400</td>
<td>22</td>
</tr>
<tr>
<td>400 to 1500</td>
<td>22</td>
</tr>
<tr>
<td>1500 to 2000</td>
<td>24</td>
</tr>
<tr>
<td>Over 2000</td>
<td>28</td>
</tr>
</tbody>
</table>

(Ref. AASHTO’s *A Policy on Geometric Design of Highways and Streets*, 2011, Table 6-7)

**Notes:**

a Clear width between curbs or railings, whichever is less, shall be equal to or greater than the approach traveled way width.

b Table X does not apply to structures with a total length greater than 100 ft. These structures should be analyzed individually by taking into consideration the clear width provided, safety, traffic volumes, remaining life of structure, design speed and other pertinent factors.
Appendix 2B
One-Lane Bridge Policy

A. Objective: This policy sets forth criteria used to determine where it would be acceptable to replace an existing one-lane bridge by another one-lane bridge.

When an existing one-lane bridge has deteriorated beyond a point where rehabilitation is appropriate, an evaluation shall be made to determine whether closure of the road or removal of the bridge is an acceptable solution. If that evaluation indicates that the bridge is deserving of replacement, then a determination must be made of the number of traffic lanes to be carried by the proposed bridge. The objective of this policy is to govern that decision.

B. Definitions:

Existing One-lane Bridge: One upon which two vehicles, traveling in the same or opposite direction, will not normally attempt to pass one another. The bridge may or may not be signed as a "One-lane Bridge". In the absence of recorded or observed experience, any bridge less than 16 ft. wide, curb to curb or rail to rail, shall be considered as a one-lane bridge. A ramp bridge, carrying traffic in only one direction, is not a one-lane bridge for the purpose of this definition.

Existing One-lane Road: One upon which two vehicles, traveling in the same or opposite direction, will pass one another only with care, usually by the slowing or stopping of one or both vehicles, and perhaps by the movement of one or both vehicles partially off the pavement surface, often accomplished at intermittent widenings which may occur naturally or which may be developed deliberately to facilitate such passing. In the absence of recorded or observed experience, any road measuring less than 16 ft. wide, edge to edge of roadway (including pavement plus graded shoulders), shall be considered as a one-lane road, unless it carries traffic in only one direction.

C. Requirements: An existing one-lane bridge may be replaced by another one-lane bridge if each of the following requirements are met:

1. The project shall meet the requirements of Table 5-11 of AASHTO’s A Policy on Geometric Design of Highways and Streets - 2011.

2. The current two-way ADT must be less than 350, and the predicted ADT for the 30th year after completion of the project must be less than 500.

3. The current and anticipated future operating speeds must be not greater than 40 mph.

4. An analysis of the three-year accident experience must reveal no more than one reported accident, with no accident being reported during that same period as being directly attributable to the narrowness of the existing one-lane bridge.
5. The replacement bridge and its approaches must be signed as a "One-lane Bridge" in accordance with the MUTCD.

6. Horizontal and vertical sight distances must be provided to allow approaching motorists to safely observe an opposing vehicle on the bridge or its far approaches.

D. **Desirable Conditions:** In addition to the above requirements, other relevant factors should be evaluated and considered before a final decision is made in favor of a bridge replacement to carry one-lane of traffic. Several of these factors are subjective in nature, and others may be very difficult to measure or identify with exactness. All should be treated as desirable conditions which should be met, but which are not absolute requirements. A list of such preferable conditions would include, but not be limited to, the following:

1. The local authorities should have no substantive objection to a one-lane bridge.

2. The existing two-way approach roadway should be one-lane wide and operating as a one-lane road (although this may be difficult to determine with confidence).

3. There should be no plans for the future improvement of the highway which would be expected to substantially alter existing operating conditions.

E. **Supporting Documentation:** Sufficient information should be supplied in the Scoping Phase so that the requirements and desirable conditions can be evaluated and a decision reached prior to the preparation of the Design Approval Document. If portions of that information are lacking, the final decision on the number of lanes may be made at a later time, but must, in any event, be resolved at or prior to Design Approval.

F. **Justification:** In order to achieve economics, one-lane bridge replacements shall be permitted when certain safety requirements have been met and certain conditions evaluated. Compared against the cost of a complete two-lane bridge, a minimum savings of 10 to 15 percent can be routinely expected, with appreciable greater savings when existing substructures can be retained.

G. **Conclusion:** When all requirements have been met, and when a final decision has been made to replace an existing one-lane bridge by another one-lane bridge, and when Design Approval, specifying that decision, has been obtained, the structural design normally shall produce plans for a bridge 14 ft. wide between railings, except that the replacement shall not be narrower than the existing one-lane bridge. Minor variations are permissible to account for the intricacies of particular structural components.
Further information on scoping and preliminary engineering is contained in the *Project Development Manual*.

### 3.3 Site Data

Once the project objectives are established, work begins on the final design and preparation of the *Plans, Specifications* and *Estimate* package. The PS&E package consists of two parts, the highway portion and the bridge portion. Information needed to establish parameters for the final design is provided to the bridge designer by the Region. The Region prepares and assembles this "Site Data" package or oversees its preparation by a consultant. The Regional Structures Engineer is responsible for verifying accuracy and completeness of the data.

The site data package consists of two parts:

- Bridge Data Sheet - Part 1 - Must be completed for all structures.
- Bridge Data Sheet - Part 2 - Waterway supplement, which must be completed for most structures over a waterway. (See Section 3.4.1)

These forms also require various supporting documentation (see Appendices 3A and 3B). An electronic version of the appendices is available on the Office of Structures web site.

Combine all required files into one PDF file before submitting. Hard copies of the site data package are optional. For designs to be progressed in the Office of Structures, the package will be reviewed by the Structures Design Bureau. For structures crossing water, the package will also be reviewed by the Hydraulic Engineering Unit. For consultant and Regional (in-house) bridge design projects, the Office of Structures Design Quality Assurance Bureau will be responsible for the review. For this type of project, see Appendix 3D for the required portion of the site data to be submitted.

With completion of these reviews and resolution of major comments, final design begins.

### 3.4 Hydraulics

#### 3.4.1 Hydraulic Design

Projects involving waterway crossings will generally require a hydraulic analysis unless it is clear that, because of the bridge's height, length, substructure configuration and construction method, there will be no significant effect on hydraulics. Consult the Office of Structures Hydraulic Engineering Unit for guidance on whether or not a hydraulic analysis is required. If an analysis is required, the necessary supporting documentation is outlined in Appendix 3B, Bridge Data Sheet-Part 2, Waterway Supplement. For definitions of ordinary high water, ordinary water and low water, see Section 17, Note 49.

Any work, permanent or temporary, that involves placement of constrictions or obstacles to flow within a channel or floodway (e.g., cofferdams, water diversion structures, causeways, etc.) will require the concurrence of the Office of Structures Hydraulic Engineering Unit or the Regional...
Hydraulics Engineer. Such obstructions have the potential to increase water surface elevations in violation of Federal flood insurance and control regulations, or to create dangerous scour potential. Since evaluation of these possibilities may at times require significant hydraulic analysis, any such proposed work should be brought to the attention of the Office of Structures Hydraulic Engineering Unit or the Regional Hydraulics Engineer as early as possible.

3.4.2 Hydraulic Table

For all projects where the hydraulic opening for the feature crossed is the controlling factor, a Hydraulic Table is required on the plans. The following table shall be shown:

<table>
<thead>
<tr>
<th>HYDRAULIC DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Area =</td>
</tr>
<tr>
<td>Recurrency Interval</td>
</tr>
<tr>
<td>Peak discharge</td>
</tr>
<tr>
<td>High Water Elevation @ Pt. of Max. Backwater</td>
</tr>
<tr>
<td>Avg. Velocity Thru Structure @ Design Flood =</td>
</tr>
<tr>
<td>Scour Analysis:</td>
</tr>
<tr>
<td>Q_100</td>
</tr>
<tr>
<td>Q_500</td>
</tr>
<tr>
<td>Begin Abutment</td>
</tr>
<tr>
<td>Pier</td>
</tr>
<tr>
<td>End Abutment</td>
</tr>
</tbody>
</table>

Scour depth is measured from minimum channel elevation.

Table 3-1
Hydraulic Data Table

For projects requiring the use of a temporary bridge to cross the waterway, the following notes should be completed and placed directly under the Hydraulic Table. Note 2 is to be used only when a hydraulic analysis permits.

1. The proposed temporary structure shall provide a minimum clear opening of _____ ft perpendicular to the flow with a minimum acceptable low beam elevation of _____.
polymer film sensors, buried float-out devices, etc. NCHRP Report #396 discusses these devices and gives their pros and cons.

The following scour monitoring devices have been used with some success at installations throughout New York State: Brisco scour monitor; Magnetic sliding collar scour monitor; Sonar scour monitor. They are described below:

1. Brisco Scour Monitor:

   It can be used in most situations, (but usually not in sandy channels); it is fairly simple with no high-tech components. If the channel consists mainly of sand, the rod will vibrate in the stream bottom so it will require a bottom plate to avoid vibrating into the sand. Sand, suspended sediment and ice could also get between the rod and the enclosing pipe, binding the rod to the pipe and inhibiting movement as it descends into the scour hole (even though this does not happen very often). It may require reinforcement or protection in streams or rivers carrying heavy ice or debris to avoid denting the outer pipe. It will not show any backfilling of the stream bottom. In a salt water environment the sleeve and the rod should be galvanized to avoid corrosion and the device should be checked for barnacles.

2. Magnetic Sliding Collar Scour Monitor (Described in NCHRP Report #397B):

   It is a simple, reliable scour monitor preferred by the New York Office of the USGS. The cables carrying the signal can be attached to the back side of the pier columns to avoid damage from ice or debris. It may be hard to install in streams with large boulders or rocks where excavating and installing the guide pipe may become a construction problem. It will not show any backfilling of the stream bottom. The collar and guide pipe will not corrode in a salt water environment nor interfere with magnets since they are stainless steel. The guide pipe must be driven to below the extent of possible scour. In salt water environment, the device should be checked for barnacles.

3. Sonar Scour Monitor (Described in NCHRP Report #397A):

   This scour monitor can be used in deep water more effectively than shallow water because if it is not always submerged, air bubbles trapped around the transducer head, will alter the reading given by the device. Fast flowing water may also introduce air bubbles, suspended sediment, debris or water turbulence at the transducer head which may alter the readings. It can show backfilling of the stream bottom. The reading during actual scour may be inaccurate due to the conditions mentioned previously. The head of the device requires regular maintenance and should be checked for barnacles, algae, or other obstacles if they exist in the vicinity. Since the sensor device in a sonar scour monitor is relatively inexpensive it may be worthwhile to use more than one sensor to measure scour at a foundation as a back up in case the first device becomes inoperative.

Further information on scour monitoring devices and guidance for their use may be obtained from the Office of Structures Hydraulic Engineering Unit.
3.4.5 Stream Crossing Permit Requirements

The Buffalo and New York City Districts of the US Army Corps of Engineers issued revised Regional Conditions for Nationwide Permits in March 2012. These are effective until March 18th, 2017. The Regional Conditions require a bridge, or an open bottom structure, or an embedded invert to create a natural streambed as well as maintaining the bank-full channel width through the structure. Consult the Regional Environmental Contact for complete requirements.

3.5 Structure Selection Process

3.5.1 Establishing Span Lengths

The geometric design policy outlined in Section 2 of this manual must be considered as well as the Design Report, site data package and correspondence to establish bridge span lengths. Design criteria for the lower roadway must also be considered.

The profiles and sections of the features being crossed as well as the crossing feature create two mathematical reference planes. The relationship of these planes to each other can be established by a NYSDOT computer program known as VERTCL (Shoulder Break and VERTical CLEarance Program). Other 2D and 3D COGO or CADD routines can also be used to determine the location of the minimum critical vertical clearance point and the maximum available beam depth. The resulting available beam depth, when used in conjunction with other project geometry, allows for the evaluation of various span lengths and configurations.

The shoulder break program also provides the limits of the bridge opening. This is known as the shoulder break area (see Figure 3.1). The overall bridge length is smaller than the shoulder break length. See the users’ manual for examples of how to use this program.

![Shoulder Break Area](image-url)
3.5.2 Bridge Type Based on Span Lengths

3.5.2.1 Span Lengths Less than 40 ft

See section 3.4.5 for stream crossing requirements. The various types of units and materials available for this span range include:

Structural Plate Pipes (aluminum and steel)

These units are available in various shapes and sizes. They can be used for shallow fills (~2 ft minimum), as well as deep fills. Their uses include pedestrian, bike and animal underpasses, railroad tunnels, and vehicular tunnels. They have been used as liners for masonry and concrete arches as well as other pipes. Submerged steel plate pipes should be rarely used for water crossings due to corrosion concerns and native streambed material requirements for structures with inverts.

Presently, the use of pipes for a bridge-size type of structure is limited to secondary roadways and low fill areas (<10'). Environmental and size constraints normally dictate whether to use steel or aluminum. For a discussion and details of this type of structure see the appropriate chapter of the latest NYSDOT LRFD Bridge Design Specifications and the latest manufacturer's catalogues. Steel and aluminum pipes are considered to be equal alternates.

Precast or Cast-In-Place Reinforced Concrete Structures

Reinforced concrete structures for culverts and short span bridges consist of four sided boxes, three sided frames and arch shapes. These structures are usually precast in segments and assembled in the field. The precast segments are usually designed by a professional engineer employed by the Contractor after the award of the contract. Four-sided boxes and prismatic three-sided frames are usually designed using the computer program ETCULVERT. Non-prismatic shaped and arch-shaped three sided structures are designed using other computer programs. For additional information on the structure types below, see Chapter 19 of the Highway Design Manual.

Four Sided Boxes have a maximum practical single-cell clear span of approximately 20 ft.

Three-Sided Frame Structures have a maximum practical clear span of approximately 40 ft. These units are supported on strip footings founded on rock or piles. A precast or cast-in-place, full-invert slab/footing unit can also be used.

Both three-sided structures and precast arches can be used for many of the same situations identified for the larger pipes. In order to obtain the necessary headroom for some cases, the units may be raised by supporting them on a pedestal wall. The use of multi-cell adjacent units to convey a waterway is discouraged due to the potential for blockage by debris catching and accumulating in the intermediate piers.
Deck Slabs or Deck/Girder Designs

Prestressed slab units, stress-laminated timber decks and concrete or timber decks with steel or timber girders cover this entire span range. Conventional reinforced concrete slabs, however, are inefficient for spans greater than 24 ft due to their excessive depth and heavy reinforcement.

Composite deck systems utilizing concrete with built-up steel girders or rolled sections can also be considered for spans in this range.

3.5.2.2 Spans Between 40 ft and 100 ft

Three-sided frames should only be used to a maximum span of about 40 ft. Arches and arch/frame hybrid structures can be used to a maximum practical span of about 60 ft. Use of three-sided structures are discouraged in the following situations:

1. Spans longer than 40 ft with low fill heights. With low fill heights the use of arch shapes may not be feasible due to hydraulic or horizontal or vertical clearance requirements. Three-sided frames (flat top slab) are inefficient because the member sizes increase dramatically for spans over 40 ft. Use of conventional bridges with prestressed slab unit or NEXT beam superstructures must be investigated.

2. Structures with stage construction, that have a skew greater than 10° and with a span longer than 40 ft. Arch shapes are very difficult to use in a stage construction with skew and three-sided frames are inefficient at these spans. Use of conventional bridges with prestressed slab units or NEXT beam superstructures must be investigated.

NEXT (NorthEast eXtreme Tee) Beam superstructures can be used for spans up to a maximum of 80 feet. NEXT beams are precast concrete double tee beams that come in two types. Type F is formed with a thin top slab that is used as a form for an additional cast-in-place superstructure slab. Type D is formed with a full structural deck section composite with the two concrete tee beams. A high strength closure pour is used between the NEXT beam Type D sections since there are no transverse tendons. See Section 9.3.3.

Adjacent prestressed concrete slab units can be used to a maximum span of about 60 ft. Prestressed concrete box beam units (adjacent and spread), concrete AASHTO I-beams, bulb-tee sections, etc., are used for spans up to and beyond 100 feet. Bulb-tees are usually preferred over concrete AASHTO I-beams. (except bridges with large cross slopes, where the smaller top flange width of concrete AASHTO I-beams may make them more attractive than the bulb-tee.) Deck/Girder systems using laminated timber beams have a maximum span of about 80 ft. Conventional composite design systems utilizing concrete decks and steel stringers can be used for the entire span range. At the lower end of the span range rolled beam sections would be used. Fabricated, welded plate girders would more likely be used at the upper end.
Special prefabricated bridge panels with precast concrete decks and composite steel beams can reach spans approaching 120 ft. They have the advantage of reduced field construction time and are therefore a preferred system for accelerated bridge projects.

### 3.5.2.3 Span Lengths Between 100 ft and 200 ft

Special modified prestressed concrete box beam units up to 4’-6” deep can span up to 120 ft. Prestressed concrete I-beams and bulb-tee beams ranging from 4’-7” to 6’-7” in depth can span up to approximately 150 ft. Bulb-tees are usually preferred over concrete I-beams. The designer should investigate the feasibility of transporting and erecting the beams, especially those with a span longer than 130 ft. Composite steel plate girder systems can easily and economically span this range. Single spans up to 220 ft have been used. Once the single span exceeds 165 ft, alternate multiple span arrangements should be considered. The cost of additional substructures must be compared to the greater superstructure cost.

### 3.5.2.4 Span Lengths Between 200 ft and 300 ft

Single spans in this range have fewer options. Plate girders or spliced concrete girders can be used at the lower end of this span range. For the majority of the cases only a thru or deck truss should be considered. Special designs utilizing arches, slant leg rigid frames, and concrete or steel box girders are also viable options. These types of special structures are used to address limited member depths, aesthetics and compatibility with site conditions. Constructability concerns and possible alternatives should be discussed in detail with the Region and the Office of Structures.

### 3.5.3 Multiple-Span Arrangements

For multiple-span bridges, a continuous design should be used whenever possible to eliminate deck joints. In the case of multiple-simple-span prestressed unit bridges, the deck slab should be made continuous for live load over the intermediate supports.

Span arrangements ranging from equal span viaduct type structures to proportionally increasing span ratios should be evaluated.

Continuous design using steel rolled beams or built-up plate girders takes into account the continuity over the interior support points. Based on the span arrangements and the span ratios, the largest span of a continuous layout can be equated to a smaller equivalent simple span. This reduces the required beam depth for the span. See the following table and LRFD Table 2.5.2.6.3-1 for guidelines. Poor continuous span ratios may result in uplift. Tie-down systems and anchored end spans are two means of addressing uplift.
<table>
<thead>
<tr>
<th>Number of Spans</th>
<th>Ratio of Spans</th>
<th>Equivalent Simple Span</th>
<th>Span to Depth Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Desired</td>
</tr>
<tr>
<td>2</td>
<td>1.0 : 1.0</td>
<td>*0.90 x 1.0 span</td>
<td>27.5</td>
</tr>
<tr>
<td>3</td>
<td>0.75 : 1.0 : 0.75</td>
<td>*0.85 x 1.0 span</td>
<td>27.5</td>
</tr>
<tr>
<td>4</td>
<td>0.80 : 1.0 : 1.0 : 0.80</td>
<td>*0.75 x 1.0 span</td>
<td>27.5</td>
</tr>
<tr>
<td>5</td>
<td>0.60 : 0.80 : 1.0 : 0.80 : 0.60</td>
<td>*0.60 x 1.0 span</td>
<td>27.5</td>
</tr>
</tbody>
</table>

* For span arrangements with less efficient ratios, the equivalent factor can be adjusted proportionally upward (i.e., 0.85 up to 0.90, 0.75 up to 0.85 and 0.60 up to 0.75).

** For ratios greater than 30, designers should consider LRFD LL deflection recommendations.

Table 3-2
Multiple Span Arrangement Ratios

3.5.4  Spans over 300 ft

Multiple-span arrangements in this range will involve balancing superstructure and substructure costs to achieve an optimum design. Site restrictions will often impact efficient substructure placement. Long multiple-span structures can utilize a variety of construction types and materials.

- **Steel**
  - Thru or deck trusses with girder approach spans
  - Trapezoidal box beams
  - Variable depth girders (‘I’ shaped beams and box girders)
  - Hybrid girders utilizing conventional steel for the web and high-performance steel for the flanges.
  - Cable-stayed girders or box beams
  - Deck or thru arches
  - Cable-stayed bridges
  - Suspension bridges

- **Concrete**
  - Segmental box designs
  - Cable-stayed trapezoidal boxes
  - Deck arches
  - Floating bridges/Pontoons
3.5.5 Selection Guidelines

A vast majority of New York's bridges are small single-span structures. The decision on what type of structure to use often depends on site limitations, foundation and geometric considerations.

The following guidelines may be used to determine what type of structure should be considered in the shorter span length ranges. These are for guidance only. Consideration should be given to the structure's relationship to the total project, geographical location, site accessibility and constructability:

A. For spans not exceeding 100 ft., prestressed adjacent box beam and slab units are always considered. If the structure is over a railroad or a stream prestressed concrete is more advantageous because of maintenance and inspection considerations. Elimination of form work for the deck slab minimizes work over the feature. Use of bulb-tees must be considered if utilities are present on the structure.

B. Frequently, prestressed concrete adjacent slab units or box beams will be chosen to satisfy critical profile and vertical clearance restrictions. Prestressed concrete structures with adjacent slabs or boxes require a 6 inch thick deck while a steel composite structure requires a 9 ½ inch deck and a 2 inch minimum haunch.

C. When skew angles over 50° are involved, adjacent prestressed concrete beam design should be chosen only after careful review, since conventional joint details and reinforcement become quite complicated, as do the size of the bearings and bridge seats. Bulb-tees or I-beams would be preferred at these sites. Approval of the D.C.E.S. is required for the use of adjacent prestressed beams with a skew over 50°.

D. For curved spans with midordinate corrections exceeding 1 foot, prestressed concrete adjacent box beams or slab units are seldom chosen because of the increased cost of the wider chord alignment and the complications that arise with regard to bridge railing anchorage and end transitions. Prestressed concrete bulb-tees, I-beams, or spread boxes are alternates worth considering.

E. Concrete bulb-tees, I-beams, or spread box beams should be considered if vertical clearance requirements can be satisfied.

F. At locations where either long piles or poor bearing capacity is anticipated, prestressed adjacent box or adjacent slab design has the disadvantage of having a heavier superstructure. Under these conditions a spread box, bulb-tee, or concrete I-beam with deck slab configuration might be considered to reduce the loads.

G. Prestressed concrete adjacent beam design is often chosen over steel beams when a structure must be opened to traffic quickly. This type of construction eliminates the need
for deck slab forming. It can also accommodate a temporary asphalt wearing surface if the time of the year prohibits placement of the concrete deck.

H. Where significant space must be provided for utilities, a spread system using steel girders, concrete I-beams, or bulb-tees is the preferred choice. Spread concrete box units can also accommodate some utilities.

I. Vertical curves are better handled with multigirder systems, since camber can be fabricated and controlled with greater accuracy. Adjacent prestressed units must accommodate any curve correction by placing a variable depth deck slab. This can result in considerable additional dead load necessitating a deeper beam. Negative cambers should not be used.

J. Adjacent prestressed concrete boxes or slabs are preferred over streams where ice and/or debris is a problem. The smooth underside of adjacent units reduces the potential for snagging.

K. Where either a steel or concrete superstructure is acceptable, the latest bid prices should be consulted.

3.6 Substructures

3.6.1 Substructure Location

When deciding where to locate substructures, the designer should identify all appropriate horizontal offsets, standards and requirements covered in Section 2. Using these constraints and the shoulder break length, the selection of either a single or multiple span arrangement, whichever is most appropriate, should be made. The available beam depth is factored in along with any special concerns such as:

- Sheeting requirements for staging and substructure construction. Cantilever sheeting design vs. tied-back sheeting vs. pile and lagging wall costs. Deep water cofferdam construction vs. shallower depths or causeway construction.

- Treatments such as high abutments with large reveal heights for form liner, masonry or brick treatments.

- Wetland encroachments - Longer spans to avoid wetlands will require additional beam depth. This can raise a profile and move the toe of slope out or require a retaining wall. Shorter spans may disturb more of the area and require additional wetland mitigation.

- Staging problems - Includes interference between the existing and new features, (e.g., substructures, beams, pier caps, pile driving - especially battered piles) as well as utilities that must remain in service.

- Misalignment with features crossed - Narrow highway medians may result in large skews for piers. For stream piers the normal direction of stream flow should be
considered to avoid the creation of eddies and turbulence. Desirable modifications of the skew for seismic reasons may be made difficult by site geometry.

- Utility Conflicts - Avoidance of utilities that would require costly relocations can further restrict the location of substructures. Pile driving and sheeting placement may be limited by overhead or underground interference.

- Integral Abutments - Must be located so that their exposed height is within the limits identified in Section 11.6.1.6.

3.6.2 Foundation Assessment

The "Site Data" package includes the substructure boring logs for the bridge and, sometimes, the highway. These logs should be evaluated with regard to:

- Location with respect to the new bridge - Do the boring locations allow the designer to confidently perform a preliminary foundation assessment?

- Consistency of the soil with respect to each log - Is the information in the different logs consistent enough to interpret rock elevations and soil types?

- Number of borings taken - Are there enough borings to extrapolate information? What if long walls are anticipated?

- Compatibility with the record plans of the existing bridge - Are rock elevations or pile lengths shown on the record plans consistent with the new boring logs?

- Location of borings with respect to the proposed substructure layout - Is there sufficient information to estimate pile lengths? Can sheeting be driven to required depths?

3.6.3 Foundation Selection

3.6.3.1 Water Crossings

The following criteria shall be applied to all structures crossing water.

- Unless founded on rock, all structures crossing water shall be supported on piles or have other positive protection to prevent scour of the substructure.

- The minimum length of pile to be considered is 10 ft.

- Cofferdams should be evaluated with regard to need, type, size, constructability and cost. Alternative types of construction such as causeways, caissons or drilled shafts should be considered and compared to conventional cofferdam costs.
The estimated maximum depth of scour should be used to determine overall structure stability. Piles should be socketed into rock if scour can affect their stability. Recommendations for details will be contained in the Foundation Design Report (FDR).

**3.6.3.2 Grade Separations**

Continuous structures will normally require unyielding foundations. Differential settlement is not acceptable since it may result in secondary stresses detrimental to the structure.

Where abutment or wingwall heights exceed 24 ft., alternate systems other than cantilevered, cast-in-place concrete wall systems should be considered. This is especially true in fill areas. Several modular wall systems are available which may provide a more economical system.

Coordination with the Office of Structures Foundation Unit and the Geotechnical Engineering Bureau is needed. Any assumptions made that are critical to the structure type and configuration should be verified. Additional boring requests or other subsurface investigations should be addressed to the Structures Foundation Unit of the Geotechnical Engineering Bureau.

**3.6.4 Orientation, Configuration and Details**

**3.6.4.1 Skew**

Orientation of the substructure units is greatly dependent upon the type of feature crossed. Whenever possible, the skew of the structure should be kept at 30° or less. Skews in excess of 30° can cause uplift problems, cracking of the concrete deck in the acute corners, and require larger bridge seats and pedestal bearing areas. Sharp acute corners should be avoided. Radial supports are preferred for curved structures. If possible, skews 10° or less should be eliminated, unless it creates problems with misalignment of the feature crossed.

**3.6.4.2 Water Crossings**

Whenever possible piers should be aligned with the stream flow to avoid the creation of eddies and turbulence which can increase scour. Skews of less than 10° can usually be avoided. Placement of abutments or piers should not result in pockets where water turbulence can increase potential for scour. The following guidelines for substructures need to be considered:

- Two piers close to each shore line may be more hydraulically efficient and economical to build than one deep water pier.

- Piers should be solid to a height of 3 ft. above maximum navigable elevation or 2 ft above the 100-year flood or flood of record, whichever is higher. If the remaining height of pier above the solid stem is 16 ft. or less, piers should be made completely solid. Use of a short column bent can result in shrinkage cracks in the columns.
• The upstream face of piers should be rounded or V-shaped to improve hydraulics. If ice and/or debris is a problem, the upstream face should be battered 15 degrees and armored with a steel angle to a point 3 ft. above design high water. This allows the ice to be broken and the debris or ice to ride up the pier face. At sites where medium or heavy drift is expected, this treatment should also be considered.

• Where wingwalls of an abutment are at or near the water's edge, wingwalls should be flared to improve the hydraulic entrance condition. If possible, the elevation at the end of the wingwall should be higher than design high water or, as a minimum, the ordinary high water.

• Wingwalls on the upstream side should be aligned to direct the flow through the bridge opening. For ease of construction, downstream wingwalls can be made mirror images.

3.6.4.3 General Details

U-wingwalls can be used when there is interference between the existing and the proposed structure or some other site restriction. They may also be used when a certain aesthetic effect is desired. Flared or in-line wingwalls are generally more cost effective.

When a wingwall length exceeds 25 ft., an alternate type of wingwall system should be investigated. Various types of sheeting or modular walls may prove to be more economical than a cast-in-place cantilever design.

Special details such as below ground cast-in-place or masonry block sills may be used to support architectural stone or brick facings. If form inserts are used to obtain an aesthetic appearance, the wall thickness must be increased by an amount equal to the relief of the insert.

Narrow roadway medians will generally require the alignment of a median pier to approximate the skew of the roadway. In wider medians, 60 ft or more, pier skews may be modified. In narrow medians where a pier will be subject to road spray, salt and snow build-up, a solid pier should be considered.

The use of small, isolated column piers is discouraged where the potential for impact by heavy trucks is possible. Where multicolumn piers are used, the potential for impact should be evaluated, and when deemed necessary, a crash-wall-type, partial-height plinth should be used. At railroad crossings, pier crash walls should be made parallel to the track and meet current AREMA specifications.

Substructure placement should also consider drainage requirements in the area around the substructure.
3.7 Work Zone Traffic Control

3.7.1 General

Before finalizing the type and configuration of the new structure, one final consideration must be evaluated. The method for temporary traffic control may become the overriding consideration in the selection of the preferred alternative as well as affect the cost and scope of the work. The temporary traffic control method for a project is generally decided in Project Design Phases I → IV. It is presented in the Design Approval Document. In order of preference typical methods for temporary traffic control used by NYSDOT are:

- Off-Site Detour
- Stage Construction
- Temporary on-site detour bridge.
- New alignment such that the existing bridge/roadway can be used to maintain traffic. This can include a partial or complete alignment shift.

Occasionally, the chosen method for temporary traffic control presents difficulties that require the method be revised during final design. Cost, constructability, safety, anticipated traffic volume, traffic capacity, and community impact are important criteria to be evaluated when comparing competing temporary traffic control methods. For example, stage construction presents construction difficulties that could result in a less desirable finished product and costly work zone traffic control. Night construction may also be considered as an optional method. Dialogue with the highway designer should be maintained through all design phases.

3.7.2 Off-Site Detour

Off-site detours often impose a cost on users in terms of the additional time and mileage needed to circumvent the construction site. Depending on the additional travel time imposed on the user, these costs can be negligible or very significant. This decision can also affect businesses, school bus operations, emergency services, etc.

Local residents and officials may prefer an off-site detour if it includes payment for a necessary roadway upgrade of the detour route or if special measures to mitigate the effects to local users/services can be arranged. An example of this is an adjacent fire district agreeing to temporarily provide service to an area separated from its normal fire service provider by bridge construction. From a construction perspective, an off-site detour presents the best opportunity for the contractor to do work efficiently. An off-site detour will almost always mean a simpler, less expensive, faster construction process that will likely yield a more durable final product (as compared to stage construction).

3.7.3 Stage Construction

Stage construction is appropriate when a suitable off-site detour is not available, or when the traffic volume is so large that off-site detouring is not practical. To accommodate high traffic
volumes, widened shoulder areas can be provided on the new structure to carry multiple lanes of traffic during staging operations. Stage construction can even be considered for existing bridges that have some form of nonredundant superstructure, e.g., thru girders, if additional supports or load carrying members can be added. Large profile changes between existing and proposed conditions can make staging difficult and require expensive sheeting schemes. The costs associated with stage construction are difficult to estimate in the early stages of a project. Until the actual staging details are developed, the cost of staging can only be indicated as an additional percentage of the estimated project cost.

The procedures and details proposed for staging should be thoroughly investigated to avoid orders-on-contract. Cost overruns associated with omissions or errors which should have been identified and addressed by additional site evaluations, record plans or subsurface investigation can be very costly.

Depending upon the complexity and extent of the stage construction, the additional cost can range from 10% to 30%.

**Guidelines for Stage Construction Details**

- The Region is responsible for determining minimum lane widths, shoulder widths and pedestrian access needs for each condition of staging. The Region should also identify any restrictions placed on any of the utilities.

- Show staging details for old and new pier(s) in each of the appropriate cross-sectional views.

- Use a dashed line pattern to identify limits of removal work in each stage. Limited removal work can also be identified as a crosshatched area, e.g., partial sidewalk removal.

- A dashed line should also be used to indicate temporary barrier and its location.

- Identify all temporary and permanent utilities in the appropriate stage.

- All transverse staging sections should include a true vertical and horizontal representation of the existing and new pier status at each stage. Any temporary supports or shoring details should also be included.

- All details should be drawn showing a true representation of the existing and proposed conditions with regard to their true elevation and horizontal relationship. When possible each preceding stage should be detailed below the previous. This downward projection should give a true representation of the location of the existing and proposed features with relationship to each other.

- Temporary cantilevered outrigger sidewalk details should be provided when the existing or proposed partial bridge section cannot accommodate both vehicle and pedestrian traffic within the dimensions proposed. This may be waived only if minimal pedestrian safety or mobility impacts will occur. Fencing may be used as the pedestrian fascia barrier in some cases.
• As a temporary condition (if alternate pedestrian routes and/or detours permit), all or a portion of the sidewalk area placement can be delayed as a means of providing room for vehicle lanes and shoulders. A temporary sidewalk width of at least 5 ft. is preferred. The absolute minimum sidewalk width is 3 ft. if a 5 ft-wide passing zone is provided every 200 ft. See the Highway Design Manual, Chapter 16 for further information.

• Temporary concrete barrier (each unit) on a bridge may be left unpinned, stiffened with box beam (Hwy. Std. Sheet 619-01, sheet 3 of 3), pinned to the deck or pinned and stiffened with box beam. The chosen option depends upon the design speed and the offset from the edge of the deck (drop off). Pinned units may be used for all design speeds and all offsets. Pinned and stiffened with box beam units may be used for all design speeds and all offsets but should be limited to locations of greatest need. Unpinned, un-stiffened units may only be used when the design speed is 45 mph or less and the offset from the edge of deck to the back of the barrier is a minimum of 1 foot. Unpinned units stiffened with box beam may be used when the design speed is more than 45 mph and when the offset from the edge is a minimum of 2 feet or when the design speed is 45 mph or less and the offset from the edge is a minimum of 1 foot.

Pinning of the barrier to the existing deck is acceptable provided the condition of the existing concrete deck is acceptable. Barriers should not be placed on large overhangs without checking the capacity of the deck slab. If possible, place the temporary barrier directly over a beam or on the deck slab between two beams. A temporary barrier stiffened with box beam is preferable to pinning on a new deck because of the potential of damage to the new deck.

• For further information on stage construction design, see Section 5.1.9.

3.7.4 On-Site Temporary Bridges

The on-site temporary bridge serves to keep the roadway facility operational during construction. The type of temporary structure to be used is greatly dependent upon site conditions. The alignment, profile, typical roadway section and the minimum span/opening will be specified by the State. The type of temporary structure may be left to the Contractor's option, or the Department may direct that a specific type be used. It will also be the Department's decision as to whether the temporary structure should be leased or purchased.

Options to consider when a temporary bridge is proposed include:

• The Traditional Manner:
  
  A temporary detour structure is specified by the designer. The Contractor is made responsible for the design and plans of the temporary bridge and must submit them to the Department for review and approval. Upon the completion of the project and the return of traffic to the permanent roadway corridor, the structure's salvage is the Contractor's responsibility. All detour structure costs are eligible for federal participation except they are limited here to a “rental-type” reimbursement.

• The Local Bridge Incentive Program:
A temporary detour structure is again specified by the designer, but additional consideration is given to the permanent disposition of the temporary detour structure. The required bridge design, specifications, plans and project development are tailored to both the temporary and permanent installation sites. All costs associated with this option are eligible for Federal participation. Even costs for removal of the existing local bridge at the site where the temporary structure is to be permanently placed are eligible. Additional guidelines can be obtained from FHWA.

- Innovative Designs:

Innovative design procedures can be introduced by either the designer or the Contractor. An example would be a roll-in or sliding technique. In this version, the temporary substructure and the new superstructure are located on a temporary alignment, parallel to the permanent corridor. The temporary substructure must be designed to carry the new bridge superstructure as well as being capable of handling the horizontal and vertical jacking forces. The permanent superstructure is then used as the temporary detour, while the old bridge is removed and the new permanent substructures are built. Once the new substructures and approach work are completed, traffic must be completely shut down for a short period of time for the jacking operation(s). The new superstructure is then moved to its final location.

Right-of-way, archeological, historical preservation, environmental and utility issues all have to be addressed as they relate to the placement of a temporary bridge. One or more of these factors may severely affect the use of a temporary bridge to maintain traffic at the site.

### 3.7.5 Alternative Alignments

Using an alternative alignment is a temporary traffic control approach most often used when it is necessary to eliminate an undesirable feature associated with the existing alignment, for example, a sharp curve. Due to high traffic volumes and certain traffic movements, it may be the most efficient way to handle traffic. The alternative alignment may either be a full or partial shift of the roadway's horizontal alignment. This approach can involve the same issues as mentioned for the on-site temporary bridge method; R.O.W., environmental, etc. In some cases the State may already own the R.O.W. adjacent to the existing bridge which will help reduce the cost. The cost and need for real estate acquisition can be a critical project concern. With an alternative alignment the project cost is also increased by the cost of roadway construction from the point of divergence to convergence with the existing alignment.

### 3.8 Alternate Designs

The process that has been outlined allows for an evaluation of options. By working through the process and applying site or design constraints, various alternatives are eliminated. This process of elimination and evaluation results in the most efficient and economical structure for most small and medium bridge projects.
For projects involving major structures (estimated cost $10 million or more) it may be more advantageous to determine the most cost efficient structure by competitive bidding. Alternate bridge types could be developed in the following manner:

- Value Engineering
- Conceptual Plans only
- Detailed Alternate Bridge Designs and Associated Plans

New York State includes a Value Engineering clause on all projects, whereby the Contractor may propose an alternate design for review and approval.

### 3.9 Hazardous Materials

The two hazardous materials most likely to be encountered in bridge replacement or rehabilitation projects are asbestos and lead-based paint. Asbestos has been used historically in several common bridge construction materials. Typical applications include bearing pads/sheet packing, joint filler, caulking, utility conduits, paint and other coatings. Removal/disturbance of asbestos-containing material is regulated under State and Federal Regulations. For guidance related to asbestos inspection and abatement design associated with bridge replacement or rehabilitation projects, refer to Chapter 1.C of the Environmental Procedures Manual.

The following items should be used to implement and maintain effective Health and Safety controls for lead removal as needed.

- Item 570.01, Lead Exposure Control Plan
- Item 570.02, Medical Testing
- Item 570.03, Personal-Exposure-Monitoring Sample Analysis
- Item 570.04, Decontamination Facilities

### 3.10 Environmental Considerations

#### 3.10.1 Introduction

When designing a bridge, a designer is required to fit a solution to a problem. A proposed work strategy of rehabilitation or replacement must adequately address a deteriorated or inadequate bridge or a newly proposed crossing. Solutions to these problems must be developed while considering certain criteria and parameters. The criteria can be found in laws or specifications governing loads, stresses or operational requirements. Some parameters are defined by site conditions, soil properties, seismic classifications, hydraulic considerations, etc. Other parameters are defined by social, economic or environmental issues. A designer attempts to develop a solution that economically addresses the conditions that define the problem while accommodating applicable criteria and parameters.

The Governor’s Environmental Initiative of 1998 re-emphasized the importance of assuring a project’s consideration of environmental parameters. These parameters are meant to assure the
maintenance of clean air and water and to advocate projects that “fit” in community settings, maintain historic significance, and accommodate recreational opportunities, where appropriate.

True support of the Governor’s initiative requires that the Department’s designers ascribe to the precepts of the initiative and integrate them into the project development and design processes. This must be done in a way that resulting products reflect the Department’s steadfast environmental ethic.

Every attempt should be made to identify environmental requirements and enhancements as early in the project development process as possible. This will allow an evaluation of the impacts they may have to project development, design and construction, the costs they impart to the project and the benefits that result in the final product. Obviously, decisions of scale, those that meaningfully impact project scope, cost or schedule, should be introduced in the scoping stage. “Details,” items that enhance appearance but do not have serious design, construction or cost implications, can be considered and introduced later in project development.

3.10.2 Types of Project Enhancements

There is a wide variety of enhancements available for bridge projects. For the purposes of this discussion, three classifications are identified as Structural, Aesthetic and Recreational.

1. Structural Enhancements

These are enhancements that affect the way a structure performs. The enhancement can be in the form of a structure type or layout which may not be optimum from an economic or a purely structural standpoint but is selected for superiority in combining sensitivity to community setting or historic ambiance and maintenance of acceptable operating standards. Examples are replacement trusses that bear extra fabrication and construction costs or haunched prestressed boxes that replicate “arch” construction but involve extra material and fabrication costs. These alternates may not be as structurally efficient as conventional designs, yet perform adequately and better replicate a desired era of construction. Designers should be cautious with the use of false structural facades, such as placing a truss in front of a girder span, to replicate a historical detail. Such treatments usually result in a bridge that is neither historical looking nor aesthetic. It is usually better to use an architecturally pleasing form that does not try to copy a historical detail.

The enhancement can also take the form of a preferred treatment, as in the use of innovative repair procedures or materials to preserve a bridge that is historic or contributes to the historic character of a setting. Examples of innovative repair procedures are the installation of an arch to reinforce an inadequate truss or lining a deteriorated masonry arch with a steel liner. Lightweight materials such as lightweight concrete or composite materials may allow the rehabilitation of bridges considered inadequate for typical design loadings.
2. Aesthetic Enhancements

Aesthetic enhancements affect the appearance of a structure and likely have economic impacts, but have minimal, if any, structural impacts. Treatments such as stone facing, form liners or concrete “stamping” are options that can be considered to enhance the appearance of a structure. Decorative bridge lighting along with decorative railing are often proposed to blend with community settings. Further information on aesthetics is available in Section 23.

3. Recreational Enhancements

Bicycle and pedestrian accommodations represent the majority of applications in this category. However, there are a variety of alternate applications to consider. Many NYSDOT bridges cross streams and rivers, some of which are prime fishing venues. Parking areas for anglers can be included as a project enhancement and, where safety considerations allow, wider bridges to accommodate anglers can be considered. Similar treatments can apply when vistas or other features that attract sightseers are encountered. Parking areas, overlooks or other accommodations such as sidewalks on the bridge can be considered.

While these enhancements can be considered in response to community sentiment or the habits of the public in general, the designer must place the safety of the traveling public as the number one priority in project development, design, construction and the eventual operation of the proposed facility.

3.10.3 When to Identify Enhancements

Enhancements should be identified early in the project development process to allow a reasonable evaluation of the costs and benefits associated with the enhancement. Obviously, parking areas or overlooks, or even facilities such as sidewalks or bikeways, may require the acquisition of right of way and should be considered in the project scoping stage. The selection of a structure type can have similar impacts. Structure type selection is generally done in the final design phase of a project. However, certain types of structures, such as trusses, cannot be constructed in stages and can impose maintenance of traffic issues that impact alignment selection, contract duration and right of way issues. These impacts should be evaluated early in the project process.

It is also important to look beyond an enhancement’s initial cost when determining its viability. The cost to maintain and inspect the facility should be considered and the responsibility for maintenance clearly defined. This is particularly important when facilities such as sidewalks continue off the bridge or when parking areas or overlooks are provided.
3.10.4 Summary

All NYSDOT projects should reflect the Department’s environmental ethic. It is the designer’s responsibility to integrate this ethic into a project’s design characteristics. The characteristics must be introduced at a point in the project development process that allows a meaningful evaluation of benefits and costs. Above all, the safety of the traveling public must remain the Department’s number one priority and any project enhancements must conform to that priority.

3.11 Final Preliminary Bridge Plan

3.11.1 General

The Final Bridge Preliminary Plan defines, by means of drawings, the concepts of the finished bridge. The following details are used to define the bridge and its approaches.

- Plan View
- Elevation View including a section of the feature being crossed
- Transverse Bridge Section including the type of pier, where appropriate
- Profiles
- Typical Section(s) of the Bridge Approach(s)
- Notes & Design Parameters

These details and drawings will become the first sheets of the detailed contract documents prepared for each new structure. (See Appendix 3F for a checklist.)

3.11.2 Format

The Bridge Preliminary Plan generally consists of at least two sheets. The following details appear on each sheet.

Sheet 1

- Plan view of the finished structure with the general features of the existing bridge shown dotted
- Full elevation view of the new structure
- Hydraulic Summary Table/Detour Opening Note
- Appropriate Highway Curve Data Table
- Preliminary Approval Signature Box

Sheet 2

Any continuation of the plan and elevation view should be broken at a point of support (pier or abutment) and continued a small distance past the support. The center line of support shall be the location of the match line.

- Full Transverse Section of the New Structure (showing a pier type where appropriate)
• All necessary profiles with banking details
• A detailed banking diagram of the bridge deck if it is in transition
• Construction and Traffic Staging Details - Start with the existing structure and continue showing the typical traffic and new construction limits in each stage. A finished bridge section does not have to be shown if it has been provided elsewhere on a preliminary bridge plan sheet. These sections should follow a true projection sequence from the top to the bottom of the sheet.
• Typical Approach Section showing the approach slab, railing transition details, and wingwall or retaining wall treatment where appropriate.
• Special elevation views to show the treatment of wingwalls, slopes, etc. (as required).

Preliminary Plan Sheet Notes
This is a listing of general and specific design notes as well as questions or proposals to the Region for review and comment. The preliminary cost estimate of the structure and preliminary foundation information (if available) are included. These notes are prepared on standard 8½ x 11 paper and included with the preliminary plan. (See Appendix 3G.)

3.12 Structure Justification Report

Each new and replacement structure requires the preparation of a Structure Justification Report. This report will also list principal dimensions and features of the existing and replacement structure. A sample Structure Justification Report form is provided in Appendix 3H. The report should include a discussion of waterway opening and alignment, skew, span length, number of spans, existing features, available structure depth, utility locations, horizontal clearances, material choice, aesthetic features, railing and constructability.

The structure type options that were considered prior to selecting the final structure type and configuration should also be discussed. If the final choice was based on an economic comparison, the supporting estimates should be provided. All Structure Justification Reports must contain a determination and statement whether or not the structure is considered innovative or unusual. See Section 20.2.2 for criteria and information on innovative and unusual bridges.

3.13 Hydraulic Justification Report

Each new and replacement structure over water requires the preparation of a Hydraulic Justification Report (HJR). The report is prepared or approved by the Hydraulic Engineering Unit (or Regional Hydraulic Engineer) prior to Preliminary Plan approval by the Deputy Chief Engineer, Structures.

Major rehabilitations may require an HJR if the waterway area is being affected. Contact the Hydraulic Engineering Unit or Regional Hydraulic Engineer to determine if an HJR is necessary.

The report contains a brief description of the stream crossing and watershed, and any existing ice or debris issues. A description of the existing structure and any hydraulic or scour deficiencies is provided. The discussion of the proposed structure includes type, material, alignment, dimensions and whether a temporary detour structure will be provided. The hydraulic
analysis is summarized and freeboard noted for both the Design flow ($Q_{50}$) and Basic flow ($Q_{100}$). Specific scour protection and hydraulic features are described.

When a hydraulic analysis is not required (i.e. bridge over gorge with abutments not near the waterway, or bridge over controlled section of NYS Barge Canal) the Hydraulic Engineering Unit prepares a statement summarizing the reasons an analysis is not needed, in lieu of the HJR.

### 3.14 Accelerated Bridge Construction

Some prefabricated bridge elements and systems may offer significant advantages over onsite cast-in-place construction. Advantages can include a reduction in field construction time, lower costs resulting due to off-site fabrication and standardized components, and improved safety because of reduced exposure time in the work zone. The controlled environment of off-site manufacturing helps ensure consistent quality of components for durability and long-term performance.

There are considerable rewards that can be attained with thorough planning, design and execution of accelerated bridge construction contracts. A detailed evaluation should be made to determine if a job should be accelerated. Consideration must be given to the applicability of the design, the contracting industry's abilities, project site conditions, costs and construction schedules. The Region and the Contractor must be committed to the accelerated schedule to ensure success. Shared responsibility, risk and control are needed for a successful project. Reduced schedules save money for all parties. When properly implemented, accelerated bridge construction can and should result in an inexpensive and durable bridge that meets schedule and budget requirements.

Additional information and guidance on selecting accelerated bridge construction for a project is available at [www.fhwa.dot.gov/bridge/prefab/framework.cfm](http://www.fhwa.dot.gov/bridge/prefab/framework.cfm).
This page intentionally left blank.
139. WEATHERING STEEL:

140. DO YOU HAVE ANY OBJECTION TO THE USE OF WEATHERING STEEL?

141. IF WEATHERING STEEL IS NOT DESIRED, PLEASE INDICATE COLOR YOU WISH STEEL TO BE PAINTED.

142. WILL YOU REQUIRE CLEANING OF THE SUBSTRUCTURE CONCRETE?

(Information notes - not for plans. Weathering steel will not be painted: use A572 instead.)
FOUNDATION NOTES

143. SCOUR ELEVATION @ABUT ___ FT. - ___ IN.
     @PIER ___ FT. - ___ IN.

144. CIP PILES ___ TON CAPACITY

145. STEEL H-PILES HP _____ x_____ WITH A ___ TON CAPACITY.

146. SPREAD FOOTINGS ON SOIL OR ROCK

     BEARING CAPACITY _________ ksi
     COEFF OF FRICTION _________
• To install stream bank protection (turbidity curtains, dikes, waterway diversions or other erosion and sediment control measures should be utilized, as appropriate, to limit turbidity at the substructure removal site or when performing bank stabilization activities. At times, a closed system may be utilized to confine turbidity without having to be dewatered. Those measures should be paid for under the appropriate Standard Specifications Section 209 pay items).

A temporary waterway diversion structure may be used for operations where stream flow needs to be relocated around a work site but the work site does not require dewatering. For example, placing stone fill along a slope, or excavating for and placing stone fill for keyways.

At the request of the designer, in consultation with the Regional Hydraulics Engineer, the Regional Landscape Architect and/or Environmental Engineer and permitting agency, the cofferdam item shall include additional streambank protection based upon installation timing and waterway flows. No less than a 2-year storm event potential shall be taken into account in designing temporary streambank protection.

When permanent sheeting is called for on the Contract Plans to protect against vessel impact, a cofferdam item shall be included to provide for the cost of de-watering and construction protection. The Contractor will have the option of installing separate cofferdam protection, or incorporating the permanent sheeting in the cofferdam system. If the latter option is chosen, the cofferdam item will cover all additional bracing required to strengthen the sheeting system, if required, and any work necessary to return the permanent sheeting to its required function after the cofferdam operation is complete. On occasion, anchor spuds are driven to facilitate construction of the cofferdam system and they are included in the price bid for the cofferdam.

When the sole purpose of the system is to protect dewatering and construction operations, the entire system will be covered under the cofferdam item.

Where stream diversion or other alternates are allowed as a substitution, the work shall be paid for at the price bid for the cofferdam at that location.

Cofferdams will be paid for on an each basis and shown as an enclosed area on the Contract Plans. This will expedite environmental reviews and permit procedures prior to PS&E. Use a separate serialized item number for each cofferdam to assure that varying field conditions are accounted for at each location. Cofferdams will be classified as either Type 1 or Type 2:

Type 1 ([Item 553.01nnnn]) cofferdams are required for a water depth exceeding 8 ft., measured from the bottom of excavation to anticipated Ordinary High Water or when special conditions warrant. They must be designed by a Professional Engineer licensed and registered to practice in New York State retained by the Contractor. The design is submitted to the Engineer-in-Charge for review by the DCES a minimum of twenty (20) working days prior to installation.

Type 2 ([Item 553.02nnnn]) cofferdams are limited to a maximum anticipated depth of 8 ft., measured from the bottom of excavation to anticipated Ordinary High Water. They must be designed by a Professional Engineer licensed and registered to practice in New York State retained by the Contractor. The Contractor submits to the Engineer-in-Charge, for review, the methods to be employed a minimum of ten (10) working days prior to installation. No design computations are required to be submitted.
The Designer shall select the appropriate cofferdam type based on anticipated water elevation and bottom of excavation. Stream integrity characteristics such as high velocity, ice pressure and scour potential may warrant a Type 1 cofferdam even if the depth is less than 8 ft.

For cost estimating purposes, assume that the cofferdam extends 2 ft. above Ordinary High Water and 3 ft. laterally beyond the limits of the proposed footing. See the appropriate section of this manual related to navigable water clearances for additional information. The Contractor shall determine the actual field limits required to satisfy conditions of the specification. (Such as not interfering with battered piles.)

When a cofferdam is used in conjunction with a tremie seal, the designer shall include Note 44 from Section 17.3 on the Contract Plans indicating the critical water elevation at which the system should be flooded in order to prevent the tremie seal from becoming buoyant. The Geotechnical Engineering Bureau will provide the flooding elevation. See Section 11 for additional information on the design of tremie seals.

The location(s) of sediment removal areas shall be indicated on the Contract Plans. The designer should obtain input from the Regional Landscape/Environmental Unit. See Section 17.3, Notes 40 – 44, for standard cofferdam notes to be placed on the contract plans. In some streams the Ordinary High Water elevation can be several feet higher than the Low Water elevation. This could lead to a cofferdam design of excessive size and cost that may be inappropriate for the majority of the construction operation. In consultation with the Regional Hydraulics Engineers it may be appropriate to designate by a note on the plans a more realistic elevation above which the system should be flooded to avoid overloading rather than expect the cofferdam to serve the most severe field condition as inferred in the specification.
When isotropic reinforcement is used the following details are followed:

- The reinforcement shall be two mats (one top and one bottom) comprised of #4 bars on 8 inches in transverse and longitudinal directions. A less desirable alternate of #5 bars on 12 inches may be used at regional request. The above spacings need to be adjusted when there is a skew as noted below.

- The top and bottom transverse and longitudinal reinforcement shall be staggered so that the top bars are centered between the bottom bar spacing, except in the end zones of decks with a skew angle over 30° where the spacing is reduced (see requirement below).

- The top and bottom mats of reinforcement shall have the same type of corrosion protection whether it is epoxy coated, galvanized or stainless steel.

- Top reinforcement cover is 3 inches for epoxy and galvanized bars and 2 inches for solid stainless steel and stainless steel clad; bottom reinforcement cover is 1½ inches for all types of bars.

- The longitudinal bars of both mats shall be placed on top of the transverse bars.

- For skew angles greater than 30° additional reinforcement shall be placed in the slab end zones at abutments and conventional deck joints. The additional reinforcement shall double the amount of the reinforcement in both mats and in both directions. This shall be done by cutting the spacing of the reinforcement in half. This additional reinforcement zone shall extend a distance from the end of the slab equal to the girder spacing.

- Fascia overhang reinforcement must be designed traditionally. An effort should be made to use #4 or #5 bars. The isotropic reinforcement extends to the fascia. Its area is included in the overhang design. Additional longitudinal reinforcement shall be placed in the overhang as shown on the BD sheets.

- Longitudinal bars are placed parallel to the girders. Transverse bars are placed parallel to the skew angle for angles up to and including 30°. On structures with curved girders the transverse bars shall be placed radially, maintaining the maximum spacing at the outside fascia girder. When reinforcement is placed on the skew, the perpendicular bar spacing shall be equal to the 8 inch nominal bar spacing times the cosine squared of the skew angle.

- For skew angles greater than 30° the transverse bars shall be placed normal to the girders.

- Additional longitudinal reinforcement in negative moment areas shall be provided as required in Article 6.10.1.7 of the NYSDOT LRFD Bridge Design Specifications.

- Welded splices are not permitted. Mechanical connectors are permitted only where stage construction requires their use due to a lack of adequate clearance for a lap splice.
5.1.5.2 Traditional Deck Slab Reinforcement

When the conditions of Section 5.1.5.1 for isotropic reinforcement cannot be satisfied, traditional deck slab reinforcement shall be used. When concrete deck slabs are designed with traditional reinforcement (nonisotropic) the design shall be in accordance with strength limit state design methods of the NYSDOT LRFD Bridge Design Specifications. Service limit states must also be checked in accordance with Article 5.7.3.4. When slabs are continuous over three or more supports, advantage shall be taken of the 0.80 continuity factor to reduce dead load and live load, simple-span bending moments. It is recommended that designers include stud shear connectors in the negative moment regions of continuous girder bridges as permitted by AASHTO. This may serve to lessen deck cracks by providing a more bonded section. Including longitudinal reinforcement in this region in section properties is permitted at the designer’s option.

Transverse reinforcement for a 9½-inch monolithic deck is given in the Traditional Deck Slab Reinforcement Table, Table 5-2. This transverse reinforcement is to be used in both the top and bottom mats. Design span is defined as the perpendicular distance between girders less one half the width of the one flange.

Ordinarily, girder spacing should not exceed 11’. Larger spacings are possible but should be used only in special cases with the approval of the D.C.E.S.

Longitudinal reinforcement in the top of the slab shall be #5 bars at 18 inches. Spacing of longitudinal reinforcement in the bottom of the slab shall be in accordance with Article 9.7.3 of the NYSDOT LRFD Bridge Design Specifications. The longitudinal bars shall be placed on top in both mats. No bars need be placed in the bottom of the slab directly over supporting members. Additional longitudinal reinforcement in negative moment areas shall be provided as required by Article 6.10.1.7 of the NYSDOT LRFD Bridge Design Specifications.

Both the top and bottom mat of reinforcement are epoxy coated, galvanized or stainless steel. Top reinforcement cover is 3 inches for epoxy and galvanized bars and 2 inches for solid stainless steel and stainless steel clad; bottom reinforcement cover is 1½ inches for all types of bars.

For skews up to and including 30° the transverse reinforcement shall be placed parallel to the skew. For skews over 30° the transverse reinforcement shall be placed normal to the girders. Skewed transverse reinforcement shall be detailed with the spacing perpendicular to the bars; not parallel to the girders and the spacing should be as given in Table 5-2 times the cosine squared of the skew angle. This intent needs to be detailed clearly with the use of arrowheads perpendicular to the bars.
If the structure is designed by a Consultant, a task for checking the fascia girder for the actual construction loads calculated by the Contractor’s engineer should be included in the Construction Support Services Agreement.

When girder depths exceed 4 ft., another potential overhang related problem can develop. If the brace supporting the overhang form is brought back to bear against the web of the fascia girder above the bottom flange, the horizontal force from the brace can buckle the web. Place Note 54 from Section 17.3 on the plans in this situation.

In cases with large overhangs, shallow beams and long spans the designer may choose to accommodate the temporary construction loads by placing additional permanent bracing (lateral system and more diaphragms) in the fascia bay.

Reinforcement in the top of the structural deck slab in overhang regions needs to be designed to resist wheel loads on the overhang as well as impact loads on the railing or barrier. Requirements for overhang reinforcement are found in the NYSDOT LRFD Bridge Design Specifications.

See the latest BD sheet for slab edge reinforcement specific to barrier/railing system.

Top transverse deck slab bars require hooks at each fascia of the slab to provide proper development. When the transverse width is less than 30 ft. use one bar with hooks at each end. When the transverse width is greater than 30 ft. and less than 115 ft., use two unequal length bars, each with a hook on one end (see Section 15.4.1). When the transverse width is greater than 115 ft., provide a long straight bar in the center lapped to shorter bars with hooks on one end. Bottom transverse deck slab bars do not require hooks, and can be straight bars up to 60 ft.

![Figure 5.1 Overhang Form Bracing](image-url)
5.1.6 Haunches

Haunches are to be provided on all bridges with steel girders or prestressed concrete I-beams, bulb-tees or spread box beams. The purpose of the haunch is to provide a means for final adjustment of the deck slab elevation to match the designed roadway profile and cross slope. The haunch allows this adjustment to correct construction and fabrication variations without having the top flange of the girder project into the structural deck.

The calculated depth of haunch shall have a 2-inch minimum concrete thickness as measured at the centerline of beam from the top of beam to the bottom of slab. A deeper minimum is required when the top flange equals or exceeds 1'-4" in width to allow for roadway cross slope. The total haunch depth shown on the plans shall include the thickness of the top flange for fabricated steel girders.

At all splice locations for steel girders, the top flange splice plates will reduce the haunch depth. The designer shall verify that a negative haunch will not occur at the splice location. If a negative haunch does occur, the haunch shall be increased to eliminate the negative haunch (such that the distance between the theoretical bottom of slab and the top of the top flange top splice plate will be greater than zero). It is not necessary to provide the full 2-inch minimum haunch at the splice location. Dimension “E” in the haunch table will still be dimensioned from the theoretical bottom of slab elevation to the top of the top flange.

Details of haunches for steel girders are shown in the current BD sheets. For simple span bridges, the calculated depth of the haunch at the centerline of bearings shall be the minimum depth, plus the difference in thickness between the maximum and minimum top flange plates plus increases to account for cross slope and horizontal curvature when straight girders are used.

The haunch shall be reinforced when the depth of the concrete portion of the haunch exceeds 4 inches. Only the section along the girder where the concrete portion of the haunch exceeds 4 inches requires reinforcement in the haunch. See haunch reinforcement details in the current BD sheets.

Steel beams shall have minimum 6-inch stud shear connectors for haunches up to 4 inches in depth. Haunches on steel beams greater than 4 inches shall comply with NYSDOT LRFD Bridge Design Specifications, Article 6.10.10.1.4 or NYSDOT Standard Specifications for Highway Bridges, Article 10.38.2.3. Haunches on fascia beams of multispans shall be set so that the top of the webs of fascia beams in adjacent spans line up.

Do not label the haunch as 2 inches minimum. Label it only as “haunch”. The Contractor shall provide the completed Haunch Table to the EIC.

A haunch table shall be shown on the plans to assist in construction. For spans 65 ft. and under, the haunch table should be done for span quarter points. For spans over 65 ft., the haunch table shall give elevations at span tenth points, but not to exceed a spacing of 20 ft. Bridges with curved girders should have a haunch table with the elevations given at the diaphragm lines. The predicted concrete slab and superimposed dead load deflections are shown at these points.
Haunch tables shall always be computed considering stage construction and the assumed pouring sequence for continuous span structures. In addition, the computations shall consider differing dead loads applied to fascia and interior girders, especially with construction loads such as barriers. Field measurements are then taken at the same points shown in the haunch table. The actual haunches are then determined from this information. An example of a partial haunch table to be shown on the contract plans is given in Figure 5.2. A full haunch table is shown in the current BD sheets.

Bridges with complex geometry, haunched girders, and significant superelevation transitions should have a Design Haunch Depth Table, Table 5-3, providing the “design” haunch depth at the supports.

<table>
<thead>
<tr>
<th>S. ABUT. (ft)</th>
<th>PIER (ft)</th>
<th>N. ABUT. (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>0.305</td>
<td>0.430</td>
</tr>
<tr>
<td>G2</td>
<td>0.430</td>
<td>0.410</td>
</tr>
<tr>
<td>G3</td>
<td>0.322</td>
<td>0.459</td>
</tr>
<tr>
<td>G4</td>
<td>0.351</td>
<td>0.440</td>
</tr>
<tr>
<td>G5</td>
<td>0.285</td>
<td>0.686</td>
</tr>
</tbody>
</table>

Table 5-3
Design Haunch Depth Table

Except for the fascia side of the fascia girder, haunches shall not extend beyond the flange of the beam. In the past, some bridges were constructed with a haunch detail as shown in Figure 5.3. This detail was commonly employed when removable wooden forms were used for a concrete deck, since it enabled precut material to be used in the forming operation. The cause of cracking and eventual loosening of portions of this unreinforced concrete has been attributed to forces generated by corrosion on the vertical edges of the flanges. Cracking occurs at the top corner of a girder flange and progresses upward and outward through the concrete to the vertical haunch face. This loosened concrete is then prone to separate and fall from the structure.
All structural plans for bridges with concrete decks supported on steel girders, Bulb-Tee and AASHTO I-beams or floor systems shall include Note 55 from Section 17.3 on the plans, in association with the standard haunch detail.
5.1.7 Forming

Current specifications permit the forming of structural slabs using removable forms, permanent corrugated metal forms and prestressed concrete form units. If one or more options are not permitted on a particular job, the remaining permitted option(s) shall be listed on the Plans.

Individual railroads and the Thruway Authority may not permit the use of permanent corrugated metal forms above their facilities. Use of these forms must be approved by the railroad or agency involved.

When permanent corrugated metal forms are specified, a small detail shall be included in the plans indicating the presence of corrugations on the bottom of the structural slab and that the bottom row of reinforcement shall be placed with 1½ inch cover above the crest of the corrugations. Such a detail is shown on the current BD sheet titled, “Superstructure Slab Optional Forming Systems”.

The additional weight of permanent corrugated metal forms with the corrugations filled with Styrofoam shall be taken as 4 lb/ft². The stringers or girders shall be designed for this additional weight.

No additional weight shall be considered when using precast prestressed concrete form units. Their details are shown on the current BD Sheet titled “Superstructure Slab Optional Forming Systems”.

5.1.8 Continuous Structure Deck Slab Placements

Deck slabs on continuous structures are subject to transverse cracking during construction. The cracking can be found in negative moment areas where the concrete has already set and the placement has continued into positive moment areas. The cracking is caused by additional deflection of the beams when the concrete in the remaining positive moment area is placed.

The frequency of the cracking can be reduced if proper construction methods are used and strict control over the timing and sequencing of the deck placement operation is exercised.

Deflection cracks usually occur for one of the following three reasons:

1. Slow Rate of Placement

   When the concreting progresses slowly, some of the already-placed concrete may take its initial set prior to full deflection of the steel. As additional concrete is placed during the same placement operation, cracks will occur in the concrete that has already set. To prevent this from happening, either the duration of the placement should be decreased or the time to initial set of the concrete should be lengthened.

   The time required to complete a placement depends on its size and complexity, concrete delivery logistics, available rate of supply, and Contractor efficiency. Responsibility for attaining the highest practical rate of placement, and the shortest possible placement time at any particular project location rests with the Contractor.
The setting time for concrete can vary widely. It depends on many factors, such as mix design, use of admixtures, and atmospheric conditions. Retarding admixtures are intended to lengthen the time to initial set of the concrete.

To avoid cracking caused by the occurrence of initial set prior to completion of the placement, the duration of each placement shall be kept to a minimum, and no concrete shall be placed without sufficient retarding admixture to assure that initial set will not occur prior to completion of the placement.

2. Incorrect Loading Sequence

Many continuous structures require a total volume of concrete which is too large to be placed prior to the occurrence of initial set at some point in the deck. In cases where the total volume of concrete exceeds 360 cubic yds., the total concrete volume must be divided into a sequence of placements. Although this method lessens the probability of cracking related to initial set, cracking may still occur if the sequence of applying concrete loads is incorrect.

When a sequence of placements is used, the location of the first placements is vital. Concrete cannot be placed in negative moment areas first because subsequent placements will impose tensile stresses on this concrete, resulting in transverse cracking.

Further, if any placement results in the upward deflection of concrete previously placed in a positive moment area, the concrete in that area may crack. Consequently, it is necessary to place concrete in each positive moment area during the initial placement. This may be difficult if the volume of concrete required to fully place all positive moment areas is very large. Therefore, either the concrete volume or the placement rate must be modified. In some cases, the placement rate can be increased by the use of an additional finishing machine. The volume can be decreased by adding some of the positive area to the negative area, to improve the balance between placement volumes. As a last resort, the positive moment area placement can be divided and placed in separate placements, but, in such a way as to minimize the potential for cracking.

On skewed structures, the concrete shall be placed parallel to the skew angle. Loading the structure in this manner equalizes the steel deflections. For skew up to 35° the finishing machine should parallel the skew angle if there is no superelevation transition. For skew over 35° the finishing machine should be operated at 35° if there is no superelevation transition. If there is a superelevation transition then the finishing machine may be operated at no skew. The combination of skew and superelevation transition makes operation of the finishing machine at a skew difficult. See Notes 162 and 165 in Section 17.3.
3. Early Application of Loads

Immediately after initial set, deck concrete has little or no compressive (or tensile) strength. At this time, minor loads or deflections can cause serious cracking in the new deck. However, compressive strength increases rapidly to a point where moderate stresses (due to loads or deflections) can be resisted. For this reason, new deck concrete that will have any measurable effect on recently placed concrete shall not be placed until adequate early strength may be assumed. A waiting period of 72 acceptable curing hours, measured from completion of previous placement to start of next placement, is considered sufficient.

Instructions to Designer

The Contract Plans for every continuous steel structure where the total volume of deck concrete exceeds 360 cubic yds. shall include a deck placement sequence. Two placements shall be shown, except for structures comprised of unusually long or numerous spans which will require special treatment. Continuous spans with the total volume of deck concrete less than 360 cubic yds. may be placed in a single placement. A placement is defined as the total volume of concrete placed during a continuous work period. It may result from one placement operation in one area, one placement operation in several areas sequentially; or two or more placement operations in several areas simultaneously.

“Placement 1” shall include the positive moment areas (except as noted below) in all spans. “Placement 2” shall include all the negative moment areas. Construction joint locations shall be shown in the deck placement sequence details. These joints shall be located at or near points of dead load contraflexure (see Figure 5.4). In addition to delineating the placements, this information may be helpful to the Engineer should it be necessary to terminate the Contractor's deck placement operation due to unforeseen circumstances.

When the total volume of deck concrete to be placed during Placement 1 exceeds 360 cubic yds., two simultaneous placement operations shall be used. The designer should specify this by including a note in the deck placement sequence details.

At a few project locations, the available supply of concrete will not support the use of two placement operations. The designer must determine that sufficient concrete is available before specifying the use of two placement operations on the plans. The determination may be obtained by asking the Regional Materials Engineer. When the use of two placement operations is impractical, or when special treatment is in order due to unusual length or number of spans, Placement 1 may be divided into Placement 1A and Placement 1B. The plans should show Placement 1A to be comprised of end span positive moment areas only. A note shall be added stating that the segments labeled Placement 1B shall not be placed until a minimum of 72 acceptable curing hours after the completion of Placement 1A. This procedure confines the risk of deflection cracking to end span areas near the points of contraflexure only.
In certain instances, where the concrete volume is very large, the designer may elect to modify the Placement 1 segment lengths such that Placement 2 includes some positive moment area. This may be accomplished in either of two ways:

1. Move the location of construction joints up to 5% of the span length into the positive moment area (see Figure 5.5).
2. Introduce an additional construction joint within 20% of the span length from the abutment, in end span positive moment areas only (See Figure 5.6).

Either, or both, of these methods will reduce the duration of Placement 1. The total placement volumes of Placement 1 and Placement 2 will also become more equal, thus facilitating the Contractor's operations.

Construction joints shall be shown parallel to the skew angle, regardless of the orientation of the reinforcement.

Longitudinal construction joints shall not be used to reduce placement size.

The direction of placement shall be shown on the plans. The direction of placement shall preferably be uphill and always uphill when the true (not theoretical) grade exceeds 3%. Also applies to simple spans.

Camber-deflection data shown on the plans shall be based on the placement sequence shown on the plans. The loads imposed by Placement 1 will be supported by the noncomposite beam section, and partial deflections shall be computed accordingly. The loads imposed by Placement 2 will be supported by the composite beam section, \( n=27 \) (assuming a modular ratio=3n), in positive moment areas covered by Placement 1, and by the noncomposite section in negative moment areas. Partial deflections from the various placements included in Placement 2 shall be computed, assuming simultaneous placement.

The Designer shall check for uplift at bearings. Where uplift is anticipated, a load vector shall be shown at the free end bearing line (usually an abutment) towards which Placement 1 is progressed, It shall be accompanied by a note reading:

*Provide uplift restraint equal to _______ kips/Bearing. The cost of this restraint shall be included in the price for the appropriate concrete deck item. (See Figure 5.4)*

See section 17 for additional slab placement notes to be shown on the contract plans.
When penetrating sealers are applied to concrete they penetrate the surface, chemically bond to the concrete, and prevent water and chlorides from entering. Because the sealers bond below, not on, the surface, they cannot be abraded away easily. Good surface preparation prior to applying the sealer is essential to achieve the desired maximum penetration. Contaminants must be totally removed and the surface allowed to dry. When the surface is properly prepared, a five-year service life of the sealer can be achieved.

Penetrating sealer should be applied to all new and concrete overlaid bridge decks, to protect the surface from scaling due to early exposure to deicing chemicals. This is recommended because the majority of bridge deck and overlay placements occur late in the construction season thereby making them prone to early exposure to deicing chemicals, and because the concrete, regardless of age, will receive some benefit from the application of a sealer.

Parapets and barriers allow the use of curing compounds. Because curing compounds prevent penetration of sealers into concrete, sealers should not be used unless the membrane cured surfaces are allowed to cure and then are sandblasted.

Usage Guidelines

New Bridge Decks: To protect new, “green,” concrete from scaling, a penetrating type sealer (which does not contain an aqueous solvent/carrier) shall be applied to the top surface of all newly constructed bridge decks, bridge deck rehabilitations, sidewalks and concrete approach slabs, in accordance with Item 559.1896__18.

Existing Bridge Decks: Application of sealers to the top surface of existing bridge decks shall be in accordance with Item 559.1796__18. Place Note 101 of Section 17.3 on the plans.

Existing decks with good quality concrete and epoxy-coated reinforcing steel should generally not be considered for sealer application. Decks with such protection are usually only sealed as a remediation for construction, material, or other problems, such as hairline cracks or an open surface. The use of sealers in these situations should be decided on a case by case basis, in consultation with the Regional Materials Engineer. Sealers are not a viable alternative for protecting improperly air entrained concrete.

Sealers may be used on existing decks with uncoated steel reinforcing bars or less than 3 inches of cover to slow down any existing corrosion and postpone more costly repairs. Sealers do not stop corrosion, but the corrosion process is slowed by reducing intrusion of water and chlorides.

5.1.11 Aggregate Requirements for Concrete Decks and Approach Slabs

To provide adequate wet-weather friction, a concrete wearing surface must have sufficient macrotexture and microtexture. Macrotexture is provided by manipulating the concrete surface during or after construction (e.g., Astroturf drag and saw-cut grooving). Microtexture is the texture on the surface of the exposed aggregate particles.

As concrete decks and approach slabs are subjected to traffic loads the cement paste abrades away, reducing macrotexture. If wear becomes excessive before the slab reaches the end of its structural life, macrotexture can be improved through relatively inexpensive treatments such as saw-cut grooving.
Traffic also reduces the microtexture of the concrete surface by “polishing” the exposed aggregate surfaces. The hardness of the aggregate determines its resistance to polishing under traffic. Once compromised, microtexture cannot be restored through inexpensive treatments, and in most cases the only remedy is to overlay the surface. Therefore, it is essential that appropriate aggregate be used during initial construction of the slab. Since harder aggregates are more expensive and of limited supply, it is not appropriate to simply use the hardest aggregates in every situation.

The required aggregate hardness depends on the traffic volume and site geometry. High traffic volume (AADT), braking traffic, or turning traffic will polish aggregate more quickly than straight rolling traffic. The *NYSDOT Standard Specifications for Construction and Materials* contains requirements for four types of friction aggregate; Types 1, 2, 3, and 9. Each type is intended for use under specific traffic and geometric conditions. The aggregate requirements are in addition to all surface texture requirements such as turf drag or saw-cut grooving. Increasing the macrotexture from these treatments does not compensate for using inappropriate aggregate.

If any portion of the bridge deck or approach slabs meets any one of the criteria listed below, use the Aggregate Type Selection table (Table 5-4) to determine the appropriate aggregate. If the bridge deck or approach slabs do not meet any of the criteria, use Type 9 aggregate. The designer shall specify only one type of aggregate for each bridge and its approach slabs by selecting the appropriate pay item.

- The deck or approach slabs are ≤ 500 ft. before a stop sign, traffic signal, or yield sign, as measured from the stop bar or yield sign.
- The deck or approach slabs are in a location where vehicles regularly queue regardless of distance from a traffic control device.
- The deck or approach slabs are ≤ 500 ft. from the point of curvature of a curve requiring reduced speed limit, chevrons, advisory speed, advisory curve or other warning signs or signals as defined in the *Manual of Uniform Traffic Control Devices (MUTCD)*.
- The deck or approach slab is ≤ 500 ft. before an exit ramp, as measured from the initiation of the taper for the deceleration lane.
- The deck or approach slab is ≤ 500 ft. after an entrance ramp, as measured from the terminus of the taper for the acceleration lane.
- The deck or approach slab is located on an entrance or exit ramp.
- Any location where the ratio of wet weather accidents to total accidents is greater than the state average for the same facility type.
Bridge Decks

<table>
<thead>
<tr>
<th>Traffic</th>
<th>Location</th>
<th>Aggregate Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Volume ¹</td>
<td>Downstate ²</td>
<td>Type 1</td>
</tr>
<tr>
<td></td>
<td>Upstate ²</td>
<td>Type 2</td>
</tr>
<tr>
<td>Low Volume ¹</td>
<td>All</td>
<td>Type 3</td>
</tr>
</tbody>
</table>

¹"High Volume" refers to single lane bridges with design year AADT over 4000, 2 or 3 lane bridges with two-way design year AADT over 8,000, or bridges with 4 or more lanes with two-way design year AADT over 13,000. "Low Volume" refers to bridges not meeting the aforementioned criteria.

² The City of New York and the surrounding counties of Dutchess, Nassau, Orange, Putnam, Rockland, Suffolk, and Westchester are referred to as “Downstate”. All other areas are referred to as “Upstate”.

Table 5-4
Aggregate Type Selection

5.2 Jointless Decks

Leaking deck joint systems are one of the most significant causes of bridge deterioration. Although deck joints function properly at first, over time they become less reliable. Therefore, current DOT policy is to eliminate deck joints whenever possible.

5.2.1 Jointless Decks over Conventional Abutments

A jointless bridge deck over a conventional abutment is one where the bridge superstructure is supported on conventional bearings and the deck slab is continuous with the approach slab over the abutment backwall with only a construction joint separating the two slabs. All expansion and contraction of the superstructure is, therefore, transmitted into horizontal movement through the expansion bearings and the sliding of the approach slab over the abutment backwall. A bond breaker is used over the backwall and over the approach subgrade at the expansion end. The deck slab should not haunch down to an end diaphragm at a jointless abutment. Haunching the deck slab to an end diaphragm designed to carry live loads is not necessary since the deck slab is supported directly by the very stiff backwall, the end diaphragm would actually carry very little load.

Jointless bridge decks at conventional abutments can be used under the following criteria:

- Approach slabs shall be used. See Section 13 of this manual for appropriate details.
- Maximum skew of 30° at the expansion end. It is difficult for the slab to slide over the backwall when the skew exceeds 30°.
- Jointless deck details may be used at abutments with U-wingwalls if the skew is less than 15° and there is a bond breaker between the U-wingwalls and the approach slab. See Section 13.1.3 for approach slab width criteria.
• Maximum skew of 60° at the fixed end.
• Jointless deck details may be used at the fixed end of the span even if a conventional expansion joint is used at the expansion end.
• On a curved girder bridge, jointless deck details may be used at the fixed end.
• Maximum expansion length at the abutment of 200 ft. (Expansion length is defined as the distance from the ℄ of the expansion bearing to the ℄ of the nearest fixed bearing.)
• When the expansion length at an abutment exceeds 65 ft., provision for expansion must be provided at the end of the approach slab by using the appropriate sleeper slab detail shown on the current BD sheet.

5.2.2 Jointless Decks at Integral and Semi-Integral Abutments

Another type of bridge with jointless deck details is one with integral or semi-integral abutments. While bridges with jointless decks over the abutments are supported on conventional abutments, they do not rigidly connect superstructure and substructure. If possible, the first choice for any bridge would be to construct an integral abutment, rather than construct a bridge with a jointless deck over conventional abutments. Integral abutments are more cost effective because of their simpler details. Situations where integral abutments cannot be used include locations where the footings are on rock, where sufficient pile penetration is not possible, or where a high wall abutment is necessary. In these situations, the possibility of a semi-integral abutment should be investigated before a jointless deck over the abutment is used. See Section 11.6.1 for Integral and Semi-Integral abutment considerations.

5.2.3 Jointless Decks at Piers (Link Slab)

Most of the multi-span bridges built prior to the mid 1970’s in New York used a simple span beam arrangement with deck joints. Over time these joints will leak and cause deterioration of bridge elements below the deck joints. Current DOT policy is to eliminate these joints whenever possible. One solution to eliminate the deck joint at a pier is to connect the simple span decks with a link slab.

The NYSDOT Office of Structures, Structures Design Bureau, has developed an innovative link slab design using Ultra High Performance Concrete (UHPC). UHPC used in NYSDOT applications contain high strength steel fibers. The presence of the high strength steel fibers in the mix allows a properly designed link slab to accommodate the necessary rotation at those connections without significant structural damage to the link slab or other parts of the bridge. The micro-cracking developed due to the bending of the link slab is very tight and should reduce moisture penetration to a negligible amount.

Our link slab is designed to flex with girder deflections as well as to transmit compressive and tensile loads between the spans. The design is influenced by, but not limited to, span arrangement, bearing type and arrangement, girder end rotation due to live load, bridge skew, and girder depth. The design and detailing of the link slab is fairly complex and the field performance data based on actual installations is currently limited. To help ensure the success of future applications the use of link slabs is limited to experimental applications and requires
approval by the DCES. Contact the Structures Design Bureau for design and detailing information.

5.3 Other Deck Types

Concrete decks are almost always used on bridges but other deck types can be used in special circumstances. Some of these deck types are discussed below:

**Timber** - Timber decks should only be used on low-volume rural roads. Timber decks can be of plank construction where timbers are fastened to stringers with their wide dimension horizontal. Timber decks can also be glue laminated or nail laminated with their narrow edge horizontal. There are many variations in details for timber decks. Timber decks will usually need some kind of wearing surface, in most cases asphalt, to make them more skid resistant.

**Open Steel Flooring** - This deck uses open steel grating supported on steel stringers. It should not be used for new construction because its open construction leaves the underlying structure vulnerable to corrosive attack. These decks also have low skid resistance. Open steel flooring is, however, a lightweight deck and is sometimes used in rehabilitation projects where reduction of dead load is important. Open steel flooring should be galvanized to increase its service life.

**Filled Steel Flooring** - Similar to open steel flooring except the grating is filled with a Class D (small aggregate) concrete, which improves protection of the structure and skid resistance of the deck.

**Composite Unfilled Grid Decks** - Composite unfilled grid decks, commonly referred to as Exodermic™ decks, are a lightweight, modular deck system comprised of a reinforced concrete slab with an unfilled steel grid. These decks can be cast-in-place or precast. Deck thicknesses may vary from 7½ to 10 inches.

Advantages are lighter weight without sacrificing stiffness or strength and speed of construction. Precast panels can often be erected during a short, overnight work window.

The specification for this product does not provide for design delegation. Therefore, it is the designer’s responsibility to design all aspects of the superstructure and provide all appropriate details in the contract plans. Use of composite unfilled grid decks requires approval of the D.C.E.S. Justification for using this system should include comparisons to other lightweight deck systems.

**Precast Concrete Decks** - There are a number of variations of this type. Their principal advantage is to shorten construction time. They can be advantageous for deck replacement projects in high traffic volume areas where detours and lane restrictions are limited. These decks can be full-depth concrete panels or a concrete deck supported by an unfilled steel grid (see Composite Unfilled Grid Decks above).

**Precast Precompressed Concrete/Steel Composite Superstructure** - This system, commonly referred to as Inverset™, is a combined superstructure and deck system made up of steel beams and a concrete slab. The deck is cast in the shop either in an inverted position or with the beams shored in an upright position. The casting process results in the steel beams
being prestressed and the concrete deck being precompressed. The advantages of this system include quicker construction, reduced superstructure depth and increased deck durability.

The specification for this product does not provide for design delegation. Therefore, it is the designer’s responsibility to design all aspects of the superstructure and provide all appropriate details in the contract plans.

**Fiber Reinforced Polymer Decks** - These decks consist of E-glass fibers embedded in a resin matrix. Although their use is new, they show great promise of increased durability. However, they are significantly more expensive than conventional concrete decks. They can be a great advantage on rehabilitation projects because of their extreme light weight (about 20% - 25% the weight of concrete). See Structures Design Advisory SDA 02-003.

### 5.4 Deck Drainage

It is important to provide good deck drainage on all structures primarily for traffic safety reasons, but also to prevent structure deterioration from ponding water and improperly directed drainage.

To facilitate runoff and provide better skid resistance, the surface of all concrete bridge decks and approach slabs is to be finished with longitudinal saw cut grooving. Grooves are 0.1 inches wide and 0.15 inches deep, spaced 0.75 inches on center, and are cut after the concrete has cured.

The most effective way to provide bridge deck drainage is to use curbless details. The required drainage must be balanced with railing/barrier requirements for the type of facility. Water quality issues must be considered before proposing to use curbless railing systems over waterways.

Good drainage design includes provisions to remove as much water as possible that would flow onto the bridge at the high end of the structure. This can be accomplished by locating drainage inlets approximately 10 ft. before either the further of the wingwall end or approach slab end when a curbed highway section exists. If there are no curbs, drainage should be handled with sod, asphalt or stone lined gutters.

If a bridge has curbs or traffic barriers it may be necessary to check the deck drainage design. Bridge deck drainage needs to be designed in accordance with FHWA Circular HEC No. 21 - Design of Bridge Deck Drainage, May 1993. The design is to be based on rainfall intensity of the most severe storm of five-minute duration likely to occur in a ten-year period.

Design criteria for bridge deck drainage are based on maintaining the following conditions:

- Maximum width for the spread of water is 12 ft.
- Maximum spread depth is ½ inch less than the curb height.
- For highways with design speeds less than 45 mph, puddles may encroach into a travel lane only to a point where 8 ft. of the lane remains unencroached by the puddle width.
- For highways with design speeds greater than or equal to 45 mph, puddles should not encroach into any portion of a travel lane.
If any of the above conditions cannot be met, scuppers (drains) must be provided. Scuppers typically become necessary with a combination of a long (over 325 ft.), wide (over 50 ft.) bridge and a flat grade (less than 2%). The average bridge typically does not require scuppers. They should not be used unless needed because of their tendency to contribute to deck and superstructure deterioration.

Consider scupper locations prior to finalizing girder spacing to avoid interference between the outlet and the girder flanges.

When used, scuppers should be located so they do not discharge onto travel lanes, sidewalks or railroad rights-of-way. Scuppers should be midway between cross frames or diaphragms and away from abutments and piers, if possible. Scuppers should have PVC downspouts extending at least 1 ft. below the superstructure. Diffusers should be used over land unless erosion protection is provided or the free fall exceeds 25 ft. Scuppers can discharge into downspouts carried down to ground level or to a closed drainage system. However, this method is discouraged because of the susceptibility of the downspouts to freezing or becoming plugged with debris. Bends in downspouts should be kept to a minimum. A clean out fitting should be located at each bend. Scupper details are shown on the current BD sheet.

Scupper grates should be of a bicycle-safe design. These are usually reticuline grates or parallel bar grates with welded transverse bars. See the FHWA publication Bicycle-Safe Grate Inlet Study for additional guidance.

In urban areas, if downspouts extend to the ground, and the potential exists for malicious damage, steel pipe may be used. Fiberglass downspout systems have more impact resistance than PVC systems.

Occasionally, downspouts have been encased in the substructure concrete. This practice should be avoided whenever possible, because it usually creates clean out problems and can also result in chloride damage to the concrete. If used, the installation shall include a 1 inch compressible protective covering between the pipe and the concrete to accommodate expansion of the pipe and shrinkage of the concrete.

Downspouts shall be placed at the least objectionable location by attempting to hide them from view behind columns. The surface below the outfall shall be protected by the use of a stone, concrete slab, or grouted block paving.

5.5 Deck Expansion Joints

5.5.1 Transverse Expansion Joints

Many deck joints and details have been used over the years, with varying results. The one constant result is that nearly all joint systems leaked after a short duration in service. Therefore, their use should be avoided whenever possible through the use of continuous spans, jointless abutments, and semi-integral or integral abutments.

Joint systems currently in use include armorless joints, armored joints and modular joints. See the current BD sheets for selection criteria for each joint system.
5.5.1.1 Armorless Joint Systems

Armorless joint systems are preferred for superstructure movement of 2½ inches or less. This range of movement has historically been handled by armored joint systems, which are no longer the preferred system (see Section 5.5.1.2). Armorless bridge joint systems are expected to alleviate many problems associated with armored joints and compression seals.

Armorless joint systems have been used by NYSDOT Bridge Maintenance for many years with excellent results. There are no skew limitations for armorless joint systems but skews over 45° require close attention to sizing criteria on the current BD sheets.

The elastomeric concrete used in armorless joint systems offers a durable header material that cures much faster than traditional concrete. This minimizes lane closure times, reduces Work Zone Traffic Control costs and shortens delays to the traveling public. Unlike traditional concrete, fresh elastomeric concrete bonds extremely well to previously placed fully cured material. It can be installed in segments, making it adaptable to stage construction as well as staged repairs or replacements. Elastomeric concrete headers shall not overhang the concrete slab.

The poured liquid sealant, closed-cell cross-linked foam seals or preformed seals of armorless joint systems are easily placed in their entirety or in segments. They require very little time to place and/or cure allowing restoration of traffic in a matter of hours.

When replacing only the existing armored joint and header for a minor rehabilitation and the opening between the deck and the backwall or deck slabs exceeds the maximum opening given in the BD sheets it may still be possible to use an armorless joint without doing additional deck work. If the maximum opening (set opening + design movement) does not exceed 5 inches an armorless joint can still be used.

5.5.1.2 Armored Joint Systems

Persistent maintenance problems with armored joints have been routinely encountered. During initial construction, proper consolidation of concrete under the horizontal leg of the armoring angle is difficult. The resulting voids lead to water collecting under the angle. When this water freezes it lifts up the armoring angle and increases the likelihood of snow plow impact.

An additional problem is corrosion of the steel angle. On the vertical face, corrosion creates a gap at the seal to angle interface which allows water to leak onto the superstructure and substructure elements below. On the horizontal face, corroding steel causes the concrete in contact with the angle to spall away, creating a larger gap for water to get under the angle. This causes leakage behind the angle in even when the seal remains watertight.

Repair of damaged armored joint systems is time consuming and difficult. Damaged compression seals cannot be repaired and must be replaced in their entirety. Typically the whole system needs replacing which requires removal and replacement of the concrete header and armoring angles. This requires jack-hammering, cutting out the steel angles, and placing new steel angles and concrete. The repaired section cannot be opened to traffic until the concrete has cured, requiring long term lane closures.
There are skew limitations for armored joint systems. See the current BD sheets for allowable skews and selection criteria.

5.5.1.3 Modular Joint Systems

Modular joint systems are used for larger movements. Single-cell modular joint systems may be used for up to 2 inches of superstructure movement. Multicell modular joint systems are used for superstructure movement over 2 inches. There are no skew limitations for modular joint systems but skews over 45° require close attention to sizing criteria on the current BD sheets.

5.5.2 Longitudinal Joints

When the bridge width exceeds 90 ft., a longitudinal deck joint should be considered. This is especially true for bridges whose width approaches or exceeds the bridge span. The type and placement of this joint should take the following bridge characteristics into consideration:

- Bridge deck drainage pattern (i.e., crossslope).
- Likelihood that traffic will have to traverse the joint.
- The existence of a raised or flush median.
- The existence and location of any median barrier.

A 1 inch joint is recommended if traffic is likely to traverse the joint. If a raised median with or without concrete traffic barrier is present, a 2 inch joint is recommended. If the joint is at or near the roadway surface, it should be sealed. If half-section adjacent concrete traffic barriers are used, the closure of the joint is optional. A compression-type seal is the recommended closure material in either case.

5.6 Sidewalk and Brush Curb Overlays

All sidewalks and brush curb overlays should be paid for under Item 557.30, Sidewalks and Safety Walks. The advantage of this item is that it includes the steel reinforcement and provides for a wet cure of the concrete.
6.3.2 Railing/Barrier Design Alternatives

Once the appropriate service level has been established, some functional and geometric criteria need to be established. These criteria are discussed as follows:

**Under-crossing Feature** - Bridges over another highway or railroad must have either a concrete barrier or a curb. This is necessary to prevent roadway drainage from dropping onto the under feature. Bridges over waterways may use a curbless section if not on an interstate or other controlled access highway.

**Pedestrian Traffic (Sidewalk on Bridge)** - Bridges carrying a sidewalk must use a concrete parapet or four-rail railing at the fascia with a minimum height of 3'-6" above the sidewalk surface. It is presumed that bridges with a sidewalk do not carry bicycle traffic on the sidewalk. When a sidewalk is separated from vehicular traffic by a traffic railing, then a minimum 3'-6" high pedestrian railing or fencing must be used on the fascia.

**Pedestrian Traffic (No Sidewalk on Bridge)** - A railing or concrete barrier with a minimum height above the roadway of 3'-6" shall be used.

**Bicycle Traffic** - If a bridge bicycle railing is to be used, it shall be a railing or combination concrete barrier and railing with a minimum height of 3'-6" above the roadway surface.

The *Highway Design Manual* (Chapters 17 and 18) should be consulted for warrants to determine when bicycle or pedestrian railing should be provided.

Bridges that carry bicycles on a bikeway that is separate from vehicular traffic may use either of the bicycle/pedestrian railings shown on BD-RP2 or BD-RP3 on the fascia of the bridge. If a steel railing is used to separate the traffic from the bikeway then a rub rail(s) should be placed on the back side of the traffic railing to protect the bicyclists from the railing posts. Fencing can be used as an alternate to the standard details shown, but the posts and rails must be designed to withstand the loads specified in the *NYSDOT LRFD Bridge Design Specifications* for bicycle and pedestrian railing.

Table 6-1 shows the available railing and barrier options for the different design service levels. Please note that precast barrier options are only available up to the TL-4 level. Current BD Sheets should be consulted for the details of the various systems.
<table>
<thead>
<tr>
<th>TL-2(Less than 500 AADT)</th>
<th>TL-2(Less than 1500 AADT)</th>
<th>TL-2(Greater than 1500 AADT)</th>
<th>TL-4</th>
<th>TL-5 and Controlled Access Interstate</th>
<th>Controlled Access Non-Interstate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Thrie Beam (BD-RL1E)</td>
<td>1. Double Box Beam-Rail Curbless (BD-RL3E)</td>
<td>1. 2’-10” Safety Shape (BD-RC1E)</td>
<td>1. 3’-6” Single-Slope [CIP and slipform options only] (BD-RC11E)</td>
<td>1. 3’-6” Single-Slope (BD-RC11E)</td>
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</tr>
<tr>
<td>2. Double Box Beam-Rail Curbless (BD-RL3E)</td>
<td>2. Steel Three-Rail Curbless (BD-RS1E)</td>
<td>2. Steel Three-Rail Curbless (BD-RC1E)</td>
<td>2. 3’-6” F-Shape (BD-RC14E)</td>
<td>2. 3’-6” F-Shape (BD-RC14E)</td>
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</tr>
<tr>
<td>5. Steel Five-Rail Curbless (BD-RS3E)</td>
<td>5. Steel Two-Rail with Brush Curb (BD-RS1E)</td>
<td>5. Steel Two-Rail with Brush Curb (BD-RC1E)</td>
<td>5. Steel Two-Rail with Brush Curb (BD-RC11E)</td>
<td>5. 3’-6” Single-Slope (BD-RC11E)</td>
<td></td>
</tr>
<tr>
<td>7. Timber Rail (BD-RT1E)</td>
<td>7. 2’-10” Safety Shape (BD-RC1E)</td>
<td>7. 3’-6” Single-Slope (BD-RC11E)</td>
<td>7. 3’-6” Single-Slope (BD-RC11E)</td>
<td>7. 3’-6” F-Shape (BD-RC14E)</td>
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<td>8. 2’-10” Safety Shape (BD-RC1E)</td>
<td>8. 3’-6” Single-Slope (BD-RC11E)</td>
<td>8. 3’-6” F-Shape (BD-RC14E)</td>
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<td>9. 3’-6” Single-Slope (BD-RC11E)</td>
<td>9. 3’-6” F-Shape (BD-RC14E)</td>
<td>9. 3’-6” Vertical Parapet (BD-RC2E)</td>
<td>9. 3’-6” Vertical Parapet (BD-RC2E)</td>
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<td>10. 3’-6” F-Shape (BD-RC14E)</td>
<td>10. 3’-6” Vertical Parapet (BD-RC2E)</td>
<td>10. 3’-6” Texas-Type (BD-RC8E)</td>
<td>10. Timber Rail (BD-RT1E)</td>
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<td>11. 3’-6” Vertical Parapet (BD-RC2E)</td>
<td>11. 3’-6” Texas-Type (BD-RC8E)</td>
<td>11. 3’-6” Texas-Type (BD-RC8E)</td>
<td>12. Timber Rail (BD-RT1E)</td>
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Table 6-1
Railing and Barrier Selection Table
6.3.3 Railing/Barrier Selection

6.3.3.1 Interstate and Controlled Access Highways

All new and replacement bridges and deck or superstructure replacements on interstate and other controlled access, high-speed highways shall use concrete bridge barrier (parkways without truck traffic and culvert structures are excluded). For interstate bridges, 3'-6" high F-Shape or single-slope barrier shall be used. For other fully or partially controlled access, high speed highways, designers should evaluate the required railing design service level according to Section 6.3.1 to determine if the service level is Test Level-4 and an 2'-10" high concrete safety shape barrier can be used.

Exceptions to this guidance should be discussed and justified in the Design Approval Document and be approved by the D.C.E.S. Exceptions that will be considered are in the cases of a deck replacement when the existing superstructure is not adequate for the increased dead load associated with a concrete barrier or where a concrete barrier on the inside of curve would reduce sight distance to less than the allowable.

A number of recent accidents have involved tractor trailers penetrating steel bridge rail and causing severe damage and injury. There is a common misperception that steel bridge railing is designed to contain a heavy tractor trailer impact. In reality, the current standard two-rail and four-rail bridge railings are designed and tested to a Test Level-4, under NCHRP 350, to contain a 4,400-lb pickup truck at 60 mph with a 25-degree angle of impact and an 18,000 lb single-unit van truck at 50 mph with a 15-degree angle of impact. The design standards for previous railing systems had significantly lower impact loads.

There are no known steel railing systems designed for an impact by an 80,000 lb tractor trailer (Test Level-5 level of service). It would be extremely difficult to design such a steel railing system because the impact force must be transferred to the deck at each post location. A concrete barrier is much more effective in that it distributes the force to the deck through the continuous deck/barrier interface.

6.3.3.2 Other Highways

The Railing and Barrier Selection Table (Table 6-1) lists the available choices for each design category. The first choice in most design categories is a concrete barrier or parapet. This preference is based on the concrete barrier’s strength, durability and low initial and maintenance costs compared to metal railing systems. Factors that may cause an alternative selection to be made are:

**Bridge Deck Drainage** - On bridges over waterways where concrete barriers would necessitate the use of scuppers, a curbless railing should be used. Generally, for most bridges it will not be necessary to use scuppers with concrete barriers. It is usually possible to carry the deck drainage off the ends of the structure without scuppers, unless the bridge becomes very long, wide or has a flat profile. The bridge deck hydraulics must be checked.
Aesthetics - In areas where the aesthetics of the railing/barrier is a prime concern, the Texas Type C411 concrete barrier is an option. However, the cost of this barrier is significantly higher than a standard barrier and its use is restricted to situations where a service level of TL-2 (PL-1) applies. A barrier with an outside face treatment using one of the many types of form liners should also be considered. Concrete cover and bridge width must be increased when form liners are used. Concrete barrier can be colored by staining the cured concrete for an aesthetic effect. Color added to the concrete mix is not recommended because of the variability of results. Exposed aggregate finishes should be avoided because of maintenance concerns.

A timber railing is also available for use in areas such as the Adirondack and Catskill Parks where a rustic appearance is desired. In certain situations it may be desirable to provide a view of scenic under features. An open railing system could be used in these situations.

Bridge or pedestrian railing may be painted the same color as the steel superstructure to achieve a uniform appearance. Use the appropriate Brown Rail item (568.nn) and place Note 63 on the plans indicating the color from 708-05 to match the color of the surrounding painted structural steel. Modify Note 63 to indicate that the railing is to be painted.

Visibility - When intersections or driveways are close to the end of the bridge, an open railing system may be selected over a concrete barrier to increase visibility of oncoming traffic from the intersecting roadway. It should be pointed out that the visibility through the steel railings is limited and becomes even less with the addition of pedestrian fencing or permanent snow fence to the railing. This factor should only be a consideration in unusual circumstances.

Snow Accumulation - In areas with heavy snowfall, Regions sometimes consider using open railing on bridges over waterways to mitigate the effect of snow accumulation on the shoulders. The intent is to push snow through an open railing during snow plowing operations to reduce the need for maintenance forces to remove accumulated snow from the bridge shoulder. However, the ability to push snow through the relatively close spacing of the rails is limited at best. Bridges over highways and railroads will ordinarily carry a snow fence on the structure. Therefore, snow accumulation is usually not a factor in the railing/barrier decision on such bridges.

Geometric design policy for new and replacement bridges ordinarily results in a shoulder wide enough to permit snow storage. The factor of snow accumulation driving a decision to use open railing rather than a concrete barrier should occur only in unusual circumstances.

6.3.4 Weathering Steel Bridge Railing

Use of weathering steel for bridge railing to achieve a “rustic” appearance is no longer allowed because accelerated deterioration has been noted inside the railing tubes. In most cases, standard galvanized guide rail should be used. If a rustic appearance is required, timber bridge railing or painted galvanized steel may be used.
6.3.5 Transitions

Approved transitions from bridge railing and barrier to highway railing are shown in the BD – RC, RL, RS and RT series. If it is necessary to transition from corrugated beam highway rail to box beam highway rail (or vice versa), make the transition away from the bridge in accordance with the details shown on the Highway Standard Sheets. The purpose of bridge railing/barrier transitions is to provide a smooth transition from the rigid bridge rail to the flexible highway guide rail without forming a snagging pocket.

When driveways or other roadways are in close proximity to the end of the bridge and make the use of the full transition length impossible, the designer shall utilize as much of the transition as possible. The highway guide rail shall be terminated in accordance with the highway standard sheets where conditions permit.

6.3.6 Modifications

Modifications to any of the standard railing/barrier systems may be made only with the approval of the D.C.E.S. Any substantial modifications would generally require a crash test to qualify the system. This will also be determined by the D.C.E.S.

6.4 Precast Concrete Barrier

The Contractor has the option of constructing concrete barrier by one of three methods: cast-in-place, slip formed or precast. If the precast method is chosen, the Contractor must use one of the preapproved precast concrete barrier systems. The approved systems are listed on the Department’s Technical Services - Materials – Approved list. The approved systems are specific in their details, materials and method of attachment to the deck slab. When a concrete barrier is cast on to a precast deck panel at an off-site location it is not considered a precast concrete barrier and does not need to be a pre approved system. The continuity details between panels need to be approved by the DCES.

In certain circumstances the designer may wish to require the use of a precast concrete barrier system. In that event, the normal barrier pay item can be used, but a note on the plans should state that only the precast option is allowed. No details of the barrier reinforcement or anchorage should be shown on the plans. A note should be placed to state that the precast barrier must be one of the approved systems.

6.5 Pedestrian Fencing

On bridges over railroads or highways where there is a potential for vandalism from pedestrians, pedestrian fencing should be provided. The fencing is attached to the back side of steel railings, concrete barriers and parapets. It is located on the back side to minimize the potential danger from flying debris if a truck impacts the railing or barrier and leans into the pedestrian fencing. As an alternate, fencing may be mounted to the top of a barrier through a longer base plate or corbelled edge as long as the standard distance from the face of the barrier to the fencing is maintained. Details are shown on the BD Sheets.
Pedestrian fencing over railroads shall be carried a minimum of 20 ft. past the center line of any single track or from the centerline of the two most external tracks. If there is an off-track maintenance roadway adjacent to the tracks, the fencing should be extended a distance of 3 ft. past the edge of the maintenance roadway. If the required limits of pedestrian fencing over the railroad corridor beneath the structure are a significant portion of the overall structure length, the Region may decide to simply run the pedestrian fencing along the entire length of the structure.

Pedestrian fencing shall have a minimum height of 8 ft. as detailed on the current BD sheets and extend to a point 10 ft. beyond edge of the shoulder of the under roadway.

6.6 Permanent Snow Fencing

Structures with open railing that pass over a roadway should be equipped with snow fence in the area over the under roadway. The purpose is to retain and disperse the snow from snow plowing operations. Permanent snow fence should be chain link fence mounted to the back side of the railing. If used, the recommended height of snow fence is 4 ft. as detailed on the current BD sheets.

Bridges with concrete traffic barriers (2’-10” high) may need snow fence installed on the back of the barrier depending on local conditions. It is recommended that bridges over interstate highways have such fencing. Bridges with higher concrete barrier or parapet (3’-6”) ordinarily do not require snow fence. If used, permanent snow fence on concrete barrier should have a height of 2 ft. above the top of the barrier. Permanent snow fence should be installed on the back side of railing and barrier for the same reason discussed under Pedestrian Fencing. As an alternate, it can be mounted to the top with certain restrictions as discussed in Section 6.5.

Permanent snow fence should be used judiciously. It has the potential to create more problems than it solves (particularly on concrete barrier) and may be unattractive. When snow fence is used, it should extend to a point 10 ft. beyond the edge of the shoulder of the under roadway.
However, if the designer notices potential problems with the bridge railing, the anchorage system, or other associated bridge rail hardware, it shall be communicated to the RSE and the RBME for their action.

The former two-rail and four-rail steel bridge railings detailed on various BDD sheets issued since 1977 are acceptable and adequate for a TL-2 service level without upgrading. See Appendix 6A, “1987 Bridge Railing Crash Test Report” for further discussion. However, any transition to highway guide railing containing the “tuning fork” detail is not adequate for a TL-2 service level.

In addition, for non-NHS roadways only, compliance to the TL-2 Service Level can be analytically determined by verifying the bridge railing as structurally adequate using the assumed loads given in AASHTO LRFD Bridge Design Specifications, Section 13. Some variance in rail, post and curb positions from crash tested systems is permissible if there are no obvious safety hazards such as snagging points and there is approval by the D.C.E.S.

2. Historic Preservation or Other Project Specific Reasons

For projects which deal with historic or aesthetic considerations, the decision regarding bridge railing can be much more difficult. The deficiencies of the proposed nonconforming bridge railing, relative to its conformance with the required service level, shall be clearly documented and shall be presented to the approving authority noted in Section 6.8.6. This information shall be accompanied by the cost differential between the two bridge railings and the logic supporting the decision to employ the nonconforming bridge railing.

6.8.5.4 Anchorage of Steel Bridge Railing

It is NYSDOT policy to allow drilling and grouting of anchor studs for steel bridge railing during rehabilitation projects. Drill and grouted anchor studs must be proof-load tested as per specification item 586.03 to ensure the quality of the existing concrete and the grout selected. Place note 102 on the plans if a significant number of anchor studs are to be installed and tested. To reduce the number of anchor studs tested the designer can designate the traffic side studs as item 586.03 (pull out testing required) and item 586.02 for the fascia side studs.

The estimated embedment depth for 1” diameter studs is 12”. The actual embedment depth shall be calculated for the 1” anchor studs based on the actual edge distance of the concrete, the spacing of the studs and the requirements of the chosen grout supplier from the appropriate Approved Material and Equipment List. Place note 103 on the plans.

Although the anchorage is compliant with current loading requirement, the overhang reinforcement in the superstructure may not be adequate. The deck reinforcement should be investigated to ensure that it can resist the larger loadings a new railing system may transmit, or a determination must be made to accept the damage to the deck that may occur during a severe impact.
6.8.6 Responsibilities and Authorities

Approval authority will be in accordance with the Design-Related Approval Matrix in the NYSDOT Project Development Manual, Exhibit 4-2.

6.9 Bridge Railing/Transition Shop Drawing Requirements

Bridge Railing and Transition Shop Drawing “Approvals” are not required in most cases. Since the recent implementation of new crash-tested bridge railing and transition details, it has become obvious that the shop drawing review process provides little value when compared to the effort of reviewing and approving shop drawings for these items. In most cases, the contract document details and construction specifications are adequate to ensure that the railing system will be fabricated in a manner that will satisfy safety and construction tolerance criteria.

Nevertheless, there are situations that warrant the review and approval of shop drawings for these items, as follows:

- Transitions requiring connections between existing bridge rail and existing highway rail.
- Transitions requiring connections between existing bridge rail and new, upgraded bridge rail or between existing bridge rail and existing truss members.
- Unique and complex end transitions.
- All nonstandard concrete and steel railing systems and all timber rail systems.

When these situations occur, Note 73 in Section 17 shall be placed in the contract plans.
Section 7
Utilities

7.1 Criteria for Utility Placement on Bridges

The New York Code of Rules and Regulations (NYCRR) states, "It is in the public interest for utility facilities to be accommodated within the highway rights-of-way when such use and occupancy does not interfere with the free and safe flow of traffic . . ." The decision to allow a utility on a bridge rests with the Region. Most Regions make all reasonable efforts to accommodate utilities on bridges. The designer needs to be aware of the responsibilities of the utilities and of the rules governing the placement of utilities on bridges. See Chapter 13 of the Highway Design Manual for information about regulations and procedures.

7.2 Design Information Furnished by Utilities

It is the responsibility of the utility to design the carrier and provide the Department with:

- Unit weight of the utility (assuming all ducts or carriers are full).
- Maximum allowable span of the ducts or pipes.
- Type of expansion system.
- Desired support details.
- Material specifications for carriers, coatings, expansion devices, etc.

The designer should not be designing the carrier pipe for a utility, only the support system. A review of the information the utility provides is prudent.

7.3 Utility Locations

The designer, in consultation with the utility, should select the utility location in the following decreasing order of preference:

- In the sidewalk for small diameter ducts carrying telephone, electrical or cable television lines. No more than six 2 1/2” ducts shall be used in a standard sidewalk.
- In the bays between main longitudinal members or in a void created by spreading adjacent prestressed concrete box beams under a sidewalk.
- A maximum of two 2 1/2” conduits may be carried in a concrete traffic barrier.
- On a utility ledge or outrigger (preferably on the downstream side).
- Attached to the fascia (preferably on the downstream side).
- In the voids of closed box bridge members.
- Structural support system separate from the bridge.
7.4 Design Criteria for Utilities and Supports

Rules governing utilities on bridges that the designer needs to be aware of are:

- Utilities are not allowed on an existing bridge if the load rating would be reduced below the legal limit.
- The plans must fully detail the utility installation.
- The utility (and all supports) must be above the bottom of the superstructure.
- The utility should not be attached to a railing.
- Utilities shall not be hung from the structural slab.
- No welding is permitted to connect utility hangers to existing structural steel.
- Thermal movements must be accommodated by:
  - Utility expansion devices located at bridge deck expansion joints (at both abutments for bridges with integral abutments), or
  - Supporting the utility on a system of rollers so it moves independently of the bridge, or
  - A combination of the above.
- The utility shall be marked with the carrier contents.
- Water and sewer lines shall either:
  - Have welded or restrained joints, or
  - Be cased for the length necessary to prevent liquid from falling on the underlying highway or railway.
- Supports for heavy utilities should be designed to minimize local bending in the support members. This can be accomplished by the use of beam clamps rather than a rod passing through a single thin flange.
- The design and placement of utility supports should consider the need to inspect, paint and otherwise maintain the bridge.
- On concrete box beam bridges it may be feasible to separate the boxes under the sidewalk to create a utility bay if fascia installation is not desired. (This should be done only under a sidewalk.)
- Flexible jointed water mains can zigzag when pressurized unless they are properly supported. This problem can be prevented by using top and bottom rollers at two locations on every other section of pipe to provide lateral restraint. Intermediate sections need only have one roller location, unless otherwise required by design.
Section Eight
Structural Steel

8.1 Design

8.1.1 Design Methods

Structural steel has long been used as a bridge material in New York State. It continues to be commonly used and is the usual choice for spans over 115 feet. Structural steel design should be in accordance with the NYSDOT LRFD Bridge Design Specifications for all new and replacement bridges. The NYSDOT Standard Specifications for Highway Bridges may be used for rehabilitation of existing bridges.

Load and Resistance Factor Design (LRFD) is the required design method for all new steel structures designed in New York State. It introduces limit states as a design philosophy and uses structural reliability methods to achieve a more uniform level of safety. Factor of Safety is replaced with a new statistically based measure of safety called the Reliability Index “β”. LRFD requires a Design Reliability β=3.5, which provides for a notional failure probability of 1 in 10,000.

The LRFD code defines four design limit state categories:

- Strength Limit States - ensure strength and stability, both local and global.
- Service Limit States - impose limits on stress and deformation.
- Fatigue and Fracture Limit States - limit the liveload stress range under regular service conditions.
- Extreme Event Limit States - ensure the structural survival of a bridge during a major event such as a vessel collision, flood, earthquake, etc.

Within each category there are multiple limit states. Steel bridges shall be designed using Strength 1 (for moment and shear), Service 2 (overload, liveload deflection, bolted connections) and fatigue. A Strength 2 limit check of new girders utilizing the NYSDOT Design Permit Vehicle is also required.

LRFD introduces new live load criteria which will provide heavier loads on shorter spans and lighter loads on longer spans than are provided in the LFD specification.

Service Load Design, also known as Allowable Stress Design (ASD), is the older and generally more conservative design method for medium to long bridge spans (over 100 ft). ASD achieves its factor of safety by limiting the stresses on the member to some percentage of the maximum stresses that the member could take before yielding. Since the dead load and live load stresses are considered at the same time, there is no provision for the certainty of the dead loads or the uncertainty of the live loads. As span lengths increase and dead loads become a much higher percentage of the total load, ASD becomes overly conservative and uneconomical.
**Strength Design**, also known as **Load Factor Design (LFD)**, achieves its factor of safety by applying multipliers, or load factors, to the design loads. These multipliers increase the load effects, or stresses, applied to the member above those induced from the design loads alone. Since the dead loads are known, the load factor applied to them is relatively small. By comparison, live loads are highly variable and, therefore, the applied load factor is relatively large. The factored stresses are then compared to the yield stress, or ultimate capacity, of the loaded member.

The benefit of handling dead loads and live loads separately is that it provides a uniform factor of safety for live load in bridges of any span length. As span length increases and dead load becomes a larger part of the total load, LFD becomes increasingly more economical than ASD because of the smaller load factor applied to the dead load.

LFD must always be checked for deflection and serviceability criteria. Designers are cautioned that at very long span lengths, typically in excess of 400 feet, LFD may not provide adequate reserve strength capacity in the bridge.

### 8.1.2 Analysis Methods

Straight girders should ordinarily be analyzed by the line element method. Only in very unusual circumstances should it be necessary to analyze a straight girder bridge by a grid, three-dimensional or finite-element analysis. The marginally increased refinement in the analysis offered by these techniques does not usually justify their substantially increased design effort. This conclusion is justified in large part by the fact that design loadings are only an approximation of actual traffic loads.

However, in some instances these more exact methods are justified. They are required for bridges with girders that have enough curvature to meet the requirements for curved girder analysis as defined by AASHTO. Some straight girder bridges that have extremely large skews (in excess of 45°), unfavorable continuous span arrangements, or faying girders (secondary girders framed to main girders for unusual geometric situations) may be candidates for a more exact analysis.

When a bridge is designed using a grid, three-dimensional or finite-element method of analysis; and has diaphragms and/or bracing members acting as primary members (load paths), the qualifying information and Note 20 from Section 17.3 shall be placed on the contract plans. These conditions have special requirements for fabrication and erection of the bridge. Bridge types where this may apply include:

- Curved girder bridges with radii less than 600 feet
- Multi-span curved girder bridges with skews greater than 45 degrees.
- Curved tub girder bridges
- Skewed truss bridges
- Arch, and Tied Arch bridges
- Rigid frames
8.1.3 Design Considerations

The LRFD specification increases the role and responsibility of the designer to anticipate construction related issues and be aware that stresses during erection or construction are sometimes the controlling conditions of design. Examples of conditions that need to be checked are the erection of the girder and the placement of the concrete deck, both of which occur when there is a long unbraced compression flange. The designer should refer to Article 6.10.3 of the NYSDOT LRFD Bridge Design Specifications for requirements for stability checks.

8.2 Steel Types

8.2.1 Unpainted Weathering Steel

The preferred structural steel is unpainted weathering steel. Two grades are available; ASTM A709 Grade 50W and Grade 70 HPS - 70W. This steel eliminates the need for painting because the steel “weathers” to form a protective patina, or thin layer of protective oxide coating, that prevents the steel from further rusting. Its slightly higher cost per pound than nonweathering steels is easily offset by the savings in initial and maintenance painting. This steel should be used in most situations.

However, weathering steel has been known to exhibit problems in certain situations. These have generally been in environments where the steel has been exposed to wet conditions, salt spray or chemical fumes over prolonged periods. In these situations weathering steel may be unable to properly form the protective patina surface. The steel may be prone to delamination during the corrosion process and rapidly lose large amounts of its weathered surface material. Therefore, unpainted weathering steel should not be used under the following circumstances:

- Grade separation structures in “tunnel like” conditions where the steel is highly exposed to salt spray from the under roadway. These conditions can occur when there is minimum vertical clearance and substructures are located relatively close to the travel lanes of the under roadway
- Bridges over low water crossings where the structural steel is less than 8 feet over the ordinary water elevation.
- Marine coastal areas.
- Industrial areas where concentrated chemical fumes may drift directly onto the structure.
- Bridges exposed to spray from adjacent waterfalls or dam spillways, or located in an area of high rainfall, high humidity or persistent fog.
- Areas where debris can collect and primary connections may be exposed to roadway drainage (e.g., bottom chords of thru truss structures).
- Any staining of substructure is unacceptable.
- Color of weathering steel is not appropriate for aesthetic reasons.

It is strongly recommended that all weathering superstructure steel be painted within a distance of 1.5 x depth of the girder from bridge joints. Additionally, if the appearance of a partially painted girder is an aesthetic concern, the exposed area of the fascia girders should be painted for the entire girder length. This would include the entire fascia girder except for the top of the
top flange and the interior surfaces of the web and top and bottom flanges. If a timber deck is used, see Section 10 - Timber for additional protective measures.

In locations where the guidelines do not specifically prohibit the use of weathering steel, but conditions such as excessive salt spray may compromise structural performance, the designer should increase flange and web thickness by approximately \(\frac{1}{16}\) inch, if weathering steel is used. This will act as sacrificial section in order to achieve the intended service life.

### 8.2.2 Drip Bars for Unpainted Weathering Steel

The use of unpainted weathering steel for bridge superstructures results in the potential for staining bridge substructures during the period when the superstructure steel is developing a protective oxide coating. Rainwater flowing along the steel carries iron oxide particulates which are deposited on pedestals, abutment stems and pier caps.

While various methods for reducing or eliminating staining of substructures have been tried with varying success, current practice is to attach deflectors, called drip bars, to the bottom flanges of stringers in selected locations.

Drip bars are normally used only on structures having substructure units clearly visible to the public, such as piers or high abutments adjacent to an under roadway. It is not expected they would be used on structures over railroads, water, or at stub abutments of structures over highways.

Use of drip bars is determined at the Preliminary Plan stage of a project. If used, they are attached to the bottom flange of each fascia stringer at the low end of appropriate spans.

### 8.2.3 Painted Steels

When painted steel is used for aesthetic reasons or in situations where uncoated weathering steel is not desirable, ASTM A709 Grade 50 steel should preferable be used. It is usually the economical choice over Grade 36 steel. In structures that have only a small portion of the steel painted, such as beneath the joint systems of typical plate girder bridges, ASTM A709 Grade 50W steel should be used.

In structures that use painted steel it is possible to design main members using ASTM A709 Grade 50 and use ASTM A709 Grade 36 for secondary members and details. However, the cost differential between ASTM Grade 50 and ASTM Grade 36 is small, and it is therefore recommended for uniformity to use all Grade 50 steel.

In structures that need to have large portions of the steel painted, such as thru trusses, the entire structure should be painted rather than use weathering steel painted only in the splash zone. It is very difficult to paint steel to match the appearance of unpainted weathering steel.

The “Structural Painting Details” note required by Item 572.01, Structural Steel Painting: Shop Applied, shall contain the following information: description of serialized items, estimated structure length, width, vertical clearance, pay items to be used, description and location for pay
items 574.02 and 574.03 if necessary, stream classification, and whether or not the structure is over a public water supply.

8.2.4 HPS Steel

The use of HPS steel requires approval by the D.C.E.S. HPS steel should be considered only when one of the following conditions exists:

- The layout of the structure can be reorganized to eliminate an entire span. As an example, if a proposed structure designed without using HPS is a five-span simply supported steel superstructure and can be replaced with a three-span continuous structure if HPS is used, HPS steel may be the best solution.
- One or more girders can be eliminated from a bridge cross section.
- The bridge requires a reduced superstructure depth, based on critical vertical clearance issues, which cannot be accomplished without using HPS.

Recent experience has shown that price analyses based on weight savings alone are not truly representative of final erected steel costs. Therefore, designers should include the following parameters in their cost analysis when deciding whether or not to incorporate HPS steel on a project:

- The added cost of splicing the higher strength steel
  - Bolted field splices must develop higher allowable strengths, which necessitate a greater number of bolts and longer length bolts to accommodate the increased pattern size. Consideration should be given to using Grade 50 steel to reduce cost.
  - For shop splices, because of the limits of the rolling stock available, there will be more splices in a specific size flange or web. Also, there will be an increased cost in extra required nondestructive testing.

- Erection cost - Because of extreme flexibility in the structure due to the large span to depth ratio high performance steel allows, there is a concern for lateral flange buckling. Additional falsework may be required to ensure the stability of members during erection.

- Shipping costs will increase because of the greater flexibility of the shipped units.

8.2.5 Other Steels

Various other steel types are used for special situations such as sheet piling and railing tubes. If any steel other than A709 Grade 36, Grade 50 or Grade 50W is to be used for primary structural members, approval of the D.C.E.S. is required.
8.2.6 Combination of Steel Types

When more than one type of steel is used in a contract, the types shall be clearly described in the plans. The payment for furnishing and placing these steels shall be made under a single structural steel item. A table titled “Total Weight for Progress Payments” shall be placed on the plans adjacent to the estimate table, indicating the quantity of each type of steel.

8.2.7 Steel Item Numbers

Depending on the type and nature of a project, steel shall be paid for under Item 564.XX or Item 656.0101 as described below. These items include the cost of the steel, shop drilled holes, and bolts.

On steel rehabilitation projects, designers must remember to include item numbers in the contract for steel removal (which includes the cost of bolt and/or rivet removal), field drilling of existing steel, and rivet removal and replacement with high strength bolts where applicable. See Section 19.4.5 for further information regarding rehabilitation of riveted structures.

Item 564.05XX, Structural Steel, L.S.
- New bridges and superstructure replacements.
- Shop drawings reviewed by D.C.E.S.

Item 564.10nnnn, Structural Steel Replacement, lb.
- Minor rehabilitation projects, with variable quantities due to unknown deterioration.
- Secondary member repair/replacement, minor repair to primary members: (e.g., diaphragm replacements and replacement of primary member stiffeners and/or connection angles.)
- Quantities verified by the Engineer-In-Charge.
- Shop Drawings reviewed by the Engineer-In-Charge.
- Stock steel option is allowed.

Item 564.51nnnn, Structural Steel, lb.
- Major rehabilitation contracts, with variable quantities due to unknown deterioration.
- Primary member replacement or strengthening: (e.g., truss rehabilitations, girder web and flange repairs, floor beam and stringer replacements, continuity retrofits and seismic retrofits).
- Quantities verified by the Engineer-In-Charge.
- Shop Drawings reviewed by D.C.E.S.

Item 564.70nnnn, Structural Steel Replacement, Each
- Minor rehabilitation projects with known quantities.
- Secondary member repair/replacement, minor repair to primary member components: (e.g., diaphragm replacements, and replacement of primary member stiffeners and/or connection angles.)
- Shop Drawings reviewed by the Engineer-In-Charge unless otherwise specified in the contract documents. Designer should consult with the Metals Engineering Unit to determine when D.C.E.S. review of shop drawings is required.
- Stock steel option is allowed.

Item 656.01, Miscellaneous Metals, lb.

- Used for extraneous items. (e.g., hand rails, metal floor grating, ladders).
- Shop Drawings reviewed as per NYSDOT Steel Construction Manual.

8.3 Redundancy - Fracture Critical Members

8.3.1 Primary and Secondary Members

Primary members are defined as structural elements that are designed to carry live load and act as primary load paths. Examples include: truss chords; girders; floor beams; stringers; arches; towers; bents; rigid frames. Additionally, lateral connection plates welded to the members listed above, and hangers, connection plates, and gusset plates which support the members listed above are primary members. Tub and curved-girder diaphragms are also included.

Secondary members are defined as those structural elements which do not carry primary stress or act as primary load paths.

8.3.2 Redundancy

Redundancy in structures is the ability of a structure to absorb the failure of a main component without the collapse of the structure. Superstructures have three types of redundancy:

- Load path redundancy.
- Structural redundancy.
- Internal redundancy.

With load path redundancy, the loads will be transferred to adjacent members or alternate paths with the failure of a single member. The best example of load path redundancy is a bridge with four or more longitudinal main girders. Structural redundancy is best typified by the middle spans in a continuous span bridge. Indeterminate trusses can also be structurally redundant. Internal redundancy occurs when a girder is composed of a number of components such as angles and plates which are connected by rivets or bolts (not welded). Only the first form of redundancy, load path redundancy, is generally counted on in design.

8.3.3 Fracture-Critical Members

Fracture-Critical Members are defined as tension members or tension components of nonredundant members whose failure would result in the collapse of the structure. Tension components include any member that is loaded axially in tension or that portion of a flexural member that is subjected to tensile stress. Any attachment that is welded to a tension area of a fracture critical member or component is considered to be part of that member or component and, therefore, also fracture critical. It is important to realize that members can be nonredundant without being fracture critical (e.g., the compression chord of a truss is nonredundant but it is not fracture critical).
Examples of fracture-critical members or components are the tension flange and web of two- and three-girder systems, tension flange and web of steel pier cap beams, the tension chord and diagonals of trusses, the tie girders of a tied-arch bridge and the floor beams in a truss or thru girder that are spaced more than 12 feet on centers. All single tub and box girder structures shall be considered fracture critical. Some columns are fracture critical as defined by the designing engineer.

Examples of non-fracture-critical members are all components of the girders in any bridge with four or more girders, the compression chord of a truss and the stringers in a floor system of a thru girder or truss. Two- and three-girder pedestrian bridges and truss pedestrian bridges should not be considered fracture critical because they are not subject to high numbers of load cycles.

Bridges containing fracture-critical members should be avoided if possible. However, it is recognized that in many situations there is no good alternative to their use. Vertical clearance restrictions may necessitate the use of thru truss or thru girder structures. When spans become very long it also becomes cost prohibitive to provide a load-path-redundant structure.

Bridges that have fracture critical members have restricted allowable fatigue stress ranges and more stringent fabrication requirements. These issues are covered in the NYSDOT Standard Specifications for Highway Bridges and in the NYSDOT Steel Construction Manual. The NYSDOT LRFD Bridge Design Specifications requirements for fatigue design do not differentiate between redundant and nonredundant members. For this specification, both redundant and nonredundant members are designed for an infinite fatigue life. Fracture-critical members designed with this code are still subject to the fabrication requirements of the NYSDOT Steel Construction Manual.

- Designers shall designate and provide a table of all fracture-critical members on the contract plans.
- Designers shall designate tension zones of all fracture-critical members on the contract plans.
- When the Designer has determined that the column or column system is fracture critical, they shall designate all column components as fracture critical on new steel bents where columns experience tension under LRFD Strength III loading.
- When the Designer has determined that the column or column system is fracture critical, they shall designate all column strengthening components as fracture critical on major rehabilitations where a significant portion of the work is associated with the seismic strengthening and/or retrofitting of the structure.

8.4 Economical Design

8.4.1 Girder Spacing

A key element in producing an economical steel bridge design is the selection of girder spacing. While no absolute rule can be stated, the most economical design is usually the one with the least number of girders. There are, however, limitations that must be worked within. There should be a minimum of four girders and their spacing should not ordinarily exceed 12 feet. In addition, restrictions on the available clearance requirements may force the use of more girders.
Stage construction requirements may have an impact on girder spacing, but there is no requirement to have more than four girders or an even or odd number of girders. Bridges can generally be stage constructed as easily with four girders as with five. It is good practice to check the economics of two or possibly three alternate girder spacings.

### 8.4.2 Girder Proportioning for Plate Girders

#### 8.4.2.1 General

It is important to remember when proportioning plate girders that the design resulting in the least weight of structural steel is not necessarily the least costly option. Increased fabrication, construction, transportation and erection costs can easily outweigh a small savings in the quantity of steel used. Economical steel designs use good details and good proportions.

Generally, web and flange plate sizes and lengths for interior and fascia girders should be the same, with differences in deadload deflections between interior and fascia girders accommodated in the camber table.

#### 8.4.2.2 Depth

There is an optimum depth to plate girder design. If there is flexibility in the allowable girder depth then a number of options should be explored to develop an economical design. Weight and cost of a girder will usually decrease as girder depth increases but only to a point. Beyond this point the weight and cost will increase as the girder depth is further increased. Very deep girders with small flanges may prove to be unstable and difficult to transport and erect.

#### 8.4.2.3 Flanges

Minimum flange thickness shall be ¾” and minimum plate girder flange width shall be 12”.

When designing flanges, it is important to keep in mind that, in general, the most economical way for steel fabricators to make up flanges is to butt weld together several wide plates of varying thickness and then strip the flanges from the wide plate. Plate is usually purchased in widths starting at 4 feet. For the ordinary bridge, this usually makes it more economical to vary flange thickness rather than width. In large bridges, where there are significant changes in girder section needed and the quantities of each plate size are large, this guideline may be impractical or irrelevant.

Flanges should not be excessively wide compared to girder depth nor should they be excessively thick compared to the girder web thickness. A good rule of thumb is that the flange thickness should be no more than six times the web thickness.

As moment and shear change along the length of the girder, the required section of the girder also changes. It is frequently economical to introduce flange splices to utilize a lighter flange plate where possible. The savings in material achieved by making the splice must be balanced against the increased fabrication cost to make the butt weld. If the mass of material saved by
making the splice is more than the amount computed by the following guidelines, then it is economical to make the splice.

   Grade 36 steel:
   lbs saved ≥ 300 + (25 x cross sectional area of smaller flange (in²))

   Grade 50 and 50W steel:
   lbs saved ≥ 1.33 x (300 + (25 x cross sectional area of smaller flange (in²)))

   When making flange plate size changes, the thicker plate shall not be greater than twice the thickness of the thinner plate. It is good practice not to change the sectional area of the flange plates by more than a factor of 2 or the width by more than 8 inches. Flange transitions shall be tapered 1 on 4 for width transitions and 1 on 2.5 for thickness transitions. It is usually preferred to transition thickness rather than width.

   8.4.2.4 Webs

   It is recommended that webs of plate girders be at least ½” in thickness.

   Web thickness is varied only in unusual circumstances. It is the standard practice to keep web thickness constant throughout the length of the girder. This is done for uniformity and in keeping splice and connection details simpler.

   The main issue in economic web designs is whether or not to use stiffeners. It is usually the best choice to thicken webs sufficiently so that transverse stiffeners are not needed on girders under 48 in. depth. For girder webs above that depth, a good economic choice is usually to thicken the web sufficiently so that only a few transverse stiffeners are required in areas of high shear. Longitudinal stiffeners are rarely used and they become an option only with very large web depths. Designers should always check to see whether a stiffened or unstiffened web is more economical. Web thickness should be determined for both cases. The following guide can be used to help make the choice. It is economical to use the thicker web if the necessary thickness increase of the web does not exceed the amounts shown:

   Grade 36 steel:
   $\text{Increase in } t_w \leq \frac{(N(36 + W_{ST})}{41L}$

   Grade 50 and 50W steel:
   $\text{Increase in } t_w \leq \frac{(N(28 + W_{ST})}{41L}$

   where:

   $t_w = \text{web thickness in inches}$
   $N = \text{number of stiffeners to be removed}$
   $W_{ST} = \text{weight in lb/linear ft. of one stiffener}$
   $L = \text{length of web in feet to be increased}$
8.4.2.5 Stability During Erection

Stability of structural steel during transportation and erection is the Contractor’s responsibility. However, designers must ensure that the structural steel can be erected without requiring extraordinary means of support. If the structure is designed using the NYSDOT Standard Specifications for Highway Bridges, the designer must check the local buckling stress of the compression flange due to steel dead load only during erection procedures. The designer must assume the location of field splices, determine segment lengths, and analyze each segment using the buckling stress and factor of safety requirements given in “Blue Page” Article 10.34.7 of the NYSDOT Standard Specifications for Highway Bridges. The stability of the spliced girder is the responsibility of the Contractor. If the calculated Factor of Safety against local compression buckling is less than 1.1, the designer shall increase the area of the compression flange or specify other means of temporary bracing.

If the structure is designed using the NYSDOT LRFD Bridge Design Specifications, splices are done by the designer and detailed in the Contract Plans. The girder segments must be checked according to the provisions of “Blue Page” Article 6.10.3.1.a (place Blue Page Note 1 or Note 2 on the plans). Detailed information on splice locations and maximum shipping lengths is provided in section 8.11.

8.4.3 Rolled Beams

Designers should check the economics of using rolled beams versus plate girders on short spans (under 100 ft.). Four alternatives in order of increasing fabrication cost should be considered.

- Rolled section
- Rolled section with cover plate on bottom flange
- Rolled section with cover plates on both top and bottom flanges
- Fabricated plate girder

Either of the first two alternatives may be more economical than a plate girder that uses less steel weight. Only in rare situations would the third alternative be cost effective because the total amount of time required to fabricate the beam would be comparable to that of a plate girder. Designers should not compare alternatives based on material weight savings alone. Rather, they should include potential savings achieved through the elimination of an operation during fabrication or through the elimination of field operations.

When specifying heavy W-shapes of any length (section weight greater than 370 lb/ft or flange thickness greater than 2 ¼”) or any rolled beam longer than 65 feet in length the designer should check with the Metals Engineering Unit for availability of the shape.
Generally, cover plates should be used only on simple span structures. Two options are available:

- Full-length cover plates.
- Partial length cover plates using the end bolted detail shown in Fig. 10.3.1C in the NYSDOT Standard Specifications for Highway Bridges or Fig. 6.6.1.2.3-1 in the NYSDOT LRFD Bridge Design Specifications.

When full-length cover plates are used, they shall be extended so that the end of the plate is a maximum distance of 12 inches from the centerline of bearings. The purpose of the limitations is to move the undesirable Category E fatigue detail to a region of low stress range. Full length cover plates shall be welded to the flanges as shown in Figure 8.1.

**Figure 8.1**
Cover Plate Connections
8.5 Metal Thicknesses

An effort should be made to design and detail steel plate in the following thicknesses:

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Thickness</th>
<th>Thickness</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/16&quot;</td>
<td>9/16</td>
<td>15/16</td>
<td>1 ⅝</td>
</tr>
<tr>
<td>¼</td>
<td>¾</td>
<td>1</td>
<td>1 ¾</td>
</tr>
<tr>
<td>5/16</td>
<td>11/16</td>
<td>1 ⅛</td>
<td>1 ¾</td>
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<tr>
<td>⅜</td>
<td>⅝</td>
<td>1 ⅝</td>
<td>2</td>
</tr>
<tr>
<td>⅝</td>
<td>7/16</td>
<td>1 ⅜</td>
<td>2 ⅛</td>
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<td>⅞</td>
<td>⅛</td>
<td>1 ⅜</td>
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</tr>
<tr>
<td>1 ⅝</td>
<td>⅛</td>
<td>1 ⅜</td>
<td>2 ⅛</td>
</tr>
</tbody>
</table>

Over 2 ¼" use ¼" increments.

Table 8-1
Steel Plate Thicknesses

Structural steel, (including lateral bracing, cross frames, diaphragms and all types of gusset plates), except for the webs of certain rolled shapes, shall have a minimum thickness of 3/8". The web thicknesses of rolled beams, channels and structural tees shall be a minimum of ¼". However, webs less than 3/8" may require special welding procedures. These minimum thicknesses are specified to insure adequate protection against potential loss of section from corrosion. In areas where the metal is exposed to marked corrosive influences, it should be increased in thickness or specially protected.

Fill plates necessary to make connections are not subject to the 3/8" minimum thickness requirements.

When plates are called out on the plans, their dimensions are called out in the following order: width x thickness x length.

8.6 Connections

8.6.1 General

Connections are a very important part of any structural steel design. Good details are important for strength, serviceability and maintenance of the structure as well as for economical construction.

Shop connections are usually designed as welded connections. Bolted connections are preferred in the field because automatic shop welding processes are often impractical in the field.
8.6.2 Bolts

All bolted connections on bridge projects shall be designed as slip critical, with Class A surface conditions, unless otherwise approved by the D.C.E.S. Bolt lengths shall be such that threads are excluded from the shear planes in the connection. When individual bolts are shown in horizontal joints on the plans, they should be shown with the bolt head up.

8.6.2.1 Bolt Types

ASTM A325 high strength bolts are preferred. A490 bolts should be used only when necessary and require D.C.E.S. approval.

Designers shall provide the following information on the contract plans for all structural steel connections: the design surface condition (Class A or B), the number of bolts, the bolt type, and the bolt diameter.

Bolt types are as follows:

- Non-Weathering steel applications (Shop applied zinc-rich primer)
  Mxx high-strength ASTM A325 (Type 1) or
  Mxx high-strength ASTM A325 (Type 1, hot dipped galvanized)
  Designers shall show both types of bolts on the contract plans. Choice is at Contractors discretion with only one type of bolt used per bridge.

- Weathering steel applications (Painted or Unpainted)
  Mxx high-strength ASTM A325 (Type 3)

- Galvanized steel applications
  Mxx high-strength ASTM A325 (Type 1, hot dipped galvanized)

8.6.2.2 Bolt Sizes

The normal size of high-strength bolts is \( \frac{7}{8} \) inch. An effort should be made to keep field bolts all the same size to avoid confusion.

\( \frac{3}{8} \) inch bolts shall not be used in members carrying calculated stress except in 2.5 inch legs of angles and in flanges of sections requiring \( \frac{3}{8} \) inch fasteners. Structural shapes which do not permit the use of \( \frac{3}{8} \) inch fasteners shall not be used except in handrails.

The diameter of fasteners in angles carrying calculated stress shall not exceed \( \frac{3}{4} \) the width of the angle leg in which they are placed. In angles whose size is not determined by calculated stress, \( \frac{3}{8} \) inch fasteners may be used in 2 inch legs.
8.6.2.3 Bolt Spacing

Bolt spacing is not ordinarily shown on the contract plans. This detail is best left to the fabricator. The contract plans should show the number of bolts and be checked to assure that the connection can be fabricated. However, bolt spacing is required on all splice design drawings.

The pitch of fasteners is the distance along the line of principal stress, in inches, between centers of adjacent fasteners, measured along one or more fastener lines. The gage of fasteners is the distance in inches between adjacent lines of fasteners or the distance from the back of angle or other shape to the first line of fasteners. The pitch of fasteners shall be governed by the requirements for sealing.

See the NYSDOT Steel Construction Manual for minimum bolt spacing and edge distances.

Stitch bolts shall be used in mechanically fastened built up members where two or more plates or shapes are in contact. The pitch of these fasteners shall be as per NYSDOT LRFD Article 6.13.2.6.4 through 6.13.2.6.6.

8.6.3 Welding

8.6.3.1 Weld Sizes

Intermediate stiffener and connection plate welds to flanges and webs shall not exceed $\frac{5}{16}$ in, unless required by design. Longitudinal stiffener to web welds shall not exceed $\frac{5}{16}$ in, unless required by design.

The minimum flange to web fillet weld sizes shall be as per NYSDOT LRFD Article 6.13.3.4.

8.6.3.2 Weld Detailing

When complete joint penetration groove (CJP) welds are called for, the only information that should ordinarily be shown on the plans is “CJP” in the tail of the welding callout. The joint configuration should not be called out. This is the responsibility of the fabricator to select and show on the shop drawings. Special finishing and contour can be shown if required.

For T and corner joints designers shall show UT testing requirements on the contract plans.

Partial joint penetration groove (PJP) welds are used only in special circumstances. They should be used only after consultation with the Metals Engineering Unit. Transversely loaded partial joint penetration groove welds shall not be used except as permitted in LRFD Article 9.8.3.7.2.

Designers and detailers are referred to the American Institute of Steel Construction (AISC) Steel Construction Manual, the American Welding Society publication D1.5, and the NYSDOT Steel Construction Manual for information on the proper method of detailing welded joints.
8.6.4  Copes

Simple shear coped beam connections have a history of being vulnerable to fatigue cracking initiating at the cope, and should be avoided whenever possible. This is especially pertinent in floor beams and stringers of truss and thru girder spans. There are design situations, however, where coped connections cannot be avoided because of framing considerations. Two cases shall be considered for main/primary members:

Case 1  Cope depths < 6 inches:
   The minimum radius of the cope shall be 2 inches.

Case 2  Cope depths ≥ 6 inches:
   The minimum radius of the cope shall be 6 inches to reduce the stress concentration that may be present at a notch or tight radius cope.
   Cope depths greater than 6 inches shall be reinforced using a horizontal reinforcement plate welded on each side of the web within the limits of the cope. (See Figure 8.2)

Designers may contact the Metals Engineering Unit for specific guidance when this situation arises.

Figure 8.2 – Reinforced Cope Detail
8.6.5 Connection Design

Connections shall be designed as slip-critical connections. Slip-critical connections are required in primary members because they carry live load. Diaphragms and laterals in curved-girder bridges carry live load and are primary members. Diaphragms in straight-girder bridges are secondary members. The NYSDOT Steel Construction Manual allows the use of oversize holes for secondary members. Oversize holes cannot be used in bearing-type connections, therefore the connections must be designed as slip critical. Where floor beams are connected directly to stiffeners, knee braces or connection plates, the floor beams shall not be cope. The flanges shall be cut and chipped to provide a smooth faying surface as shown in Figure 8.3.

Article 6.13.1 of the NYSDOT LRFD Bridge Design Specifications states that the end connections of diaphragms and cross frames shall be designed for the calculated member loads. It is not necessary to design the end connections of diaphragms and cross frames for 75% of their shear or axial capacity.
8.7 **Stiffeners**

### 8.7.1 Bearing Stiffeners

Bearing stiffeners shall be a minimum of $\frac{3}{4}''$ thick and a minimum of 7" wide. Bearing stiffeners shall be placed parallel to the skew for skews $\leq$ 20 degrees, and normal to the web for skews $>20$ degrees.

Bearing stiffener welds shall be as shown on the current BD SG sheets.

The ends of all beams and girders and all bearing stiffeners shall be vertical after dead load deflection.

When two pairs of bearing stiffeners are used for very large reactions, the stiffeners must be placed a sufficient distance apart to permit access to weld the stiffeners to the web. The spacing between stiffeners should be at least equal to their width.

### 8.7.2 Intermediate Stiffeners and Connection Plates

Intermediate stiffeners shall be a minimum of $\frac{3}{8}''$ thick and 4" wide. Connection plates for straight girder cross frames and diaphragms shall be a minimum of $\frac{1}{2}''$ thick and 7" wide. Connection plates for curved girder cross frames and diaphragms shall be a minimum of $\frac{9}{16}''$ thick and 7" wide. Connection plates also serve as intermediate stiffeners.

Connection plates shall be placed parallel to the skew for skews $\leq$20 degrees, and normal to the web for skews $>20$ degrees. Transverse intermediate stiffeners that are not connection plates shall be placed normal to the web.

On fascia girders, intermediate stiffeners shall be placed on the side of the web which is not exposed to view. On interior girders, they shall be located on alternate sides of the web, except where they are used in conjunction with a longitudinal stiffener on the other side.

Intermediate stiffener welds shall be as shown on the current BD SG sheets.

Connection plate welds shall be as shown on the current BD SG sheets.

- When welding directly to the tension flange, designers shall limit the fatigue stress range to category C’.

### 8.7.3 Longitudinal Stiffeners

Use of longitudinal stiffeners should be avoided whenever possible.

Generally, longitudinal stiffeners shall be continuous for their entire length, with intermediate transverse stiffeners and connection plates cut short to avoid intersecting welds. Exceptions are when the longitudinal stiffener is interrupted by a field splice in the girder, or when the stiffener...
is no longer required by design. In these circumstances, the designer shall be responsible for providing the appropriate termination details that comply with the *NYSDOT Steel Construction Manual* on the contract plans.

When longitudinal stiffeners are required, show them placed on one side of the web only. On fascia girders they shall be placed on the web surface exposed to view. The intermediate transverse stiffeners, if necessary, shall be placed on the opposite side of the web. The longitudinal stiffeners shall be attached to the web plate with full-length, continuous, \( \frac{5}{16} \)" fillet welds. Fabrication details including transverse connection plate and longitudinal stiffener-intersection details shall be in accordance with the *NYSDOT Steel Construction Manual*.

### 8.8 Designation of Tension Zones

The Contract Plans shall clearly indicate the limits of tensile stress on each flange of all continuous steel girders. This will facilitate control of materials and welding inspection during fabrication and erection, as specified in the *NYSDOT Steel Construction Manual*. This requirement shall apply to reconstruction projects which require new deck slabs, as well as to new structures.

On continuous steel girder bridges a sufficiently accurate approximation of the point of combined load contraflexure may be obtained from moment diagrams alone. Using the moment tables shown on the plans, the designer can total dead load moment, superimposed dead load moment, and the appropriate live load moment at incremental points along the girder. The point where zero combined moment occurs can be found by interpolation. This point will reasonably represent the end of a tension zone and shall be shown as such on the plans.

If stress calculations are available, stresses may be used instead of moments. Designers need not calculate stresses for this purpose alone. The moment diagram method produces a conservative estimate of the tension zone limits. Stress calculations improve on this estimate by factoring in the effect of differing section moduli. However, actual loadings and section moduli may vary from the assumed values.

Where tension zones terminate less than 10 feet beyond the dead load point of contraflexure, the distance of 10 feet± shall be shown. The actual distance computed shall be shown for distances greater than 10 feet.

### 8.9 Camber

Design cambers include: structural steel dead load, concrete dead load, superimposed dead load, vertical curve, and total of the above. The dead load from a future wearing surface shall be included in the determination of camber. When cambers vary between girders due to differing concrete slab loads, concrete placement sequence, or stage construction issues, they shall be shown separately in the table.

A camber table and camber diagram shall be shown on the plans. See the current structural steel Bridge Detail (BD) Sheets for details.
If a steel member is designed with no camber, a note shall be placed on the plans instructing the fabricator to place the mill camber up.

8.9.1 Sag Camber

By definition, a girder is said to have sag (or negative) camber if any portion of the curve formed by the top of web in the completed structure falls below a working line constructed through the top of web points at the girder ends.

Note that all intermediate support points are ignored when applying the above definition. The designer's attention is directed to the fact that sag camber can be introduced into a girder from superstructure geometry other than from a sag vertical curve. These other conditions include any superstructure (straight or curved) in which a superelevation transition length occurs within the span, or a horizontally curved superstructure supported on straight girders.

Girders with sag cambers are to be avoided because their unstable appearance is aesthetically objectionable. An exception to this policy may be made when the under feature of the structure is a waterway. This exception recognizes a reduced concern for aesthetics.

Designers may find that approved geometrics for a bridge project have not considered the Office of Structures' policy regarding sag cambers. If this condition exists, the Designer shall use the following guidelines to minimize the effect or eliminate, when possible, designing a sag cambered superstructure.

1. Investigate the possibility of revising the geometrics (i.e., modifying or relocating the sag vertical curve and/or modifying or relocating the superelevation transition off the superstructure). In those cases where a deeper haunch is required, the 8” reinforced haunch should be used in conjunction with a sag camber.

2. If a revision of the geometrics is not possible, a variable haunch shall be introduced to eliminate the need for the sag camber. The depth of haunch for this purpose shall be limited to a nominal 8”.

8.10 Moment, Shear and Design Load Tables

A table showing moment, shear, and design loads shall be provided on the plans. See the current structural steel BD sheets for details. Moments and shears shall be given at the same intervals as the camber table. Moments and shears for AASHTO HL 93 and the NYSDOT Design Permit Vehicle need to be shown separately.
8.11 Splices

8.11.1 Girder Splices

Girder details for all LRFD projects with spans of more than 140 feet or where a splice is otherwise required shall be prepared with field splice locations and splice design details shown on the plans. Details and location access constraints control the erection procedure. However, designers must always assure themselves that girders can be field spliced following the criteria shown in this section.

In the design of long stringers and girders, simple or continuous, straight or curved, consideration should be given to the need for field splices. Bolted field splices are preferred over welded field splices, because of substantial savings in time and money. Fill plates are not allowed.

Except for those cases where it is obvious that no field splice will be required (span lengths less than 130 feet for straight or large radius curved members), the flanges should have sufficient excess area at points where splicing is anticipated to permit a bolted splice to be made.

Splice locations are generally selected near points of dead load contraflexure and where there is sufficient flange area to permit hole drilling while still maintaining the required net area.

DESIGN

General Practice

For simple spans or continuous spans where the total girder is less than 140 feet in length, the girder may be assumed to be erected as a single segment and no splice design will be necessary.

For simple spans greater than 140 feet in length, the preferred location for the splice, based on load considerations only, is at the one-third point.

For continuous spans greater than 140 feet in length, the preferred location for the splice, based on load considerations only, is near the dead load contraflexure point. Note that on longer structures the points of dead load contraflexure can be greater than 140 feet apart, in which case the preferred locations would be where the size of the splice and number of bolts is minimized.
Additional constraints on splice location include the following:

- The minimum distance from a flange plate transition groove weld to the nearest flange splice bolt hole or lateral gusset plate bolt hole is 12 inches.
- The centerline of field splice shall be located >5 feet from a flange plate transition groove weld.
- The minimum distance from a lateral gusset plate to the end of a flange splice plate is 6 inches.
- The minimum distance from a stiffener or connection plate to the end of a flange splice plate is 12 inches.
- The minimum distance from a stiffener or connection plate to a groove welded splice in either the flange or web is 6 inches.

As is current practice, the compression flange must be designed considering the steel dead load acting on the unbraced length (before diaphragms are attached). Refer to Section 8.4.2.5 for requirements for stability of the structural steel during transportation and erection.

It is preferable to group the design of the splices at any splice location by designing all splices using the heaviest section or greatest moment rather than vary the splice designs across the structure. This avoids confusion and possible construction problems, and should provide the most economical solution. In addition, it is preferable to have one design for all splice locations rather than having a different design at each splice point.

**Vertical Clearance**

When locating the splice, the designer shall consider the effect of the splice on vertical clearance. Vertical clearance at the splice location will be reduced by the bottom flange splice plate, washer, nut and free end of bolt (see AISC table titled "Entering and Tightening Clearances"). If the splice affects minimum or critical vertical clearance, the designer shall show the revised minimum or critical vertical clearance on the plans. Vertical clearance issues may control the location of splices.

**Erection**

Erected and spliced segments must be statically stable. Depending on the span arrangement, this may require the use of falsework or splicing of the girder on the ground. Note that when a girder is spliced on the ground the unbraced compression flange length may increase. The girder must be stable during all phases of erection and construction.

Structures which are difficult to erect (e.g., tub girders, long simple spans) should show a suggested method of steel erection in the Contract Plans. This is required because the Contractor is responsible only for additional stresses caused by their erection scheme, and the Contractor may assume the simplest erection method possible if none is shown on the plans.
Falsework

A generalized falsework schematic should be shown on the plans when it is required for stability of the compression flange or stability of the structure. When falsework is required, the designer must get approvals from the appropriate agencies. The Rail Unit, Real Estate or Highway Design (for Temporary Traffic Control) may typically need to be contacted. Railroads will not allow falsework within the track zone and also may not allow any splices above the tracks. Maintenance and Protection of Traffic issues may also control the location or use of falsework. Design of the falsework is the responsibility of the Contractor, subject to the approval of the D.C.E.S.

Shipping

The maximum shipping length is 140 feet based on permitting and geometric limitations. The maximum girder depth is typically 14.0 feet, although depths up to 16 feet may be used in special circumstances with the approval of the Metals Engineering Unit. The issue of special hauling permits is typically handled by the fabricator and is controlled by weight of the girder segment and the configuration of the truck and trailer used. The maximum shipping weight of a segment is 100 tons.

Cranes

For typical structures, the designer may assume the maximum single crane pick is 100 tons. Nearly all structures constructed for the Department are erected by a single crane of this type. For structures which require larger or multiple cranes to erect, contact the Metals Engineering Unit for assistance. When splicing needs to be done before erection it should be noted on the plans so the Contractor is aware of the possible need for a larger (or multiple) crane(s) at bidding.

Additional Items

A High-Performance-Steel simple span may be long enough to require the use of two field splices.

Falsework up to 16 feet in height may be assumed to cost $5,000 per location for typical 40 to 50 foot wide structures. It is preferable to avoid the cost of these temporary structures and strengthen the compression flanges if the cost is similar.

Fracture-Critical Members shall have splice plates constructed from Fracture-Critical material.

Design Calculations

Bolted designs shall use ASTM A325 bolts only. Bolts should be designed as per the NYSDOT LRFD Bridge Design Specifications and the NYSDOT Steel Construction Manual (SCM). Bolts must be designed both for strength and for slip-critical loading using Class A surface conditions unless otherwise approved by the D.C.E.S. Bolt lengths shall be such that threads are excluded from the shear planes in the connection. Designers should reference NYSDOT SCM-Section 2 on bolting and splices (including fill plates, as appropriate). Use 7/8 inch bolts for typical girder splices. Unusual structures may require a larger bolt size.
Refer to the American Institute of Steel Construction Table titled “Entering and Tightening Clearances” and to Section 8.6.2 of this manual for a discussion of bolted connections.

Computer Programs

AISIsplice is the recommended program for splice design. For questions involving this program contact the Structures IT Systems Unit or Metals Engineering Unit.

AISIsplice has the following limitations:

- It is limited to straight steel I-girders.
- It will not design hybrid splices, girders must be homogeneous.
- Flanges should be parallel at the location of the splice (do not locate the splice at a location the web depth is varying).
- Bolt patterns are limited to constant pitch, nonstaggered patterns.
- The program designs only symmetric splices, which may not be the most cost effective.
- The program may calculate section properties of the concrete deck slab incorrectly when the top flange of the girder is embedded into the slab.

Currently the department has no software which can design curved girder, tub girder, hybrid or box section splices. Contact the D.C.E.S. when designing splices for these types of girders.

Estimate

The splices should be paid for under the appropriate items. No additional weight calculations are necessary for typical structures, as the typical 3% accounts for the splice plates and bolts.

8.11.2 Rolled Beam Splices

When rolled beams are used for continuous structures, the field splices should be located in areas where no cover plates are required and consideration should be given to the fact that the fatigue strength of the section adjacent to the bolted connection (Category B*) is less than the fatigue strength of the base metal in areas where there is no splice (Category A*).

* See Article 6.6.1.2 of the NYSDOT LRFD Bridge Design Specifications or Article 10.3.1 of the NYSDOT Standard Specifications for Highway Bridges.

8.12 Framing Plans

Typical framing plans for steel structures are shown in the current structural steel BD sheets. Diaphragms shall be placed parallel to the skew angle for skews 20° and less. Diaphragms shall be placed perpendicular to the girders for skews over 20°.
8.13 **Curved Girders**

On curved girder projects with radii less than 600 feet it is important to coordinate with the Metals Engineering Unit early in the design phase to assure that special fabrication and erection concerns are addressed.

Diaphragms in curved girder structures are primary members and designed to carry dead and live load. Except for end diaphragms they should be placed radial to the girder in a single line across the bridge. A diaphragm should not be placed along the line of support at an interior skewed support. Curved girders have special diaphragm and lateral details that are shown on the current structural steel BD sheets.

Curved girders that are designed as straight girders because their curvature does not exceed the limitation contained in the *NYSDOT LRFD Bridge Design Specifications* still need special provisions for design and detailing. These girders must also use the diaphragm and lateral connections details for curved-girder bridges.

8.14 **Trusses**

It is important to coordinate with the Metals Engineering Unit of the Office of Structures early in the design phase of a truss project to assure that fabrication concerns are addressed.

8.14.1 **General Considerations**

Trusses are a viable structural form when there are clearance restrictions on beam depth that would preclude the use of girder spans. Trusses also become an economic option when span lengths are long enough to make plate girders impractical. Trusses are a very efficient structural form in the use of material, however their complex fabrication tends to make them costly. They are also usually nonredundant structures which leads to special design considerations.

A modified Warren truss (incorporates verticals) is usually appropriate for most highway bridge applications, although other truss forms can be considered.

Skewed trusses should be avoided if possible. The skew makes fabrication difficult and costly and introduces out of plane bending problems to the structure. Small skew angles can often be eliminated by a small increase in the span length.

End portals and sway bracing should be placed a minimum of 16’-6” clear above the roadway surface (includes usable shoulder), regardless of minimum vertical clearance requirements for that highway classification.

It is desirable to keep sidewalks inside the trusses rather than placing them on outside cantilevers. A vertical faced concrete parapet should be used between a sidewalk and the truss. This provides more lateral stability to the structure and keeps traffic and road salts away from critical members. Adequate clearance should be maintained between the concrete barrier or parapet and the truss to accommodate formwork.
When a metal railing system is used on bridge rehabilitation projects with concrete decks, it is preferred that the system be anchored in the deck and not attached to the truss elements. Consideration should be given to providing a clear zone to accommodate lateral deflection of the railing system.

Weathering steel is recommended for trusses because of its superior toughness. See Section 8.2.3 for painting guidelines. Galvanized steel may also be an option for trusses.

8.14.2 Truss Design Guidelines

Geometry:

Truss and member proportions should follow the guidelines provided in the NYSDOT LRFD Bridge Design Specifications or the NYSDOT Standard Specifications for Highway Bridges.

Sections:

Designers should keep variations in member shapes and sizes to a minimum. To achieve this objective, it is often desirable to establish a constant out-to-out dimension for all chord members. Based on past experience, it is frequently more cost effective to use fabricated members than rolled sections because of their tighter tolerances. Rolled sections may vary for “tilt” and “in-out” by more than $\frac{3}{16}$", and sometimes require further work to bring them into the necessary tolerances.

Designers should use closed box sections for bottom chords whenever possible. Although closed box sections are more expensive to fabricate, they eliminate the long term maintenance and durability concerns associated with H-shaped sections. H-shape sections tend to trap debris and moisture.

Framing:

The floor system framing of trusses should be designed as simply supported although it is recognized that some negative end moments can and probably will develop. This should be considered when designing fatigue resistant details.

Stringers should be framed from floorbeam to floorbeam. Stringers that run continuously over the tops of floorbeams have led to uplift and fatigue problems. Additionally, consideration should be given to framing stringers below the plane of the floorbeam top flange to eliminate the cope at the top of the stringer.

Internal Diaphragms:

Designers shall include internal diaphragms within fabricated closed box chord sections. These diaphragms are to be located at panel points, and elsewhere where required by design.

Camber:

Because the steel fabrication industry prefers assembling trusses in a fully cambered position, (i.e.: member lengths adjusted for deadload and vertical curve cambers), designers are advised
to evaluate the secondary force effects which will arise when the truss is fabricated in this fashion. It should be noted however, that these secondary force effects are generally minor when the truss proportions follow the guidelines provided in the NYSDOT LRFD Bridge Design Specifications or the NYSDOT Standard Specifications for Highway Bridges.

**Gusset Plates**

Design guidance for gusset plates can be found in Structure Design Advisory 08-001 (LRFD) and load rating guidance can be found in Technical Advisory 09-001 (LFD)

**8.14.3 Truss Detailing Guidelines**

Floor beam to truss connections should be blocked and never coped.

Details should be used that allow accessibility to make field bolted connections. Hand holes in the bottoms of closed box sections will be needed for erection purposes. These holes shall be protected with screening to prevent roosting birds from entering.

Details that allow accessibility for cleaning and high pressure washing are desirable.

Fill plates in bolted connections are sometimes necessary. Fillers greater than or equal to ¼" thick shall be designed in accordance with Section 6.13.6.1.5 of the NYSDOT LRFD Bridge Design Specifications.

Use Category “C” or better welded fatigue details on all fracture critical members.

Internal diaphragms on closed box sections should be detailed as being fillet welded to three sides, and tight fit to the fourth.

Designers shall include the following information on the contract plans, to facilitate the quality assurance review of the steel fabrication drawings:

- Table of Fracture Critical Members
- Table of LRFD Member Forces: DC1, DC2, DW, LL + Impact (AASHTO HL-93 and NYSDOT Permit Vehicle)
- Truss Camber Diagram: Provide the lengths members must be lengthened or shortened to compensate for dead load and vertical camber. Dimensions provided should include total unfactored deadload (DC1 + DC2 + DW) and vertical curve camber.
- Truss Working Lines Diagram: Provide member lengths (with horizontal components adjusted for grades greater than 3%), and offsets to datum for grade.

**8.15 Miscellaneous Details**

**8.15.1 Bolsters**

Bolsters are steel supports placed beneath the girder and above the bearing. They are typically used at piers when two spans have different depths. In new construction it is almost always
preferable to step the concrete of the cap beam or pedestal instead of using bolsters. For aesthetic reasons it may be appropriate to investigate alternative designs that would not have adjacent spans with different girder depths. (See Section 23.)

When bolsters are used, they must be carefully designed and detailed. Two types of bolsters are available, based on their aspect ratios.

- Low bolster, $A/B < 1$ Use rolled section, See Figure 8.4
- High bolster, $A/B \geq 1$ Use fabricated section, See figure 8.5

Bearing stiffeners on bolsters should meet the same design, detailing and fabrication requirements as bearing stiffeners on girders.

Note: weld sizes shown are minimums

**Figure 8.4**
Low Bolster Detail and Section A-A
Bolsters shall be paid for separately under Item 564.70, Structural Steel Replacement. They are not included in the bearing item in order to assure that the steel fabrication is performed in the proper manner.

### 8.15.2 Safety Handrail

Safety handrails for use during bridge inspections shall be used on girders having a web depth of 5 feet or greater. They should be used on both sides of interior girders and on the inside of fascia girders. Details of field-erected and shop-erected handrails are available on current BD sheets. Cost of handrails shall be included in the unit prices bid for the structural steel.
8.16 Railroad Structures

8.16.1 General Considerations

Railroad structures are commonly 2 or 3 girder structures that contain fracture critical elements.

Contract plans shall include:

- A listing of all primary/main members.
- Tension zones defined for floor-beams and girders
- A table of all fracture critical members.

8.16.2 Design

Design of railroad structures shall be in accordance with current A.R.E.M.A. specifications.

8.16.3 Details

The purpose of knee brackets is to brace the compression flange of through girders and support the ballast curb plate. The flanges of the knee brackets should not be interrupted by notching to accommodate the curb plates. Although this will cause the cover plates to be installed in multiple segments, the integrity of the knee bracket outweighs the ease of installation issue.

Curb plates should be notched to fit around stiffeners and girder web attachments as needed. The curb and cover plate needs to be contiguous to protect the membrane system. Curb plates shall be bolted to knee braces and the girder web using clip angles. Welding should only be considered where access is a problem. Unless alternatives are impractical, curb plates should not be welded to the intermediate stiffeners.

The deck plate may be welded to the curb plate. The knee bracket must be cut short to allow for the attachment of the curb plate to the deck plate. The deck plate needs to be installed under the knee bracket during construction. This leaves a gap underneath the knee bracket to allow the deck plate to be installed. The curb plate is configured to have a v-groove joint at the junction of the curb and deck that can be welded with a partial penetration groove weld in the field.

8.17 Movable Bridges

Design of projects of this complexity requires special consideration. Early involvement with the Metals Engineering Unit is highly recommended.

A very different set of criterion must be followed on moveable structures, such as bascule or post-lift bridges. Specifically the nondestructive testing requirements for the machine parts, etc., for the electrical and mechanical portions of the bridge must be clearly defined on the contract plans. Additionally there may be stair wells, hatches, and other appurtenances that should be detailed and shown with the proper steel payment item on the contract plans.
Contract plans should also include:

- Identification of main members and/or tension components
- Identification of the tension or reversal zones.
- Listing of fracture critical members or members that must meet minimum toughness (CVN) requirements i.e. bascule lateral bracing or edge beams.
- Special Non-destructive testing requirements.

Designers should consult the AASHTO LRFD Movable Highway Bridge Design Specifications.

8.18 Pedestrian Bridges

8.18.1 General

Pedestrian bridges may be detailed as I-beams, box girders or a prefabricated truss. I-beam or box girder pedestrian bridges should be completely designed and detailed in the contract plans. Prefabricated truss pedestrian bridges require a different approach because they are designed by the manufacturer after the contract has been awarded. See Section 2.6.4 for loading requirements.

The contract documents must provide sufficient details so the manufacturer can supply the intended type of structure. Discussions with the owner should include any project specific aesthetic or architectural treatments required. The Regional Landscape Group can provide guidance on aesthetic or architectural treatments choices and any special requirements from the Americans with Disability Act (ADA).

While the designer must provide the manufacturer with enough information so that an accurate bid can be prepared, the designer must also recognize that unnecessary restrictions may result in excessively high bids. Only specify those specific requirements absolutely necessary for the bridge to meet the project’s safety, aesthetic and structural requirements.

8.18.2 Design Guidelines

The designer should follow the considerations and guidelines for trusses in Section 8.14. While Section 8.14 is intended for highway trusses, designer should review it for application to pedestrian trusses.

Skewed supports should be avoided for prefabricated trusses. Grades greater than 5% should be avoided on pedestrian bridges because it involves additional ADA requirements.
8.18.3 Detailing Guidelines

If a particular type of truss is required it must be clearly indicated in the contract plans. The truss types typically used are Warren, Pratt, Bowstring and Howe. Designers should indicate all acceptable truss types in the contract documents. For additional information see the following section on truss member styles.

Deck joint and accessory details should be indicated but full details should not be shown, as they are included as part of the proprietary bridge superstructure.

Bearings need to be shown on the plans but not detailed because they are designed by the contractor’s engineer in accordance with the pedestrian bridge specification.

Truss Member Style:

Top chords: The top chords may be sloped or horizontal. Trusses with sloping top chords or lenticular configurations are often preferred based on aesthetic considerations.

Verticals: The locations of vertical members must be indicated on the contract plans, if required.

Overhead/portal bracing: Overhead/portal bracing details must be shown if allowed or required. If overhead/portal bracing details are not allowed that must also be clearly indicated on the plans. If overhead bracing is allowed vertical clearance requirements shall be shown. Standard vertical clearances can be found in Section 2.

Camber:

The required camber shall be indicated in the contract plans for non-prefabricated bridges.

For prefabricated bridges the camber is the responsibility of the fabricator. However, the designer should indicate desired final appearance of bottom chord (e.g., flat or follow profile) for aesthetic reasons.

Finish:

The required finish of the steel shall be indicated on the contract plans. Finish options include painted, weathered, or galvanized steel. When steel is to be painted, the required color must be indicated in the contract documents.

Weathering steel should be specified in accordance with the guidelines provided in Section 8.2.1. Weathering steel tubes shall not be specified when the bridge is expected to remain open during the winter months and will be salted.
Deck Type:

The type and specific details of decking are required on the contract plans. The designer may choose from many types of decking including concrete, timber (glulam), steel grating, fiber reinforced polymer (FRP), or plastic (pvc or composite). Timber shall not be placed directly on weathering steel.

The recommended minimum depth for a concrete deck is 6”.

Width Requirements/Guidelines:

Horizontal clearance requirements shall be indicated on the contract plans, i.e. clear width between trusses/railing/curbing etc.

Deck widths should generally be greater than span/22 when overhead bracing is not allowed and span/30 when overhead bracing is allowed.

A minimum clear width of 10 feet is recommended for passage of emergency vehicles.

Deck widths for prefabricated structures greater than 14 feet should be avoided, as they require a longitudinal splice in the bridge for shipment.

Railing/Protective System:

Indicate railing height, material type (steel tube or wood) and type of finish.

Provide details for Horizontal safety rails, vertical pickets and protective fencing if required. When fencing is required provide: height, type (galvanized or epoxy-coated), color and maximum opening.

Indicate minimum 5” toe rails located no more than 2” above the deck.

Provide ADA compliant handrail when grade exceeds 5%.

Indicate end treatment and approach railing types.

Truss Accessories:

Bollards, ramps, stairs, lighting, signing, and utility hangers are examples of items that can be provided on a pedestrian bridge. Specific details that are required must be shown on the contract plans and the item(s) under which they are to be paid must be indicated.

Bollards should be used to limit vehicular traffic.
Section 9
Prestressed Concrete

9.1 Introduction

Concrete is approximately ten times stronger in compression than in tension. Typical reinforced concrete design assumes that concrete under tensile loads will crack, and steel reinforcing bars are used to carry the tensile forces. Prestressed concrete design, on the other hand, applies compressive force to the anticipated tension zones of the concrete member by using pretensioned or post-tensioned, high-strength steel strands. When properly designed, tension in the member under service loads is reduced or eliminated and concrete cracking is reduced.

Precast concrete members are especially advantageous in situations where quick erection is desired. Precast concrete members are fabricated year-round and can be delivered, erected, and put into service in a very short time. All prestressed concrete beams are produced using high strength, high performance concrete. The corrosion resistance of the prestressed beams is further enhanced by the addition of 5 gal/cubic yd. of 30% calcium nitrite corrosion inhibitor and two coats of silane sealers. These beams are expected to provide long, maintenance-free service.

A number of prestressed concrete bridge types are used in New York. Although adjacent box beams are the most commonly used, designs using I-beams and bulb-tee sections are also becoming common.

9.1.1 Pretensioning

Pretensioning a concrete member is accomplished by tensioning prestressing strands to the required tensile stress using external jacks and anchors, casting the concrete member around the tensioned strands and, releasing the external strand anchors after the concrete has achieved the required minimum strength. Precompression is induced by the transfer of force through the bond between the prestressing strands and concrete.

9.1.2 Post-Tensioning

Post-tensioning a concrete member is accomplished by tensioning unbonded prestressing strands using an external jack on one end of the member and an anchor placed directly against the hardened concrete on the other end. The strands are typically internal to the member, but may be placed externally. A second anchor is secured against the member and the jacking force is released to transfer the load into the member as a precompression force.
9.2 Adjacent Prestressed Units

The typical three types of prestressed members used for adjacent prestressed unit superstructures are solid slab units, voided slab units, and box beams. The design concept for these types of units is identical and the only differences between them are the member depth, shape of the voids (if any), and the casting procedure.

Adjacent prestressed concrete slab units and box beams are especially appropriate at stream crossings having limited freeboard because they provide a continuous flat surface along the bottom of the superstructure that prevents debris from becoming trapped under the bridge and impeding the hydraulic flow. In addition, their relatively shallow depth provides greater clearance than spread beam types of superstructures.

However, because interior deterioration can not be observed by a visual inspection and they are difficult to repair the use of adjacent prestressed concrete slab units and box beams should be limited to situations where they are essential. For example: stream crossings having limited freeboard.

Type F NEXT beams and Deck Bulb Tee girders can also be placed adjacent and should have either a composite slab or asphalt overlay for a riding surface.

9.2.1 Unit Width

Standard box beam and slab units are available in widths of 4 ft. and 3 ft. Designs that use the fewest number of beams for a given superstructure will achieve the greatest economy in fabrication, shipping, and erection costs. Therefore, even if it results in a wider superstructure than is actually required, an adjacent precast concrete unit superstructure should be made exclusively out of 4 ft. wide units. A combination of 4 ft. and 3 ft. wide units may be selected if the required construction staging sequence or other constraint prevents the exclusive use of 4 ft. units. Configurations involving a single 3 ft. unit mixed with 4 ft. units are inefficient to fabricate and should be avoided. The overall beam deck width shall be shown on the contract plans.

4 ft. wide units should be used for the fascia beams to provide adequate space for the placement of the bottom railing anchor plates or concrete barrier reinforcing bars. This is especially important for alignments requiring curved railing or barrier.

Type F NEXT beams are available in widths between 8’-0’ and 12’-0”.

9.2.2 Unit Depth

Typical prestressed sections are shown on the BD sheets. For multi-span bridges, a constant unit depth is preferable across all of the spans since variable depth units are difficult and expensive to construct. BD sheets for NEXT beams are currently under development. Designers should contact the Concrete Engineering Unit for recommended details until those BD sheets are adopted.
9.2.3 Deck Overhangs

Overhangs on the reinforced deck of adjacent prestressed slab units and box beams shall be a minimum of 4" and a maximum of 6". Overhangs less than 4” require approval of the D.C.E.S. Overhangs greater than 6” are not allowed. The bottom of the overhang shall slope to drain away from the beam so that chloride-laden runoff water will not run down the side of the beam.

Overhangs for Type F NEXT beams shall be limited to 3'-6”.

9.2.4 Longitudinal Joints

The standard longitudinal joint size between adjacent prestressed slab units and box beams shall be a minimum of ¾" and a maximum of 1 ¾" at the bottom. The use of larger joints requires approval of the D.C.E.S. Joints between stages of stage construction shall follow the details shown on the BD sheets.

The standard longitudinal joint size between adjacent Type F NEXT beams shall be ½” at the top of the beam.

9.2.5 Skew

The designer should make every reasonable effort to reduce or eliminate bridge skew. This may require early discussions with highway design personnel. The maximum allowable skew angle for a bridge using box or slab units is 50°. Larger skews require approval of the D.C.E.S.

The maximum allowable skew angle for a bridge using Type F NEXT beams is 15°. Larger skews require approval of the D.C.E.S.

9.2.6 Diaphragms and Transverse Tendons

Internal diaphragms in adjacent precast concrete slab units and box beams shall be positioned parallel to the skew and have a minimum width of 1'-2‖. Transverse tendons shall also be placed parallel to the skew of the unit and be placed as close to the middepth of the section as possible. Each transverse tendon consists of three ½" diameter low relaxation strands tensioned to 28,000 lb. per strand. Transverse tendons are tensioned after the shear keys have been grouted and before the deck slab has been placed.

Internal diaphragms and transverse tendons within precast units shall be spaced as follows:

- For span lengths less than 50 ft., a total of three transverse tendon locations are required. One group of tendons is located at each end of the unit approximately 7” from the centerline of bearings and another group of tendons is located at the centerline of the span.
For spans greater than or equal to 50 ft., a total of five tendon locations are required: one group of tendons at each end approximately 7” from the centerline of bearings, one group of tendons at the centerline of the span, and one group of tendons midway between each end group and the centerline of the span.

For stage construction placing of transverse tendons and diaphragms shall be as shown on the appropriate BD sheets.

The transverse tendon holes in all units and the transverse tendon blockout on the fascia units should be checked to ensure that they do not interfere with either the longitudinal prestressing strands or bar reinforcement.

NEXT beams do not require any diaphragms or transverse tendons.

**9.3 Spread Precast Concrete Beam Superstructures**

Although a spread precast concrete beam superstructure requires a thicker concrete deck with heavier reinforcement and the necessary form work for the deck placements, the reduced number of beams used per span may prove economical for spans up to 100 ft. In addition, bridge superstructures utilizing spread concrete beams have some advantages over adjacent precast concrete unit superstructures:

- Spread beams have open bays to accommodate utilities when required.
- Spread beams can accommodate field adjustments due to variations in camber and/or camber growth, especially for staged construction.
- Spread beams are better suited to handle large deck cross slopes and curved alignments.

**9.3.1 Spread Prestressed Box Beams**

The provisions of Section 9.2 of this manual shall apply except as specifically noted below:

- Only 4 ft. wide box units should be used. Alternate widths require approval by the D.C.E.S.
- For spread box units, external diaphragms shall be placed as follows:
  
  End diaphragms are always required.

  No intermediate diaphragms are required for spans up to 65 ft.

  One intermediate diaphragm is required at midspan in spans over 65 feet and up to 100 feet. The external and internal diaphragms at midspan must be on the same line.

  It is not anticipated that spread boxes will be used over 100 feet. I-girders should be used for spans over 100 feet.

- Contract plans and beam details shall show provision for attaching reinforcement in cast-in-place or precast concrete diaphragms to the spread box beams.
9.3.2 Prestressed I-Girders

The preferred I-Girder shape is the Bulb-Tee.

- The framing plan for prestressed I-girders shall be as shown on the appropriate BD sheets. Contract plans shall normally show only galvanized steel diaphragms. The contractor shall be allowed to substitute cast-in-place diaphragms or precast concrete diaphragms as options. Occasionally, the designer may require concrete diaphragms because of site specific conditions. The cost of diaphragms shall be included in the cost of the beams. All inserts for diaphragm connections adjacent to a deck joint shall be stainless steel.

- No intermediate diaphragms are required for spans up to 65 ft. Midspan diaphragms are required for spans greater than 65 ft., and up to 100 ft. Spans greater than 100 ft. require diaphragms at the third points.

- For superstructures with cross slope greater than 4%, AASHTO I-beams should be considered. These shapes have narrower top flanges, which will eliminate the need for large haunches. Bulb-Tee beams may also be used by reducing the top flange down to three feet in width.

9.3.3 Type D NEXT Beams

Type D NEXT Beams are formed with a full structural deck section composite with the two concrete tee beams and require a closure pour between beams. The actual closure pour width depends upon the closure pour material. The closure pour utilizes Ultra High Performance Concrete (UHPC) or HP concrete between the Type D NEXT Beam sections since there are no transverse tendons.

Type D NEXT beams are available in widths between 8'-0’ and 10'-0”.

NEXT beams do not require any diaphragms or transverse tendons.

The maximum allowable skew angle for a bridge using Type D NEXT beams is 30°. Larger skews require approval of the D.C.E.S.

Overhangs for Type D NEXT beams shall be limited to 3'-6”.

BD sheets for NEXT beams are currently under development. Designers should contact the Concrete Engineering Unit for recommended details until those BD sheets are adopted.
9.4 Segmental Precast Box Girders

Segmental precast box girder superstructures may be viable and economical alternates for the following type of structures:

9.4.1 Long Multi-Span Bridges

Segmental precast box girders are well suited for long multi-span bridges on straight or slightly curved alignments in locations where Temporary Traffic Control issues and/or environmental concerns require that field work be minimized. Repeated use of an erection set up for the box girder segments is the main advantage. The Span-by-Span method of erection is generally used for these bridges.

9.4.2 Long Span Bridge on High Curvatures

Segmental precast box girders are well suited to accommodate high curvatures on long spans due to their high torsional stability. The balanced cantilever method of erection is generally used for these bridges.

9.4.3 Aesthetics

When long open spans with clean visual lines are desired, segmental precast box girder superstructures are a good solution. Haunching of the segmental girders to improve the visual impact and structural efficiency is possible with this type of superstructure.

9.4.4 Durability

The expected durability of segmental box girder bridges is relatively high. Segmental precast box girder bridges utilize post-tensioning in both the longitudinal and transverse directions to be free of tensile cracks. This results in an expected substantial increase in the durability of the overall structure. However, there are areas of vulnerability unique to this type of bridge.

1. Since the deck slab is an integral part of the box girder system, the complete replacement of the bridge deck is nearly impossible. To reduce this risk, the structure should be designed so there is no tensile stress at the top surface of the segment under service load conditions both including and excluding time dependent effects.

2. Deck run-off should not be allowed to flow over the grouted block-outs for tendon anchorages. When end anchorages are located in vulnerable areas, such as beneath a deck expansion joint, additional protective measures shall be provided. Post-tensioning ducts within the deck shall be polyethylene. Fabrication and erection of these structures shall be as per the *Prestressed Concrete Construction Manual (PCCM)*.
9.5 **Bearings for Prestressed Concrete Structures**

All new prestressed concrete superstructure designs, with the exception of those using integral abutments, require elastomeric bearings of sufficient thickness to ensure that the bottom of the prestressed unit will be above the bridge seat a minimum of $\frac{3}{8}''$ for box beams and $\frac{1}{2}''$ for slab units. Cement mortar pads shall not be placed under the bearings.

For rehabilitation projects that require mortar pad replacement, the designer should choose one of the following alternatives:

- Replace the existing elastomeric bearings and mortar pads with thicker elastomeric bearings.
- Replace the mortar pad with a galvanized steel plate of equivalent thickness.
- Step the bridge seat or pedestal to an elevation sufficient to provide the necessary clearance (This option will normally require the use of Class DP Concrete, as specified in Section 582 of the *NYSDOT Standard Specifications for Construction and Materials*).

When choosing an appropriate alternative, the designer should strive for the most cost effective solution.

Bearings must be placed perpendicular to the centerline of the unit. The bearing width, at a minimum, must be $\frac{1}{2}$ the width of the unit measured perpendicular to the centerline of the unit.

When the height difference across the width of the bearing due to camber and grade is in excess of the limitations set in the design specifications, then a tapered bearing (for adjacent box or slab units) or a constant thickness bearing with a tapered sole plate (for Bulb Tees and AASHTO I-beams) matching the required slope must be used.

9.6 **Concrete Strength**

High-Performance Concrete shall be the standard concrete for prestressed bridge elements. The minimum concrete strength $f'_{c}$ for prestressed concrete bridge beams shall be 10 ksi. The concrete strength at transfer $f'_{ci}$ can be taken as $0.7f'_{c}$ unless the designer determines a higher transfer strength is necessary.

9.7 **Prestressing Strand Type**

Only 270ksi Low-Relaxation Prestressing Steel Strand shall be used. The standard diameter used by NYSDOT is 0.6 inch. Other diameters are available, but may only be used with approval of the D.C.E.S. Strength requirements and areas for the strand are available in ASTM A416.
9.8 Strand Pattern for Pretensioned Elements

9.8.1 Precast Box and Slab Units

A 2” x 2” center to center grid pattern shall be used for the prestressing strands in prestressed concrete beams. Strands shall not be placed within 2” of the centerline of the beam to provide room for the anchor dowel holes at the end of the beam. Strands shall not be placed such that they will conflict with the transverse tendons or tendon recesses. For additional information, see the appropriate BD sheet.

Prestressing strands shall be distributed evenly across a row to achieve uniform pretensioning in the end zones. Clustering of strands in the bottom corners of beams should be avoided as the uneven stresses can cause distortions to the beam. This is especially critical in longer beams with large skews.

9.8.2 Precast I-Girders, Bulb Tees and NEXT Beams

Prestressing strands are arranged in a 2” x 2” grid pattern as shown on the appropriate BD sheet. Prestressing strands shall be distributed evenly across a row to achieve uniform pretensioning in the end zones.

9.9 Tensile Stresses Due to Pretensioning

If higher than allowable tensile stresses are encountered during the design of prestressed members (usually at the top surface of the beam ends) the following design modifications are suggested in the order of preference:

1. Rearrange the strand pattern, including addition of strands near the surface exhibiting excessive tension. In general, four fully tensioned strands is a reasonable maximum number of strands to be placed near the tension surface for slab units. For box units, six is a reasonable maximum. For Bulb Tees and AASHTO I-Beams, 20% of the total number of strands (not including draped strands) is a reasonable maximum. In all cases, engineering judgment is required.

2. Drape strands for I-Girders (Bulb Tees and AASHTO I-beams).
   
   Note: Where draped strands are used, the total hold down force of all draped strands shall not exceed 75% of the total beam weight.
   
   Note: Prestressing strands in slab units or box units shall not be draped.

3. Debond some prestressing strands at the end of the unit to avoid excessive end stresses. Typically, this is accomplished in the fabrication plant by wrapping strand with a plastic sheath to prevent the bond from developing between the concrete and the prestressing strand.
When debonding of prestressing strands is required, design shall be in accordance with the *NYSDOT LRFD Bridge Design Specifications* with the following criteria:

a. The maximum allowable number of debonded prestressing strands is 25% of the total number of strands.
b. No more than 40% of the number of prestressing strands in any one row may be debonded.
c. The debonding pattern shall be symmetrical about the beam centerline.
d. The spacing of debonded strands shall be a minimum of 4”.
e. The outermost prestressing strands in a row shall not be debonded.
f. The debonded length(s) shall be clearly detailed on the contract plans. A maximum of four prestressing strands are permitted to be debonded for a given length. A minimum difference of 2'-0" is required between debonding lengths.
g. Do not debond prestressing strands in units 1'-3" or less in depth.

4. Provide a reasonable amount of bonded reinforcement as per the provisions of the design specifications.

### 9.10 Prestress Losses

Loss of prestress is the difference between the initial tensile stress in prestressing tendons at the time the strands were seated in their anchorages, and the effective prestress at a particular time at the considered location.

Losses that apply to both pretensioned and post-tensioned elements are Concrete Shrinkage, Elastic Shortening, Concrete Creep, and Steel Relaxation. Losses that apply only to post-tensioned elements are Anchorage Set and Friction (for drape and wobble). Computation of the losses shall be as per the applicable provisions of the design specifications.

**Concrete Shrinkage** - Shrinkage, after hardening of concrete, is the decrease with time of concrete volume. The decrease is due to changes in the moisture content of the concrete and physical-chemical changes, which occur without stresses attributable to actions external to the concrete. Shrinkage is conveniently expressed as a dimensionless strain under steady conditions of relative humidity and temperature.

**Elastic Shortening** - The concrete beam shortens at transfer when the prestressed strands are released and the force in them is transferred to the concrete. This elastic shortening is immediate and results in a reduction in the strain of the prestressing steel and therefore a prestress loss. The loss from elastic shortening should be included in both initial and total loss computations.

**Concrete Creep** - The time dependent increase of strain in hardened concrete subjected to sustained stress is defined as concrete creep.

**Steel Relaxation** - Steel relaxation is very similar to concrete creep. With steel relaxation the length of the strand is held constant under stress and there is a time dependent loss in stress.
The designer shall use a “t” of 18 hours for computing steel relaxation loss at transfer. This represents the shortest time that is likely to occur between jacking and detensioning. For initial stresses the main problem is over stressing the beam ends due to excessive prestressing force.

**Anchorage Set** - Some loss of prestress occurs to post-tensioned tendons as the anchorage hardware deforms and sets at the transfer of tension. The amount of set is a function of the type of anchorage system used. The amount of prestress loss is a function of this anchorage set and the length between anchorages. Power seating of the chucks tends to reduce this loss. For design purposes assume anchor set as \( \frac{3}{8} \) in.

**Friction** - Tendons also lose some prestress due to friction inside the ducts during stressing operations.

**Total Losses** - Some of the losses mentioned above are interdependent. Shrinkage and concrete creep reduce the strain in the prestressing steel, which reduces the force in the prestressing steel. The reduction in force in the prestressing steel affects elastic shortening, future concrete creep and steel relaxation.

### 9.11 Allowable Stresses

#### 9.11.1 Temporary Stresses

Temporary stresses correspond to the stresses that are present at transfer. Only initial losses should be considered when checking these stresses.

#### 9.11.2 Final Stresses

Final stresses represent the stresses at service load after all losses have occurred. When AASHTO HL-93 live loading is used, the maximum allowable tension in the precompressed tensile zone shall be \( 0.0948 \sqrt{f'_{c}} \text{ksi} \), as per the LRFD specifications. When the NYSDOT Design Permit Vehicle is used the maximum allowable tension shall be \( 0.14 \sqrt{f'_{c}} \text{ksi} \). The design allowable stresses shall be shown on the contract plans.

### 9.12 Reinforcement

Reinforcement in prestressed units shall not be epoxy coated with the exception of the composite stirrups extending into the deck, or the top longitudinal bar extending into the approach slab which may be epoxy coated or galvanized.

#### 9.12.1 Shear Stirrups

Detailing of shear reinforcement shall follow the guidance shown on the appropriate BD sheets.
9.12.2 Composite Design Reinforcement

Composite flexural members consist of prestressed members acting with a cast-in-place concrete deck. In order for the deck to act compositely, reinforcement must be provided extending out of the beams into the slab to resist the horizontal shear developed across this plane. Composite shear reinforcement shall be provided for the full length of prestressed concrete bridge beams, including the negative moment areas of continuous spans.

9.12.3 Anchorage Zone Reinforcement

When pretensioned strands are released and their stress is transferred to the hardened concrete bonded to the strands, the concrete at the beam ends experiences tensile stress perpendicular to the direction of prestressing. Anchorage zone reinforcement shall be provided to resist these stresses. For slab units and box beams, stirrups with multiple legs can be used to accommodate required reinforcing within the specified distance from the end of the beam.

9.13 Camber

Due to the eccentric nature of prestressing, prestressed concrete units are typically curved upward under low values of externally applied loads. The resulting upward deflection is called camber. Camber may increase or decrease with time, depending on the stress distribution across the member under sustained loads.

Units shall be designed so that the algebraic sum of the beam camber at prestress transfer due to prestress force, the beam dead load deflections due to non-composite dead load, and superimposed dead load deflections due to applied superimposed dead loads results in a positive (upward) camber. The dead load from a future wearing surface shall be included in the determination of camber.

Allowed camber deviations for beams (see Section 7 of the PCCM.) should be considered in determining minimum expected camber based on design calculations. The following minimum net positive cambers are recommended:

- Spans 80 ft and above: ½” minimum
- Spans 50 ft to 80 ft: ¼” minimum
- Spans less than 50 ft: ⅛” minimum

The contract plans shall show the camber at prestress transfer and the deflections due to non-composite dead load and superimposed dead load.

9.14 Stage Construction Camber Differences

For a given project, fabricators typically cast all of the beams of a given size at the same time to minimize the time required to set up the casting beds. If these beams are subsequently erected at the same time, differential camber between beams is rarely a significant problem.
On stage construction projects, the precast beams may be fabricated at relatively the same time and erected many months, even years apart. The haunch provided for spread prestressed box beams, AASHTO I-beams and Bulb Tees is sufficient to accommodate this differential camber growth and need not be considered. Since adjacent precast unit superstructures have no haunch, the differential camber due to time dependent effects shall be considered.

The anticipated camber growth during storage of Stage 2 units may be assumed to be 50% of the camber at transfer. For all staged construction bridge superstructure projects, the minimum Stage 1 deck slab thickness shall be 7” in order to provide a minimum 6” deck slab over the Stage 2 units. The additional Stage 1 slab thickness of 1” shall be considered as extra dead load in the unit design calculations.

If the anticipated camber growth with no control measured during storage is greater than 1”, specific measures to control camber growth of the Stage 2 beams or other methods to limit the different camber growth between Stage 1 and Stage 2 must be specified in the contract documents. Typical notes in Section 17 of this manual must be placed on the contract plans.

Example:
Camber at transfer (w/o creep) = 1 inch
Anticipated camber growth = 0.5 x 1 inch = ½ inch

9.15 Continuity Design at Interior Supports

Unless significant differential settlement between supports is expected, all multi-span prestressed concrete superstructures shall be designed and detailed with continuity connections at all interior supports except at locations on long structures were periodic joints are needed to accommodate thermal movements.

Other than post-tensioned structures, multi-span prestressed concrete superstructures shall be designed and detailed as continuous for live load only. All multi-span prestressed concrete superstructures with post-tensioned and spliced beams shall be designed and detailed as continuous for all live and dead loads. See Section 9.17 for guidance on post tensioned structures.

Current practice for continuous for live load design is to establish the continuity connection at the same time as the placement of deck concrete. Hence, dead load due to the deck concrete will be handled by prestressed beams acting as simply supported beams. Live loads and superimposed dead loads applied after the deck concrete hardens will be handled by the continuous composite (beams and deck acting together) structure. Loads applied after the hardening of the deck concrete will cause negative moment over the continuity connections.
The use of high-strength, high-performance concrete (10 ksi) for all prestressed concrete bridge beams and requiring the beams age for a minimum age of 60 days prior to deck placement helps reduce stresses on the continuity connection. However it is not possible to accurately predict all of the loads applied on the continuity connections. The unknown number of actual field loading conditions and the number of assumptions needed makes it difficult to design the continuity connection in full accordance with the requirements of the LRFD Design Specification. Therefore, designers of prestressed concrete beams designed and detailed with continuity connections shall adhere to the following guidance:

- The beams shall initially be designed as simple spans, neglecting the effect of continuity, including full live load and superimposed dead load. The design shall then be analyzed assuming full continuity for the live load plus impact loads, and for the superimposed dead load. The final beam design shall satisfy the requirements for both conditions.
- Designers shall assume full continuity in calculating the tensile stresses in the deck and at the top of the beam ends. The designer shall ensure that there is adequate top mat reinforcement in the slab to resist this condition.
- All Load Ratings shall be calculated on the simple span condition only.
- NYSDOT standard continuity connection details may be used without design of the continuity connection.

Continuous for live load details for box beams and slab units can be found on BD-PA10E. Continuous for live load details for bulb tee and AASHTO I-beams can be found on BD-PS8E.

9.16 Corrosion Inhibitors and Sealers

Prestressed concrete elements shall use corrosion inhibitor and penetrating silane sealer. See the PCCM for details.

9.17 Post-Tensioned Spliced Girder Designs

Prestressed concrete bridge beams may be spliced by joining two or more beam segments to form one beam. Typically, splicing is achieved by cast-in-place concrete along with longitudinal post-tensioning. Splicing of bridge beams is generally used for one or more of the following reasons:

- Increasing span lengths to reduce the number of sub-structure units and total project cost;
- Increasing the girder spacing to reduce the number of girder lines and total project cost;
- Increasing span lengths to improve safety by eliminating shoulder piers or interior supports;
- Minimizing structure depth through the use of long, continuous members to obtain required vertical clearance for traffic, waterways, and so forth;
- Avoiding the placement of piers in water to reduce environmental impact and total project cost;
- Placing piers to avoid obstacles on the ground, such as railroad tracks, roadways, and utilities;
Improving aesthetics through various design enhancements, such as more slender superstructures, longer spans, of haunched sections at piers;
- Eliminating joints for improved structural performance, reduced long-term maintenance/increased service life, and improved rideability.

Whenever possible, part of the longitudinal post-tensioning shall be applied after the hardening of the deck concrete so that net tension on top of the deck surface is less than or equal to the modulus of rupture.

The Contract plans shall show a recommended installation method and post-tensioning sequence. See current BD Sheets for additional guidance. The structural analysis should consider the effects of fabrication and erection tolerances on bridge performance.
The minimum distance from the center of a pile to the nearest footing edge should be 1’-6”. The minimum distance from the edge of a pile to the nearest footing edge shall be 9”. The minimum distance from the center of a pile to the nearest edge of the capbeam shall be 1’-6”. The minimum distance from the edge of a pile to the nearest edge of the capbeam shall be 1’-0”.

The tops of C.I.P. piles shall be embedded 6” into the footing. The tops of all other piles shall be embedded 1’-0” into the footing. Additional embedment requirements apply to integral abutments (see Section 11.6.1.6).

When a pier is composed of steel H-piles extending above the ground surface and embedded directly into a concrete capbeam, the piles shall be embedded a minimum of 2’-0” into the capbeam. The same embedment applies to C.I.P. piles unless the pile reinforcement projects into the cap. In this case, the embedment shall be 1’-0”.

The front row of piles (at the toe) of all abutment and wall footings shall be battered. The outside rows of piles of all pier footings shall be battered. Minimum batter is 6 on 1; however, analysis may indicate that a greater batter is required. The maximum batter shall be 3 on 1. If a critical clearance problem exists (e.g., underground utilities), it may be possible to place some piles vertical that would ordinarily be battered. In this case, the Office of Structures’ Foundations and Construction Unit should be consulted. Horizontal forces must be resisted through a combination of the horizontal component of a battered pile and the lateral resistance of the soil to the pile. Lateral resistance of piles is specified in the Foundation Section of the latest NYSDOT LRFD Bridge Design Specifications unless modified in the FDR.

### 11.1.4.3 Numbering and Tabulation of Piles

All piles used in a structure shall be numbered on the plans. The pile numbering shall begin with the number one (1) and proceed continuously through all piles in that substructure unit. The pile numbers shall restart with the number one (1) for each different substructure unit encountered.

In order to record the actual driven length, a table shall be placed on the plans for each different substructure unit. The table shall include a column titled “PILE NO.” and a column titled “LENGTH BELOW CUT-OFF.” The length below cut-off of each pile shall be filled in by the E.I.C.

### 11.1.4.4 Pile Splices

Piles requiring splices shall be spliced by using either complete penetration groove welds or mechanical splices.

Complete penetration groove welds are allowed on splices for all conditions.

Mechanical splices are not allowed in the following situations:

- Any type of pile subject to uplift loads.
- CIP piles in integral abutments due to bending.
- CIP piles in any substructure subject to scour, when the depth of scour from either the Q100 or Q500 flood is below the bottom of footing.

The FDR will provide notes to be included in the contract plans when Mechanical splices are not allowed. When mechanical splices are allowed on CIP piles, a seal weld will still be required.

When the estimated length of pile exceeds 30 ft., the designer's estimate shall allow for at least one-half the total number of piles to be spliced. This is a contingency to cover the situation where the actual length of driven pile exceeds the estimated length by more than 10 ft.

Details of pile splices and reinforced tips are shown on the current BD sheets. These details shall be included in the contract plans.

11.1.5 Drilled Shafts

Drilled shafts are typically used as an alternative to piles. They are capable of carrying very large loads. Drilled shafts are usually advanced with a steel casing, although a slurry solution is sometimes used to keep the excavation open. The FDR may require that the shaft be socketed a minimum distance into bedrock to develop the necessary skin friction to support the applied loads. When the shaft is excavated, reinforcement is placed and the shaft is filled with concrete.

11.1.6 Pilasters

Pilasters are typically square concrete columns that are used when rock is located too near the surface to drive piles. They are capable of handling very large loads. Pilasters are usually constructed in an open excavation down to bedrock and may be socketed into bedrock a minimum distance.

11.1.7 Design Footing Pressures and Pile Capacities

Notes that specify either the maximum foundation pressure for spread footings or the maximum allowable pile load and ultimate pile capacity for pile foundations shall be shown on the contract plans. The wording and format of these notes are given in the FDR. For spread footings on rock, the actual design bearing pressure shown on the plans should be rounded to the nearest one-half ton/ft².

11.1.8 Footing Depth

The depth to which footings are carried below the ground surface is usually determined by three factors: frost depth, scour action, and foundation type.

Frost heaves in soil can cause displacement of the footing and damage to the structure. Spread footings founded on soil shall have their bottom of footing a minimum of 4 ft. below finished ground to assure that the bottom of the footing is below the maximum frost penetration. Spread footings on rock are not susceptible to frost heaves and, therefore, do not require the minimum 4 ft. depth. Spread footings on soil are not ordinarily used near water because of their
vulnerability to scour action. If they are used near water, however, their bottom of footing needs to be well below any potential scour depth and special protective measures may be needed.

Although footings supported on piles or drilled shafts are not normally susceptible to frost action, they are often subject to erosion or scour action. Footings on piles, drilled shafts, or pilasters should be set at least 4 ft. below the (stream bed, river bed, lake bottom, etc.) or finished slope. The top of the footing should be at least 1 ft. below the finished ground surface, therefore, thicker footings may require more than a 4 ft. depth.

If a stone apron is to be used for bank protection, sufficient room must be left to place the stone over the top of the footing.

### 11.1.9 Stepped Footings

Stepped footings introduce construction difficulties and, in the case of spread footings on soil, an increased risk of differential settlement. They are, therefore, very seldom used. The use of stepped footings may be warranted in some cases, such as a variable rock elevation or a long wall where the required bottom of footing elevation changes considerably.

The most common reason for stepping footings is to accommodate spread footings on a sloping rock surface. Stepped footings on rock shall have steps at least 8 ft. in length and at least a 2 ft. change in height. Footing continuity is not required.

Stepping spread footings on soil or pile foundations should only be done under wingwalls and retaining walls longer than 25 ft. The minimum length of each step section should be 12 ft and the change in height of each step should be at least 2 ft. Footing continuity is preferred for all steps up to 3 ft., but is not mandatory. Steps more than 3 ft. will require a construction or contraction joint to facilitate construction. Any joint introduced shall be continued up through the stem or walls above.

Stepping of the leveling pad for a Mechanically Stabilized Earth System (M.S.E.S.) on embankments is permitted. The minimum length of a step section is the width of one panel. The minimum height of a step for this type of wall system is one half the panel height. The manufacturer of the mechanically stabilized earth system shall set the final configuration of the leveling pad as part of the panel layout.

Any proposed steps in footings should be included in the Preliminary Structure Plan and approved by the Office of Technical Services’ Geotechnical Engineering Bureau.

### 11.1.10 Tremie Seals

A tremie seal is concrete placed under water through the use of a tremie placement tube. As the concrete is placed, water is displaced and the tube is gradually raised keeping the outlet below the level of the placed concrete. Tremie seals are usually used where piers need to be constructed in fairly deep water and it is difficult to dewater the excavation. A sheet piling cofferdam is usually placed to surround the excavation. Piles, if required, are driven inside the cell with water still inside. The tremie seal is then placed to a level where its submerged weight will exceed the hydrostatic pressure of the dewatered excavation. The water can then be
pumped out of the excavation and the footing constructed on top of the tremie seal in the dry. The piles should be sufficiently long to project above the tremie seal and into the footing. Tremie concrete uses Class G concrete which has a higher cement content and slump range than Class A concrete.

In the design of a tremie seal, the designer must remember to use the buoyant weight of the concrete in balancing the hydrostatic pressure. In calculation, the dry weight of the tremie seal should be conservatively taken as 140 lb/ft$^3$. Tremie seals are normally designed to resist the hydrostatic pressure at ordinary high water. The excavation should be designed to flood when the water level exceeds ordinary high water to prevent unequal hydrostatic pressure from “floating out” the tremie seal during construction. A minimum safety factor of 1.25 is recommended in tremie design. See Section 4 of this manual for further information on cofferdams and tremie seals.

11.1.11 Footing Thickness

The minimum footing thickness for spread footings shall be 2 ft. The minimum footing thickness for pile supported footings shall be 2'-2" for C.I.P. piles and 2'-6" for steel H-piles.

11.2 Forming Considerations

In heavily reinforced concrete structures, the labor and material costs for formwork often average between 30% to 50% of the total in-place cost of the structure. Within that total formwork cost, the labor cost to build and place the forms is generally two to three times the formwork material cost. In other words, an efficient structure is one that not only conserves cubic yards of concrete, but also reduces the labor involved in formwork. The shape should be such that large flat forms and large placements may be employed.

Simplicity and repetition are the keys to achieving economy in forming. Configurations that lend themselves to commercially built forming systems will generally be less expensive than those requiring custom built forms. If special forms are required, the high initial cost of those forms may be offset if those forms can be used several times.

Generally, in normal column construction the circular shape is the most economical to form because commercially prefabricated forms are available in many standard diameters. These forms are easy to set up, strip and require no form ties.

Battered forms are more expensive than vertical forms and should be avoided whenever possible, especially on short wingwalls. If a thicker wall section is required at the base of a wall, the designer should consider using the thicker section for the full height of the wall or to a construction joint and stepping the thickness. If battered forms are used, the batter should remain constant. Battering only one side is the least expensive battering system. Battering on three or four sides always requires special forming and should only be considered when the hydraulic flow characteristics require special pier geometry.
when the top 8 ft. is excavated and backfilled with sand. If no pre-excavating for the piles is required, penetrations as low as 10 ft. can be used.

A pile bent configuration is to be used for the integral abutment detail. For steel and spread concrete girder bridges, a minimum of one pile per girder shall be used.

Wingwalls

Unlike other abutments, the wingwalls for integral abutments have special requirements. In-line wingwalls cantilevered from the abutment are the preferred arrangement. Flared walls cantilevered from the abutment may be considered by the designer on a case by case basis. The use of flared wingwalls should generally only be considered at stream crossings where the alignment and velocity of the stream would make in-line walls subject to scour. Piles shall never be placed under flared wingwalls that are integral with the abutment stem. Generally, the controlling design parameter is the horizontal bending in the wingwall at the fascia stringer caused by the large passive pressure behind the wingwalls. In-line or flared wingwalls connected to the abutment stem with lengths in excess of 13 ft. should be avoided.

Because of high bending moments due to passive soil pressure, it may be necessary to support long wingwalls (13 ft. or more measured along the wall) on their own foundation, which is independent of the integral abutment system. In this case, a flexible joint must be provided between the wingwalls and the backwall. The joint between the abutment and the wingwalls shall be parallel to the centerline of the roadway to accommodate the longitudinal movement of the bridge. A joint that is not parallel to the direction of movement will likely lead to binding between the abutment stem and wingwall. Separate wingwalls may be designed as conventional walls with a footing or a stem with a single row of alternately battered piles. The choice will be governed by the site and loading conditions, but walls using a single row of piles should generally be limited to a height of 13 ft. Separate wingwalls for integral abutments on bridges over water shall be pile supported.

U-wingwalls cantilevered from the abutment stem shall be allowed only if in-line or flared walls cannot be used because of right-of-way or wetlands encroachment. The U-wingwalls shall not measure more than 6’-6” from the rear face of the abutment stem. No piles shall be placed under U-wingwalls physically connected to an integral abutment. This would inhibit the abutment's ability to translate and would cause internal stresses. The distance between the approach slab and the rear face of the U-wingwall should be a minimum of 4 ft. If the approach slab must extend to the U-wingwall, it shall be separated from the U-wingwall by a 2” joint filled with at least two sheets of Premoulded Resilient Joint Filler, Material Subsection 705-07.

Utilities

Rigid utility conduits, such as gas, water and sewer, are discouraged for use with integral abutments. If they are used, expansion joints in the conduits must be provided at each abutment. Sleeves through the abutment should provide at least 2” clearance all around the conduit. Flexible conduits for electrical or telephone utilities that are properly equipped with an expansion sleeve through the integral abutment are acceptable.
Stage Construction

When stage construction is used with integral abutments, the use of a closure placement between stages in the abutments shall be considered. The use of a closure placement can reduce the mismatch of the top of slab between stages caused by deflection from the superstructure. A closure placement in the abutment stem shall be required when the dead load deflection from the deck slab placement is calculated to be 3 inches or greater.

11.6.1.7 Semi-Integral Abutments

Description and Design Methodology

Semi-integral abutments use conventionally designed abutments where superstructure girders are supported by bearings and pedestals on a bridge seat. The girders extend over the bridge seat and are embedded in a backwall that hangs behind, but is not connected to, the abutment stem.

Full integral abutments have been used successfully by NYSDOT since the late 1970s. Their performance in terms of durability and first cost has been clearly superior to conventional abutments. This has mainly been due to the elimination of the deck expansion joint and the simple concrete forming required. Unfortunately, site condition criteria sometimes prevent their use. This is usually caused by rock being too close to the ground surface preventing the driving of piles or the necessity of using high abutments because of geometric constraints.

When site conditions have prevented the use of integral abutments, jointless decks at abutments have often been used. Jointless decks at abutments are conventionally designed but the deck slab extends and slides over the backwall. While jointless decks at abutments have performed better than conventional abutments with deck joints, there have been some problems with transverse deck cracking near the abutment backwall. Jointless decks at abutments are also limited to a maximum expansion length of 200 ft. Semi-integral abutments should be considered for use where site conditions prevent the construction of full integral abutments.

Semi-integral abutments are designed as conventional abutments with the following exceptions:

- Backwalls must be designed for full passive soil pressure.
- Wingwalls must be independent from the backwall to allow movement. Clearance details are shown on the applicable BD sheets.
- Adequate clearance to handle expected movements must be provided between the suspended backwall and the abutment stem.
- Provision for expansion at the ends of approach slabs should be provided in accordance with the details on the applicable BD sheet.
- The top reinforcement in the decks slab at the end of the span should be designed for the negative moment produced from the reaction of half the approach slab dead load and a live load reaction placed on the backwall. The dead load of the backwall should not be included because the backwall is constructed in a separate placement before the deck and will not contribute to tensile stress in the deck slab.
- Semi-integral abutments are not allowed for use on one end of a bridge opposite a conventional abutment. Each end of a bridge must have a semi-integral abutment to match passive pressure resistance.
Substructures

Stage Construction

When stage construction is used with semi-integral abutments, the use of a closure placement between stages in the backwall shall be considered. The use of a closure placement can reduce the mismatch of the top of slab between stages caused by deflection from the superstructure. A closure placement in the backwall shall be required when dead load deflection from the deck slab placement is calculated to be 3 inches or greater.

Selection Criteria and Details

- Maximum skew = 30°.
- Maximum expansion length = 230 ft. (distance to nearest fixed bearing).
- No restriction on abutment height.
- No restriction on maximum grade.
- No restriction on footing type (spread or pile foundation).
- Utility restrictions are the same as integral abutments. See § 11.6.1.6 of the Bridge Manual.
- Single span bridges with two semi-integral abutments can have expansion bearings at each end as long as the grade between bearings is less than 2.5% and there is no stop sign or signal at either end of the bridge.
- Single-span bridges should have one of the abutment bearings fixed if the grade between bearings is ≥2.5%. Multiple-span, continuous bridges can have both abutments with expansion bearings as long as there is a fixed bearing at a pier.
- Fixed bearing designs should size the anchor pin in the bearing to handle passive pressure from the opposing backwall as well as the friction resistance of the approach slab.
- Curved girder structures are allowed if the curved girders are designed as straight as provided in NYSDOT LRFD Bridge Design Specifications, § 4.6.1.2.
- Backfill procedures are the same as for Integral Abutments.
- The hanging backwall may have its bottom surface cast on the ground or formed at the option of the Contractor.
- Polyethylene curing covers need not be placed under the hanging backwall.
- Bearings should be reset to their neutral position after the girder rotates due to deck dead loads.

11.6.2 Abutment and Wall Details

11.6.2.1 Stem Thickness

The stem thickness of cantilevered high abutments is almost always governed by the size of the bridge seat required for clearance between the superstructure and the backwall, the bearings and the backwall, and seismic criteria. For bridges with a pier, seismic criteria may dictate the support length at the ends of beams. The minimum support length (N) in the longitudinal direction should be measured perpendicular to the centerline of bearing. The minimum support length (N) in the transverse direction should be measured perpendicular to the centerline of the beam. The minimum support length shall meet the requirements of NYSDOT LRFD Bridge Design Specifications § 4.7.4.4. The minimum bridge seat width is 3 ft. for steel, bulb tee and AASHTO I-beam superstructures and 2 ft. for adjacent concrete beam superstructures.
The stem thickness of integral and semi-integral abutments shall be as shown in the current BD-ID series. The centerline of the piles and the centerline of bearings of the beams shall always line up.

11.6.2.2 Pedestal Dimensions

The minimum height of the shortest pedestal is 6” when used with elastomeric bearings. If multi-rotational bearings are used, then the minimum height shall be 8”. The extra 2” is added for tolerance to allow the use of a taller multi-rotational bearing than the one used in the design and still provide a minimum pedestal height of 6”. If the difference in height between fascia pedestals is more than 6” then a sloping bridge seat should be used with both fascia pedestals being set at the minimum height. Pedestals more than 1’-6” high should usually be avoided for aesthetic reasons. Pedestals greater than this height should be investigated for their strength acting as a column.

The minimum distance from the center of the bearing anchor bolt to any exposed vertical face of the pedestal shall be 8”. In addition, the minimum distance from the edge of the masonry plate to any vertical face of the pedestal shall be 3” unless otherwise accounted for in the design. Masonry plate corners may be cropped to satisfy this requirement. The front face of all pedestals shall be flush with the front face of the abutment.

11.6.2.3 Drainage

The fill material behind all walls shall be effectively drained and weepholes shall be placed at a maximum spacing of 25 ft. In counterfort walls, there shall be at least one weephole for each pocket formed by the counterforts. Weepholes shall be located so that their invert is 6” above finished grade or low water in the case of stream bridges. Integral abutments generally do not require weepholes because of their minimal exposed height above finished ground. Weepholes
should only be placed in wingwalls over 40’ in length. Weepholes should not be placed so they drain onto sidewalks or shared-use paths, if possible.

11.7 Bridge Piers

For the purposes of this section, the term “pier” is defined as an intermediate support for a bridge superstructure, between the abutments, extending from below the ground surface to the bottom of the superstructure.

Piers may be required because of long spans, beam depth restrictions, or both. The pier may be a support point along a continuous superstructure, or it may be at the end of one simple span and the beginning of another. In either case, the pier must be designed to safely handle the dead, live, seismic and other loads introduced from the superstructure while at the same time handling any loads acting on the pier from flood water, ice flow, wind, and vehicular or ship impact. Suggested proportions of bridge piers can be found in Section 23.

11.7.1 Pier Types

Fig. 11.5
Typical Pier Types
(a) Solid (b) Hammerhead (c) Multi-column (d) Pile Bent
11.7.1.1 Solid Pier

Solid piers consist of a solid mass of reinforced concrete, without overhangs, that is usually rectangular in plan. Solid piers are used primarily for river or stream crossings, low-clearance bridges, bridges over divided highways with narrow medians, and where short columns on wide bridges would have high stress due to shrinkage. Solid piers can also be used to meet crash protection requirements adjacent to railroads. This type of pier is currently used by NYSDOT for new bridges.

11.7.1.2 Hammerhead Pier

With increasing pier height and narrow superstructures, the hammerhead pier becomes more economical by reducing the required amounts of material and forming. Hammerhead piers consist of a single large column with a capbeam overhanging on either side. Both the column and cantilevered ends of the capbeam support the superstructure beams. When located in a waterway, pier protection may be required. The overhangs of hammerhead piers may need to be investigated for the bracket and corbel effect as described in Section 15.10. This type of pier is currently used by NYSDOT for new bridges.

11.7.1.3 Multi-Column Pier

When piers need to be tall and wide, a multiple-column pier is usually the best choice. This pier type consists of two or more columns that can be either rectangular or circular. The columns are usually connected by a capbeam that supports the superstructure at points between the columns. For some highly skewed bridges with large beam spacing, it may be necessary to place individual columns under each bearing and to connect the top of the columns with a simple tie strut. When there are only two columns with overhangs, this pier is called a \( \pi \) (pi) pier. The overhangs may need to be investigated for bracket and corbel effects as described in Section 15.10. These types of piers are currently used in NYSDOT for new bridges.

A feature of most multi-column piers is the presence of the capbeam. This capbeam is subject to many design considerations that are not applicable to any other type of pier. The width of the capbeam is governed by the necessary width to support the bridge bearings with sufficient cover for the anchor bolts and the required support length for the beams. When the simply supported end of a beam rests on a pier, seismic criteria dictates the support length required. Support length (N) in the longitudinal direction should be measured perpendicular to the centerline of bearings. Support length (N) in the transverse direction should be measured perpendicular to the centerline of the beam. See Section 11.6.2.1. Round columns require that the capbeam be at least 2" wider than the columns on all sides.

For seismic response reasons, high concrete columns (slenderness >60) in multi-column piers shall have reinforced concrete struts between the columns in the middle half of the column height.
11.7.1.4 Pile Bents

Pile bents are the simplest and least expensive piers to construct. This pier consists of driven piles with a concrete cap beam cast over the top of the piles to support the superstructure. This type of pier is inexpensive because there are no footings or columns to form or cast. Pile bents are not frequently used by NYSDOT due to concerns about aesthetics, corrosion of the exposed steel piles or steel pile casings, and the closely spaced piles trapping debris during a flood and reducing the available hydraulic opening.

11.7.2 Pier Protection

Bridges in navigable waterways that are subjected to heavy commercial traffic may require additional protection according to AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges. Additional information can be found in Section 2 of this manual.

For stream bridges, a recommendation shall be obtained from the Office of Structures’ Hydraulic Design Unit regarding the need for and type of ice breaker for pier noses. If required, the ice breaker shall consist of a steel angle or other device secured to the concrete by a suitable anchor system. For solid piers, this breaker may be attached to the pier stem. For hammerhead piers and multi-column piers, a plinth may be required to provide sufficient strength against the anticipated ice flows. A plinth is a solid mass of concrete that surrounds the pier to an elevation 2 ft. above the 100-year flood or flood of record, whichever is higher. In a navigable stream, the plinth should be carried to 3 ft. above design high water or maximum navigable pool elevation, whichever is higher.

For piers between opposing directions of traffic, appropriate care must be taken to ensure that minimum horizontal clearances and highway traffic barrier requirements are satisfied. For more information, refer to the Highway Design Manual and Standard Sheets.

For multi-column or hammerhead piers adjacent to railroad tracks, the need for crash walls must be investigated based on the proximity of the pier to the tracks in accordance with current AREMA Specifications. Additional information can be found in Section 2 of this manual.
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12.6  Rehabilitation Projects

When a designer is evaluating a structure for rehabilitation, replacement of the existing rocker or slider bearings with maintenance free elastomeric bearings should be explored.

12.6.1  New Bearings on Existing Pedestals

For bridge rehabilitations where new bearings are to be put on existing pedestals, the designer shall detail the bearings so that the total bearing system heights will accommodate the bottom of steel elevations and top of pedestal elevations from the existing as-built plans.

When there are more than 20 bearing replacements in a project, the designer shall include Item 564.51.nnnn, Structural Steel, to pay for any shim plates that may be required due to differences in pedestal heights between actual field conditions and what is shown in the contract documents.

To estimate the quantity for this steel, designers should assume that half of the bearings will require a 1/2" thick shim plate the same length and 1" wider than the sole plate.

The shim plates shall be detailed on the bearing drawings in the contract plans. Include note 108 on the contract plans.

12.6.2  New Bearings on New Pedestals

Shim plates shall not be used on bridge rehabilitation projects with new bearings on new pedestals. Pedestal elevations shall be detailed based on the proposed bearing height and the bottom of steel elevations from the existing as-built plans. Include note 107 on the contract plans.
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14.3 Detailing Standards

14.3.1 Bridge Detail (BD) Sheets

Bridge Detail (BD) Sheets are provided to assist in bridge plan standardization. These sheets serve as a guide in the preparation of the contract plans and may be accessed in CADD format through Projectwise or in PDF format through the DOT website.

14.3.2 Title Blocks

Care should be taken to ensure consistency in the TITLE BLOCKS of all sheets within a set of plans, including multiple bridge projects. Most Title Block information is filled out using the Plans – Plan Sheet Border and/or Plans – Detail Sheet Border interfaces in Projectwise. For an overview of document attributes, see Section 2.6 in Appendix 14 of the Project Development Manual. For a complete list of standard Projectwise interfaces, see Section 2.7 in Appendix 14 of the Project Development Manual.

The bridge label featured in the LOWER TITLE BLOCK should be shown like this (format may be varied because of space constraints):

```
FEATURE CARRIED
OVER
FEATURE CROSSED
```

14.3.3 Scales and Scale Bars

Refer to Section 20.9 in Chapter 20 of the Highway Design Manual for a discussion of scales. Scale bars shall be provided for larger scale drawings that are site oriented such as the General Plan and Elevation, General Subsurface Profile and earthwork and embankment plans. Scale bars shall not be shown on roadway profiles.

All details that are drawn proportionally shall be fully dimensioned and shall not display a numeric scale or scale bar. Any drawings intentionally drawn not to scale shall be labeled “NOT TO SCALE” and shall be fully dimensioned. Note # 11 of Section 17.3 shall be included on the General Notes sheet.
The following are suggested scales (based on B-sized sheets, 11”x17”) to be used by detailers in the preparation of contract plans:

### Preliminary Plan

1” = 40’

### Abutments

- **Plan and Elevation**: 1/4” = 1’-0” no smaller than 1/8” = 1’-0”
- **Reinforcement**: 1/4” = 1’-0” no smaller than 1/8” = 1’-0”

### Piers

- **Plan and Elevation**: 1/4” = 1’-0” no smaller than 1/8” = 1’-0”
- **Reinforcement**: 1/4” = 1’-0” no smaller than 1/8” = 1’-0”

### Transverse Section

1/4” = 1’-0”

### Railings

1/2” = 1’-0”

### Bearings

1/2” = 1’-0”

### Superstructure Slab

1” = 10’, 1” = 20’

### Prestressed Concrete

1/8” = 1’-0”, 1/4” = 1’-0”

### Excavation

- **Plans**: 1” = 10’, 1” = 20’
- **Sections**: 1/8” = 1’-0”, 1” = 10’
- **Approach Slabs**: 1/8” = 1’-0”
- **Steel Framing Plan**: 1” = 10’, 1” = 20’
- **Girder Elevations**: Not to scale
- **Joints**: Not to scale

### Table 14-1

**Suggested Sheet Scales**

### 14.3.4 Dimension and Table Value Rounding

The following is presented as a guideline to rounding dimensions and table values on the contract plans:

- **Concrete**: Nearest ¼ in
- **Steel**: Nearest ¼ in
- **Reinforcement Length – Bent Bars**: Nearest ¼ in (rounded down)
- **Reinforcement Length – Straight Bars**: Nearest 1 in (rounded down)
- **Stations**: Nearest 0.01 ft
- **Elevations**: Nearest 0.01 ft
- **Camber Table**: Nearest 0.005 ft
- **Haunch Table**: Nearest 0.01 ft
- **Design Load Table**: Nearest 0.01 kip/ft
- **Moment Table**: Nearest 0.01 kip-ft
- **Shear Table**: Nearest 0.01 kips
- **Skew Angle**: Nearest 1 second
- **Bearing Azimuth**: Nearest 1 second

### Table 14-2

**Dimension Rounding Guidelines**
## STAGE CONSTRUCTION DETAILS

- Plan
- Tendon details
- Concrete Closure placement detail

## DEBONDING OR DRAPE STRANDS DETAIL

- Number of debonded strands
- Length of debonded strands
- Draped tendon profile
- Sections showing draped strands at midpoint and end

## POST-TENSIONED DETAILS

- Duct location
- Post-tensioning notes
- End block recess detail
- End zone reinforcement (Elevation and Sections)
- Clearance requirements for ducts at anchorage and midspan
- Post-tensioning duct profile
- Assumed construction sequence
- Grout tube schematic and vent details
- Splice detail end view
- Splice detail section
- Shear key details

## MISCELLANEOUS DETAILS

- Threaded insert/mechanical connectors details
- Embedded bearing plate details (not for integral abutments)
- Fascia details
- Anchor stud clearance detail
- Haunch details
- Continuity reinforcement and diaphragm details at piers
TRANSVERSE SECTION
(Prestressed Box Beams and Slab Units)

TRANSVERSE SECTION
- Overall width of structure
- TGL, Station line/HCL and POR
- Limits of structural slab item
- Limits of sawcut grooving
- Limits of protective sealing
- Travel lane widths
- Shoulder widths
- Usable shoulder to fascia dimension
- Cross slopes
- Crown of roadway
- Concrete slab thickness
- Applicable bar marks of all bars totally contained or originating in the slab
- All applicable sidewalk bar marks
- Cover to exposed faces
- Lap lengths
- Indicate if a bar is lapped to another bar with a different bar mark
- Spacing of reinforcement tied down to an exposed face
- Beams numbered
- Joint widths dimensioned
- Overhang dimensioned
- Transverse tendon
- Railing/barrier/screening
- Utilities
- Slab closure placement detail

FASCIA DETAIL
- Partial Railing/barrier/screening shown but not dimensioned (to be shown on the Railing/barrier/screening sheets)
- Concrete slab thickness
- Applicable bar marks of all bars totally contained or originating in the slab sidewalk or brush curb
- Cover to exposed faces
- Lap lengths
- Indicate if a bar is lapped to another bar with a different bar mark
- Slab depth dimensioned
- Fascia depth dimensioned
- Overhang dimensioned
- Indicate relationship between reinforcement placement and railing anchorage
END BLOCK REINFORCEMENT DETAIL

☐ Outline of beam end
☐ Outline of voids
☐ All applicable bar marks
☐ Cover to exposed faces
☐ Lap lengths
☐ Indicate if a bar is lapped to another bar with a different bar mark
☐ Spacing of reinforcement tied down to an exposed face

DESIGN LOAD TABLE

Dead loads (kips/ft), maximum shear (kips) at support and moment (kips-ft) at midspan:
☐ Beam
☐ Slab
☐ Haunch (spread beams only)
☐ Utilities
☐ SIP/FSIP forms (spread beams only)
☐ (Include internal diaphragms in beam loading)

Superimposed dead loads (kips/ft), maximum shear (kips) at support and moment (kips-ft) at midspan:
☐ Sidewalk
☐ Railing or Barrier
☐ Future wearing surface

Live load in HL-93 truck notation and NYSDOT Permit Vehicle
☐ Live load information denoted below table

BEAM REINFORCEMENT TABLE AND BAR BENDING DIAGRAMS

☐ All bar marks and bar bending diagrams required to construct the beam

CAMBER TABLE

☐ Camber due to prestressed force and beam dead load at transfer
☐ Camber due to Deflection due to slab dead load
☐ Camber due to Deflection due to superimposed dead load
☐ Total camber
STAGE CONSTRUCTION DETAILS
☐ Plan
☐ Tendon details
☐ Concrete Closure placement detail

ANCHOR DOWEL DETAIL
☐ Hole opening diameter
☐ Anchor dowel diameter
☐ Hole filler placed in top of hole opening

INTERNAL DIAPHRAGM DETAILS
☐ Outline of beam end
☐ Outline of voids
☐ All applicable bar marks
☐ Cover to exposed faces
☐ Spacing of reinforcement tied down to an exposed face

DEBONDING DETAILS
☐ Number of debonded strands
☐ Length of debonded strands

POST-TENSIONED DETAILS
☐ See Bulb Tee and I Beam Details

MISCELLANEOUS DETAILS
☐ Transverse tendon plan, section and elevation
☐ Continuity reinforcement and diaphragm details at piers
☐ Shear key detail
☐ Bearing placement
☐ Bearing pad placement (integrals)
Section 15
Concrete Reinforcement

15.1 Introduction

This section is intended to aid the bridge designer and detailer in the area of concrete reinforced design and detailing. The tables in this section simplify the design and detailing of concrete reinforcement splices and required covers. Also included are suggested details intended to ease the construction process and provide seismic resistance.

15.2 Spacing

The minimum spacing shall meet NYSDOT LRFD Bridge Design Specification Section 5.10.3.1 requirements. The maximum clear spacing between parallel bars shall not be more than 1’-6”. The clear space between bars shall also apply to the clear distances between the contact splices and adjacent splices of bars. Bar spacings as indicated are always between the center of the bars unless otherwise noted as a clear distance. When reinforcement in beams or girders is placed in two or more layers, the bars in the upper layers shall be placed directly above those in the bottom layer.

15.3 Cover

The following list pertains to the minimum cover for plain, epoxy and galvanized reinforcing bars. Refer to Section 5 for cover of monolithic decks.

Top of sidewalk slabs................................................................................................ 1 ½"
Beams and Columns.................................................................................................... 2"
Pedestal (Top)............................................................................................................ 2"
Pedestal (Sides)......................................................................................................... 3"
Walls and Piers above footing (Including those adjacent to water)................................. 2”**
Footings (Including unformed bottom) ........................................................................ 3”**
Approach slab (Top).................................................................................................. 3”
Approach slab (Bottom and Sides) ................................................................................ 3”
Bottom of bottom slab of cast-in-place culvert .............................................................. 3”
Bottom of top slab of cast-in-place culverts and rigid frames......................................... 2”
All other cast-in-place culvert faces ............................................................................. 2”
Top of top slab of precast culverts (Fill <2 feet).......................................................... 2”
Top of top slab of precast culverts (Fill ≥2 feet)........................................................... 1”
All other precast box culvert faces .............................................................................. 1”
Exposed faces of precast three-sided culverts ............................................................. 1 ½"
All other faces of precast three-sided culverts ............................................................. 2”
Arches (Intrados and extrados)....................................................................................... 2”
Precast and cast-in-place piles ........................................................................................................ 2”
Precast piles exposed to sea water ..................................................................................................... 3”
Post-tensioned cylindrical piles (Centrifugally cast, no slump concrete exposed to sea water) ................................................................................................................................. 1 ½”
All other surfaces exposed to sea water ................................................................................................. 4”
* When aesthetic treatment (formliner) is used, the maximum relief of the treatment shall be added to the minimum cover.
** May be increased to accommodate piles when necessary.

Table 15-1
Minimum Reinforcement Cover

15.4 Reinforcing Bar Guidelines

Grade 60 is the standard strength reinforcing bar to be used on Department projects. Grade 75 reinforcing bar is available, though in limited quantities and at greater cost. Use of Grade 75 reinforcing bars should be limited to areas of high tensile stresses where the number of Grade 60 reinforcing bars results in insufficient spacing between the bars for concrete placement.

<table>
<thead>
<tr>
<th>TABLE A</th>
</tr>
</thead>
<tbody>
<tr>
<td>STANDARD REINFORCING BAR PROPERTIES</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
<th>#9</th>
<th>#10</th>
<th>#11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (in²)</td>
<td>0.20</td>
<td>0.31</td>
<td>0.44</td>
<td>0.60</td>
<td>0.79</td>
<td>1.00</td>
<td>1.27</td>
<td>1.56</td>
</tr>
<tr>
<td>Dia. (in)</td>
<td>0.500</td>
<td>0.625</td>
<td>0.750</td>
<td>0.875</td>
<td>1.000</td>
<td>1.128</td>
<td>1.270</td>
<td>1.410</td>
</tr>
</tbody>
</table>

15.4.1 Maximum Bar Lengths

Most reinforcing bar plants in the United States produce bars in a standard length of 60 feet except solid stainless steel larger than #6 is only available in maximum lengths of 40 feet. Therefore, plans should not include any straight bars or bent bars with a length in excess of 60 feet (40 feet for solid stainless larger than #6). Due to handling concerns, the maximum length of a bar that requires a hook on both ends should be limited to 30 feet.

15.4.1.1 Deck Slab Bars

Refer to Section 5.1.5.4 Deck Overhangs for guidance on deck slab bars.
TABLE P

CLASS C SPLICE-EPOXY COATED (NOT TOP BARS)

<table>
<thead>
<tr>
<th>Size</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
<th>#9</th>
<th>#10</th>
<th>#11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing ≥ 6&quot;</td>
<td>2'-2&quot; [1'-9&quot;]</td>
<td>2'-9&quot; [2'-2&quot;]</td>
<td>3'-6&quot; [2'-10&quot;]</td>
<td>4'-9&quot; [3'-10&quot;]</td>
<td>6'-4&quot; [5'-1&quot;]</td>
<td>8'-0&quot; [6'-4&quot;]</td>
<td>10-1&quot; [8'-1&quot;]</td>
<td>12'-4&quot; [9'-11&quot;]</td>
</tr>
<tr>
<td>Spacing &lt; 6&quot;</td>
<td>2'-9&quot; [2'-2&quot;]</td>
<td>3'-5&quot; [2'-9&quot;]</td>
<td>4'-5&quot; [3'-6&quot;]</td>
<td>6'-0&quot; (N/A)</td>
<td>7'-10&quot; (N/A)</td>
<td>9'-11&quot; (N/A)</td>
<td>12'-7&quot; (N/A)</td>
<td>15'-5&quot; (N/A)</td>
</tr>
</tbody>
</table>

The lengths in parentheses can only be used as described in TABLE H.

TABLE Q

CLASS C SPLICE-EPOXY COATED (TOP BARS)

<table>
<thead>
<tr>
<th>Size</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
<th>#9</th>
<th>#10</th>
<th>#11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing ≥ 6&quot;</td>
<td>2'-6&quot;</td>
<td>3'-1&quot;</td>
<td>4'-0&quot;</td>
<td>5'-5&quot;</td>
<td>7'-1&quot;</td>
<td>9'-0&quot;</td>
<td>11'-5&quot;</td>
<td>14'-0&quot;</td>
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<tr>
<td>Spacing &lt; 6&quot;</td>
<td>3'-1&quot;</td>
<td>3'-10&quot;</td>
<td>5'-0&quot;</td>
<td>6'-9&quot;</td>
<td>8'-11&quot;</td>
<td>11'-3&quot;</td>
<td>14'-3&quot;</td>
<td>17'-5&quot;</td>
</tr>
</tbody>
</table>

15.5.3 Length of Splices for Compression Bars

TABLE R

<table>
<thead>
<tr>
<th>Size</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
<th>#9</th>
<th>#10</th>
<th>#11</th>
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</thead>
<tbody>
<tr>
<td>Beams</td>
<td>1'-4&quot;</td>
<td>1'-8&quot;</td>
<td>2'-0&quot;</td>
<td>2'-3&quot;</td>
<td>2'-7&quot;</td>
<td>2'-11&quot;</td>
<td>3'-3&quot;</td>
<td>3'-8&quot;</td>
</tr>
<tr>
<td>Tied Columns</td>
<td>1'-1&quot;</td>
<td>1'-5&quot;</td>
<td>1'-8&quot;</td>
<td>1'-11&quot;</td>
<td>2'-2&quot;</td>
<td>2'-5&quot;</td>
<td>2'-9&quot;</td>
<td>3'-0&quot;</td>
</tr>
<tr>
<td>Spiral Columns</td>
<td>1'-0&quot;</td>
<td>1'-3&quot;</td>
<td>1'-6&quot;</td>
<td>1'-8&quot;</td>
<td>2'-0&quot;</td>
<td>2'-2&quot;</td>
<td>2'-6&quot;</td>
<td>2'-9&quot;</td>
</tr>
</tbody>
</table>
15.6 Marking of Bars for Bar Lists

Bars shall be marked consecutively, beginning with the number one (1), through each structural unit. A structural unit, such as an abutment, includes all concrete subdivisions (abutment footing, abutment stem, wingwall footing, wingwall stem, etc.) which together comprise the entire unit. In the bar list, structural units are to be identified by a general heading (e.g., Beginning Abutment). Appropriate subheadings shall also precede the listing of bars in each subdivision (e.g., Wingwall 1, Beginning Abutment Stem). When a subdivision is still further divided into more than one concrete placement, the listing of bars in each placement shall also be preceded by appropriate identification (e.g., Beginning Abutment Stem, Placement 1).

Typical bar marks shall specify the bar size, structural unit the bar originates in, whether the bar is plain, epoxy coated (E), galvanized (G), stainless steel clad (C) or solid stainless steel (S), and the bar number.

Exception: The dowels between all types of Permanent Concrete Traffic Barrier and Parapet for Structures and the structural slab or U-wingwall shall not be listed in the structural slab or wall bar list even though the bars originate in the slab or wall. These bars are to be paid for in the traffic barrier item and placed in the standard bar list table. These bars shall not appear in the superstructure slab bar list. The reason for this policy is that the bars associated with all types of Permanent Concrete Traffic Barrier and Parapet will change if the contractor chooses the precast option for the barrier. See Notes 70 and 71 in Section 17.3.

In applying the bar marks where two or more structure units are involved, such as two or more similar abutments, piers, spans, etc., it is desirable that the same bar marks be applied to bars in similar locations in the structure unit. The fact that two bars lying in different structure units may have the same bar mark but have different lengths, or they may have the same length but have different sizes, or any combination of these factors will not be confusing to the fabricator due to the practice of providing a separate bar list, properly titled, for each structure unit.

For varying length bars, give minimum, maximum and average lengths of bars. Give number of sets of bars, even if the number of sets is one.

Any deviation from the above system of marking bars must have the approval of the D.C.E.S. See Section 15.13 for guidance on projects without bar lists.
15.7 Footing Reinforcement

Footing reinforcement shall be designed for the applied loads, but the following minimum requirements shall be provided to maintain the integrity of the footing in the event of seismic loading:

1. The bottom reinforcement mat in footings with piles shall be placed 2” clear above the tops of the piles. In special cases, where design requirements dictate and the pile pattern permits, the bars may be located between piles. In this case, a minimum clear distance of 3” shall be maintained between the reinforcing bars and the piles.

2. The vertical compression reinforcement of all abutment stems and walls shall be doweled into the footing. These dowels should have 90° hooks on the bottom end. See Table C of Section 15.5.1 for required embedment length. Minimum reinforcement shall be as per NYSDOT LRFD Bridge Design Specifications thermal and shrinkage requirements §5.10.8.

15.8 Abutment Reinforcement

The top layer of bridge seat reinforcement for steel girder, prestressed concrete I-beams, and spread prestressed concrete box beams shall be #8 bars at 6”. For adjacent prestressed concrete box and slab unit structures, the top layer of bridge seat reinforcement shall be #8 bars at 8”.

Dowels on the compression side of the abutment stem shall meet the requirements of Note 2 of Section 15.7.

The minimum reinforcement on non-exposed faces shall be #5 bars at 1’-6”. The entire capacity of these bars shall be developed by embedment or lapping the bar.

15.9 Column Reinforcement

Lap splices shall not be located within the plastic hinge zones (LRFD §5.10.11.4.1.c). Dowels shall extend at least ¼ of the column height or 10 feet, whichever is greater. Splices in the vertical design reinforcement shall be staggered. Vertical reinforcement shall be extended into the pier cap for the full embedment length.

Continuous ties shall surround the vertical reinforcement. Ties shall be not less than #4 bars. Spacing of lateral ties in the interior length of pier columns shall not exceed the least plan dimension of the compression member or 1’-0”, whichever is less. In plastic hinge zones vertical spacing of ties shall be as specified in NYSDOT LRFD Bridge Design Specifications §5.10.11.4.1.e. All stirrups shall be provided with 135° hooks. When spirals are provided in lieu of lateral ties, the pitch is as AASHTO specifies. Spirals should stop at the level of the footing or the capbeam and circular ties shall be used for a distance equal to ½ the greater column plan dimension, but not less than 1’-3” into the footing or cap beam. In lightly reinforced footings, where there would be minimal interference between the spiral and the footing reinforcement, spirals may continue in lieu of the circular ties into the footing and the cap beam. Lateral ties shall be as specified in NYSDOT LRFD Bridge Design Specifications §5.10.11.4.1d.
For seismic reasons, when a plinth is provided at the base of a column, the design vertical reinforcement of the columns shall extend into the footing. Additional reinforcement in the plinth may be required due to other design forces.

15.10 Pier Cap Reinforcement

The splices of top bars in the cap beam shall be staggered so no more than 50% of the bars are spliced at one location. The splices shall be located in areas of low negative moment. The splices of bottom bars in the cap beam shall be staggered so no more than 50% of the bars are spliced at any one location. The splices shall be located in areas of low positive moment.

When pier cap bars are spliced, the lap splices of the bars shall be in a vertical plane so the bars will be in the proper position for attachment to stirrups. To accommodate this type of splice, where more than one layer of reinforcement is required, it may be necessary to increase the distance between the layers of reinforcement.

Capbeams with overhangs require special attention. Two cases need to be investigated based on the geometry of the applied loads on the overhang region of the capbeam. First, AASHTO requires that shear due to concentrated loads within a distance "d" (d = capbeam depth) from the column face be included in the flexural design shear.

For capbeam cantilever ends where the fascia beam load falls within a distance "d" from the column face, the actual behavior of the cantilever end may not be compatible with beam theory and must be checked against the requirements of NYSDOT LRFD Bridge Design Specifications §5.13.2.4, Special Provisions for Brackets and Corbels. An alternative method to analyze such cantilever ends is the strut and tie method described in the NYSDOT LRFD Bridge Design Specifications §5.13.2.4.2. Both the Bracket and Corbel and the Strut and Tie methods recognize that direct shear is the primary behavioral mode instead of flexure, and is resisted by tension reinforcement across the shear plane. As a result of these methods, more reinforcement may be required in the top of the overhang than would be required if a normal cantilevered beam is assumed.

15.11 Temperature and Shrinkage Reinforcement

Temperature and shrinkage reinforcement design shall be in accordance with NYSDOT LRFD Bridge Design Specifications §5.10.8.
15.12 Protecting Reinforcement from Corrosion

Corrosion of reinforcing steel is a major concern for an aging infrastructure. Repairing and replacing damaged concrete caused by rusting reinforcing steel requires time, money and an imposition on the traveling public. There are technologies that slow or prevent this corrosion but this protection comes at a price. A balance must be struck between the higher initial cost of these technologies and the long term benefits of enhanced performance. As such, use of these technologies should not be indiscriminately included where the costs obviously outweigh the perceived benefit. However, the designer is encouraged to investigate the applicability of these technologies and recommend their use where appropriate.

The designer has three choices available for protecting reinforcement: corrosion inhibitors, coating the reinforcement (epoxy, galvanized) and corrosion resistant metal (stainless). The decision of which protection(s) to specify is dependent on a variety of factors including location within a structural element, cost, durability, ease of placement, expected service life, and importance of the structure. See the Prestressed Concrete Construction Manual (PCCM) for details on corrosion inhibitors.

In general, uncoated (plain) steel is the most economical choice when the concrete members provide adequate cover, and the reinforcement is not exposed to chlorides or other severe environments. For most other applications, epoxy or galvanized reinforcement is the proper choice.

Solid stainless steel and stainless steel clad reinforcement are appropriate when the added durability reduces cost, either long-term or during construction. This can occur when environmental conditions are particularly severe, when the cost of repairs is unusually high, due to heavy traffic or construction conditions, when design of concrete sections as uncracked under service load is not feasible and when cover is less than standard. In these situations solid stainless steel and stainless steel clad reinforcement will continue to be effective because it will not detrimentally corrode.

Examples of situations where stainless steel bars or equal shall be used include:

- Concrete decks with high-volume (Two-way AADT = 50,000±, One way AADT = 25,000±) roadways where the additional cost for more durable reinforcement is outweighed by the future costs associated with traffic delays, safety of the workers and traveling public and costs to businesses served by that roadway
- Extreme environments such as a substructure located in or near a body of salt water or highly corrosive industrial area.
- Exposed areas of a pier cap beam beneath an expansion joint.

Examples of situations where stainless steel bars, stainless steel clad bars or equal should be used include:

- Work on a signature structure where construction work is difficult and detracts from the image that the structure conveys about the surrounding community.

Although there are situations where use of more durable reinforcing steel may be justified, the engineer must remember that the situations where epoxy-coated, galvanized and plain bars are the better choice are far more common. Use of solid stainless steel and stainless steel clad...
reinforcement is unnecessary in concrete members that have adequate cover, no exposure to chlorides, and corrosion protection methods are used such as low-permeability concrete or corrosion inhibitors.

Table 15-3 compares approximate current cost ratio estimates for reinforcing bars at the time of publication using plain reinforcing bars as a base. Please note that prices change over time and vary by geographic location. Designers should check current prices when cost is a consideration.

<table>
<thead>
<tr>
<th>Bar Protection Type</th>
<th>In-Place Cost Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid Stainless Steel</td>
<td>2.0</td>
</tr>
<tr>
<td>Stainless Steel Clad</td>
<td>1.6</td>
</tr>
<tr>
<td>Galvanized</td>
<td>1.1</td>
</tr>
<tr>
<td>Epoxy Coated</td>
<td>1.1</td>
</tr>
<tr>
<td>Plain</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 15-3
Approximate Reinforcement Cost Comparison

A review of the average bid prices (in place costs) indicates that the cost to fabricate, ship, and place plain reinforcing bars is $0.56/lb over the material cost. The cost to fabricate, ship, and place epoxy-coated bars is an additional $0.14/lb ($0.70/lb over the material cost) due to the extra care required during placement and repair to the epoxy coating after placement. In the above table, it was estimated that the cost to fabricate, ship, and place solid stainless steel bars is similar to the cost for plain bars and that the cost for stainless steel clad bars falls between the costs for plain and epoxy-coated bars.

The cost of solid stainless steel bars is expected to fall in the future. See Section 15.12.4.

Table 15-4 illustrates the expected service life for the different types of reinforcing bars in conventional concrete with standard cover exposed to a corrosive environment:
Section 16
Estimate of Quantities

16.1 General

The Engineer’s Estimate is the Department’s estimate of the construction cost of the project. The bridge estimate is an important component of that estimate and the contract plans for many reasons. Besides providing a list of quantities to the contractor, the estimate also provides some very important internal information to the Department. By breaking down the materials and tasks required for a bridge into measurable standard units and then dividing the bid price by the number of units, it is possible to establish a “per-unit” cost for each item bid for that particular project. These “per unit” costs are averaged with the “per-unit” costs from other similar projects. These averages can then applied to future projects to estimate the bid price.

Once these averages are well established, they can be used to determine the most cost efficient design between competing alternates. As an example, a determination could be made whether two continuous shorter spans with a pier are more economical than a single longer span bridge.

Since there is usually some highway approach work associated with a bridge project, the bridge estimate in most cases is only a part of the larger project estimate. The total project estimate is usually coordinated by the functional area having overall project management responsibility or responsibility for the highway portion of the project estimate.

All estimate calculations, and any sketches associated with them, shall be verified and preserved as part of the design computations. Estimate work up sheets are usually requested by the E.I.C. and should be provided before construction begins. Further information on estimates can be found in Section 14.2.6, Bridge Estimate File, and in Chapter 21 of the Highway Design Manual.

16.2 Precision Versus Practicality

It is important to consider the items being estimated and the relative amount of precision required for that item. For example, it may be necessary to estimate a certain item, such as a concrete placement, to the nearest tenth cubic yard in a concrete table, while it may be unnecessary to apply this level of accuracy to a less precise item such as earthwork items. The following is a sample list of the desired level of precision for the Estimate of Quantities Table:
### Table 16-1

**Precision for Estimate of Quantities**

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNITS</th>
<th>Level of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Select Structure Fill</td>
<td>Cubic Yards</td>
<td>Round to the nearest 5 cubic yards</td>
</tr>
<tr>
<td>Steel H-Piles</td>
<td>Foot</td>
<td>Round to the nearest foot</td>
</tr>
<tr>
<td>Perm. Steel Sheet Piling</td>
<td>Square Foot</td>
<td>Round to nearest square foot</td>
</tr>
<tr>
<td>Concrete for Structures</td>
<td>Cubic Yard</td>
<td>Round to nearest cubic yard (to nearest tenth in a placement table)</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>Pound</td>
<td>Round to the nearest 100 lbs</td>
</tr>
<tr>
<td>Armored Joint System</td>
<td>Foot</td>
<td>Round to nearest foot</td>
</tr>
<tr>
<td>Perm. Conc. Traffic Barrier</td>
<td>Foot</td>
<td>Round to the nearest foot</td>
</tr>
<tr>
<td>Type E.B. Bearings</td>
<td>Each</td>
<td>Give exact number required</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>Pound</td>
<td>Round to nearest pound</td>
</tr>
<tr>
<td>Stone Bridge Curb</td>
<td>Foot</td>
<td>Round to nearest foot</td>
</tr>
</tbody>
</table>

16.3 **Utility Share of Bridge Estimate**

It is common for bridges to carry utility lines (water or natural gas pipes, telephone or electrical lines, etc.) in addition to vehicular and pedestrian traffic. These projects may have separate utility shares in the Engineer’s Estimate. See Section 7.5 for more information on utility shares.
GENERAL NOTES SHEET

1. GENERAL NOTES

In the following notes, insert the month and year of the PS & E:

2. DESIGN SPECIFICATIONS: NEW YORK STATE DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES WITH ALL PROVISIONS IN EFFECT AS OF _________. (FOR DESIGN PURPOSES, COMPRESSIVE STRENGTH OF CONCRETE FOR SUBSTRUCTURES AND DECK SLABS AT 28 DAYS: $f'_c = 3000$ psi.)

or

3. DESIGN SPECIFICATIONS: NYSDOT LRFD BRIDGE DESIGN SPECIFICATIONS WITH ALL PROVISIONS IN EFFECT AS OF _________. (FOR DESIGN PURPOSES, COMPRESSIVE STRENGTH OF CONCRETE FOR SUBSTRUCTURES AND DECK SLABS AT 28 DAYS: $f'_c = 3000$ psi.)

The following live load notes are to be used for new and replacement bridges. On superstructure replacements, the existing substructures shall not be upgraded solely to accommodate these live load criteria.

4. LIVE LOAD: HS25 OR TWO 24,000 LB AXLES SPACED 4 FEET ON CTRS.

(Use only for bridges carrying either the mainline of Interstate highways or the Southern Tier Expressway designed with NYSDOT Standard Specifications for Highway Bridges.)

5. LIVE LOAD: HS25

(Use for all other highway bridges designed with NYSDOT Standard Specifications for Highway Bridges.)

6. LIVE LOAD: AASHTO HL-93 AND NYSDOT DESIGN PERMIT VEHICLE.

(Use for bridges designed by the LRFD specifications.)


Use the following two notes for structures carrying railroads.

8. DESIGN SPECIFICATIONS: CURRENT AMERICAN RAILWAY ENGINEERING AND MAINTENANCE ASSOCIATION MANUAL FOR RAILWAY ENGINEERING.

9. RAILROAD LIVE LOAD: COOPER E80.
10. **CONSTRUCTION AND MATERIALS SPECIFICATIONS**: STANDARD SPECIFICATIONS, CONSTRUCTION AND MATERIALS, NEW YORK STATE DEPARTMENT OF TRANSPORTATION, OFFICE OF ENGINEERING, DATED MAY 1, 2008, WITH CURRENT ADDITIONS AND MODIFICATIONS.

11. DETAILS ON THE DRAWINGS LABELED AS “NOT TO SCALE” ARE INTENTIONALLY DRAWN NOT TO SCALE FOR VISUAL CLARITY. ALL OTHER DETAILS FOR WHICH NO SCALE IS SHOWN ARE DRAWN PROPORTIONAL AND ARE FULLY DIMENSIONED.

12. ALL SHOP DRAWINGS SUBMITTED FOR THIS PROJECT SHALL BE IN US CUSTOMARY UNITS.


The following note may be used in lieu of the previous note:

14. THE COST OF WATER USED FOR COMPACTION OF SELECT FILL ITEMS SHALL BE INCLUDED IN THE UNIT PRICE BID FOR ITEM 203.21 – SELECT STRUCTURE FILL.

If an investigation of assumed construction loads determines that bracing beyond that typically necessary is required, the following note shall be placed on the plans to notify the Contractor:

15. **THE CONTRACTOR’S ATTENTION IS DIRECTED TO THE LARGE DECK OVERHANGS FOR THIS STRUCTURE. THE FASCIA GIRDER DESIGN ASSUMES TYPICAL CONSTRUCTION LOADS DURING PLACEMENT OF THE CONCRETE DECK. THE CONTRACTOR SHALL PROVIDE ADEQUATE TEMPORARY SUPPORT AND BRACING TO PREVENT THE FASCIA GIRDER FROM TWISTING OR EXCESSIVELY DEFLECTING UNDER THE LOADS OF THE CONCRETE DEAD LOAD AND THE CONSTRUCTION LOADS. ALL DESIGN EFFORT REQUIRED BY THE CONTRACTOR’S ENGINEER TO ASSURE THAT THE CONSTRUCTION LOADS DO NOT ADVERSELY AFFECT THE FASCIA GIRDER ARE AT THE CONTRACTOR’S EXPENSE. THE CONTRACTOR SHALL SUBMIT OVERHANG FORMING DESIGN CALCULATIONS AND DETAILS TO THE D.C.E.S. FOR APPROVAL.**

16. **THE CONTRACTOR’S ATTENTION IS DIRECTED TO SUBSECTION 105-09, WORK AFFECTING RAILROADS, OF THE STANDARD SPECIFICATIONS.**

The following note shall be used when Fracture-Critical Members are used in new construction. Designers shall also provide a table of fracture critical tension members on the Contract Plans. The designer should consult with the Metals Engineering Unit to confirm which items should be included in the table.
18. **THIS STRUCTURE CONTAINS FRACTURE-CRITICAL MEMBERS. THESE MEMBERS ARE IDENTIFIED ON THE PLANS. THE CONTRACTOR SHALL COMPLY WITH THE APPLICABLE PROVISIONS OF SECTION 9 OF THE NEW YORK STATE STEEL CONSTRUCTION MANUAL (SCM).**

The following note shall be used when Fracture-Critical Members are present in a rehabilitation project. Designers shall also provide a table of fracture critical tension members on the Contract Plans. The designer should consult with the Metals Engineering Unit to confirm which items should be included in the table.

19. **THIS STRUCTURE CONTAINS FRACTURE-CRITICAL MEMBERS. THESE MEMBERS ARE IDENTIFIED ON THE PLANS. IF REPAIRS TO THESE MEMBERS OR ADJACENT MEMBERS NEED TO BE DONE, THE CONTRACTOR SHALL COMPLY WITH THE APPLICABLE PROVISIONS OF SECTION 9 OF THE NEW YORK STATE STEEL CONSTRUCTION MANUAL (SCM).**

When a bridge is designed using a grid, three-dimensional or finite-element method of analysis; and has diaphragms and/or bracing members acting as primary members (load paths), the following note shall be placed on the plans to notify the Contractor of the special requirements for fabrication and erection of the bridge.

20. **THE STRUCTURE SHALL BE ASSEMBLED AS PER SECTION 1103.4.5 OF THE NYS STEEL CONSTRUCTION MANUAL (SCM). COST SHALL BE INCLUDED UNDER THE RESPECTIVE STRUCTURAL STEEL UNIT BID PRICE.**

The following note shall be included on steel viaduct and rigid frame bridge projects. The designer shall consult with the Metals Engineering Unit to confirm which items in the list require testing to meet minimum CVN values. Note: Fracture critical tension members must be identified in a table on the contract plans, as they require higher CVN values.

21. **IN ADDITION TO THE ITEMS LISTED IN §715-01, THE FOLLOWING ELEMENTS OF BIN XXXXXXXX SHALL BE FURNISHED TO MINIMUM CHARPY V-NOTCH FRACTURE TOUGHNESS REQUIREMENTS:**
   - CANTILEVER BRACKETS AND TIE PLATES
   - BENT COLUMNS AND BRACING SUBJECT TO TENSION

The following note shall be included on steel arch bridge projects. The designer shall consult with the Metals Engineering Unit to confirm which items in the list require testing to meet minimum CVN values. Note: Fracture critical tension members must be identified in a table on the contract plans, as they require higher CVN values.

22. **IN ADDITION TO THE ITEMS LISTED IN §715-01, THE FOLLOWING ELEMENTS OF BIN XXXXXXXX SHALL BE FURNISHED TO MINIMUM CHARPY V-NOTCH FRACTURE TOUGHNESS REQUIREMENTS:**
   - ARCH RIB FLANGE AND WEB PLATES SUBJECT TO TENSILE STRESS
   - ARCH RIB SPLICE PLATES
   - HANGERS
   - ARCH RIB LATERAL BRACING
The following note shall be included on steel movable bridge projects. The designer shall consult with the Metals Engineering Unit to confirm which items in the list require testing to meet minimum CVN values. Note: Fracture critical tension members must be identified in a table on the contract plans, as they require higher CVN values.

23. IN ADDITION TO THE ITEMS LISTED IN §715-01, THE FOLLOWING ELEMENTS OF BIN XXXXXXX SHALL BE FURNISHED TO MINIMUM CHARPY V-NOTCH FRACTURE TOUGHNESS REQUIREMENTS:
   - TENSION FLANGES AND WEBS PLATES OF BASCULE GIRDERs
   - CANTILEVER BRACKETS AND TIE PLATES
   - SPLICE PLATES
   - VERTICAL GUSSET PLATES
   - TOP AND BOTTOM LATERAL BRACING
   - SWAY FRAMES
   - END PORTAL FRAMES
   - TENSION COMPONENTS OF LIFTING BOXES AND LIFTING FRAMES

24. THE LOAD RATINGS ARE IN ACCORDANCE WITH THE AASHTO MANUAL FOR BRIDGE EVALUATION.

The following note shall be used when preparing structural plans:

25. DIMENSIONS FOR THICKNESSES OF STEEL ROLLED ANGLE SHAPES AND STRUCTURAL TUBING ARE SHOWN ACCORDING TO THE AISC MANUAL.

Maintenance Guideline Note (use for new and replacement contracts):

26. THIS BRIDGE SHALL BE MAINTAINED IN ACCORDANCE WITH THE GUIDELINES CONTAINED IN THE CURRENT EDITION OF THE AASHTO MAINTENANCE MANUAL: THE MAINTENANCE AND MANAGEMENT OF ROADWAYS AND BRIDGES.

Maintenance Guideline Note (use for rehabilitation contracts):

27. THIS BRIDGE, INCLUDING EXISTING ELEMENTS AND THOSE REPAIRED OR REPLACED UNDER THIS CONTRACT, SHALL BE MAINTAINED IN ACCORDANCE WITH THE GUIDELINES CONTAINED IN THE CURRENT EDITION OF THE AASHTO MAINTENANCE MANUAL: THE MAINTENANCE AND MANAGEMENT OF ROADWAYS AND BRIDGES.

The following asbestos caution notes shall be used when materials containing asbestos exist on a bridge and are not to be disturbed or removed:

28. THE CONTRACTOR IS CAUTIONED THAT MATERIALS CONTAINING ASPHALT ARE BELIEVED TO EXIST AT VARIOUS LOCATIONS ON OR IN CERTAIN STRUCTURES OF THIS CONTRACT. THESE MATERIALS WERE NOTED ON THE ORIGINAL CONTRACT PLANS OF THE STRUCTURES AND/OR DURING FIELD INSPECTIONS.
29. UNLESS OTHERWISE INDICATED ON THE PLANS, WORK TO BE PERFORMED UNDER THIS CONTRACT DOES NOT REQUIRE THE DISTURBING, DESTRUCTION OR REMOVAL OF ANY KNOWN MATERIALS CONTAINING ASBESTOS. UNLESS OTHERWISE INDICATED ON THE PLANS, IT IS THE EXPRESS INTENT OF THIS CONTRACT THAT THESE MATERIALS NOT BE DISTURBED IN ANY WAY. SHOULD THE CONTRACTOR BE FORCED TO DISTURB IN ANY WAY ANY SUCH MATERIALS, THE CONTRACTOR SHALL FIRST BE FAMILIAR WITH INDUSTRIAL CODE RULE 56 OF THE N.Y.S. DEPARTMENT OF LABOR. THE CONTRACTOR SHALL ALSO OBTAIN WRITTEN PERMISSION OF THE REGIONAL DIRECTOR OF TRANSPORTATION BEFORE PROCEEDING.

The following note shall be placed on the General Plan of each bridge which is in proximity to high voltage (600 volts or more) electric lines or systems:

30. HIGH VOLTAGE ELECTRICAL LINES ARE IN PROXIMITY TO THIS BRIDGE. REFER TO SUBSECTION 107-05 OF THE STANDARD SPECIFICATIONS FOR CONTRACTOR SAFETY REQUIREMENTS.

31. FOUNDATION NOTES

Indicate on the Contract Plans those notes recommended by the Foundations and Construction Unit in the "Foundation Design Report" (FDR).

32. SUBSTRUCTURE NOTES

33. ALL PLACEMENTS OF SELECT STRUCTURE FILL, ITEM 203.21, SHALL BE COMPACTED TO 95 PERCENT OF STANDARD PROCTOR MAXIMUM DENSITY.

34. WHERE PILES ARE TO BE PLACED THROUGH THE EMBANKMENT (6 INCH TOPSIZE), THE EMBANKMENT SHALL BE COMPACTED TO 95 PERCENT OF STANDARD PROCTOR MAXIMUM DENSITY.

35. HIGHWAY EMBANKMENT MATERIAL (FROM HIGHWAY ESTIMATE OR FROM STRUCTURE EXCAVATION BACKFILL) AND SELECT STRUCTURE FILL, ITEM 203.21, SHALL BE PLACED SIMULTANEOUSLY, IN CONTACT, ON BOTH SIDES OF THE VERTICAL PAYMENT LINE.

Use the following note when the deck slab is continuous over the backwall.

36. TOP OF BACKWALLS SHALL BE STEEL TROWEL FINISHED. SHEET GASKET (TREATED BOTH SIDES), 728-06, SHALL BE PLACED ON THE TOP OF THE BACKWALLS OF FIXED AND EXPANSION ABUTMENTS. TWO SHEETS SHALL BE USED; PAYMENT SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE APPROACH SLAB ITEM.

37. CLEANING CONCRETE EXPOSED TO VIEW:

If cleaning all or a portion of the substructure concrete is not required, as determined by the Regional Office, the following special note shall be included in the plans for the structure. The
note shall be modified as required for use when only portions of piers or abutments will be cleaned, or when a pier or abutment is cleaned and the rest of the substructure elements are not.

38. **THE PROVISIONS OF SECTION 555-3.08, FINISHING, WITH REGARD TO REMOVING RUST STAINS FROM CONCRETE EXPOSED TO VIEW ARE WAIVED. RUST STAINS SHALL NOT BE REMOVED FROM THE SUBSTRUCTURE ON THIS BRIDGE.**

39. **THE CONTRACTOR, WITH THE PERMISSION OF THE D.C.E.S., MAY ELECT TO INTRODUCE CONSTRUCTION JOINTS IN THE ABUTMENTS AT LOCATIONS NOT SHOWN ON THE PLANS. THESE CONSTRUCTION JOINTS SHALL BE PROVIDED WITH SHEAR KEYS AND WATERSTOPS. VERTICAL CONSTRUCTION JOINTS INTRODUCED IN THE BACKWALL SHOULD PREFERABLY BE PLACED MIDWAY BETWEEN THE PEDESTALS.**

Cofferdam Notes

40. **SHOULD THE CONTRACTOR ELECT TO LAY BACK A PORTION OF THE EXISTING EARTH ADJACENT TO AN EXCAVATION REQUIRING A COFFERDAM, ANY REQUIRED EXTENSIONS OF THE COFFERDAM NECESSARY TO KEEP WATER FROM ENTERING THE EXCAVATION SHALL BE FURNISHED AND PLACED AT NO COST TO THE STATE.**

41. **WHERE A COFFERDAM IS USED, THE COST OF DEWATERING THE ENTIRE EXCAVATION, REGARDLESS OF SOURCE OF WATER, SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE COFFERDAM ITEM.**

42. **DELETED**

43. **SHOULD FIELD CONDITIONS REQUIRE A CHANGE FROM THE TYPE OF COFFERDAM SYSTEM CALLED FOR ON THE PLANS, THE ENGINEER-IN-CHARGE SHALL CONTACT THE D.C.E.S. FOR COORDINATION WITH APPROPRIATE AGENCIES TO APPROVE THE CHANGE.**

Include the following note on the contract plans when cofferdams are used with a tremie system:

44. **THE COFFERDAM AND TREMIE SYSTEM SHALL BE DESIGNED TO AUTOMATICALLY FLOOD BY NON-MECHANICAL MEANS WHEN THE WATER ELEVATION EXCEEDS ___________ .**

Include the following notes on the contract plans as applicable:

45. **IF MULTIPLE COFFERDAMS ARE REPLACED BY A SINGLE SYSTEM, AS PERMITTED BY THE REGIONAL HYDRAULICS ENGINEER, PAYMENT SHALL BE BASED ON ALL OF THE APPLICABLE COFFERDAM ITEMS INDICATED ON THE PLANS.**
46. DEWATERING OF THE COFFERDAM SHALL BE ACCOMPLISHED BY PUMPING THE WATER TO AN APPROVED UPLAND VEGETATED AREA OUTSIDE OF THE STREAMBED AS SHOWN ON THE PLANS AND/OR APPROVED BY THE E.I.C. TEMPORARY SOIL EROSION AND WATER POLLUTION CONTROL, SUCH AS STRAW BALES OR APPROVED EQUAL, MAY BE REQUIRED AS DETERMINED BY THE ENGINEER-IN-CHARGE. NO SETTLEMENT BASIN SHALL BE CONSTRUCTED.

47. THE CONTRACTOR SHALL HAVE THE OPTION OF INSTALLING A SEPARATE COFFERDAM OR INCORPORATING THE PERMANENT SHEETING INTO THE COFFERDAM ITEM.

48. IF THE CONTRACTOR ELECTS TO INCORPORATE THE PERMANENT SHEETING IN THE COFFERDAM ITEM, THE CONTRACTOR SHALL BE REQUIRED TO PROVIDE ANY ADDITIONAL BRACING REQUIRED TO STRENGTHEN THE PERMANENT SHEETING SYSTEM AND PROVIDE ANY WORK NECESSARY TO RETURN THE PERMANENT SHEETING TO ITS INTENDED FUNCTION AFTER THE COFFERDAM FUNCTION IS COMPLETE.

The following note shall be provided to specify water elevations developed for use at this location. They have been obtained by field observations from Regional forces at the time of preparation of the Bridge Site Data submission and they are included in Bridge Data Sheet #2.

49. ORDINARY HIGH WATER IS ESTIMATED TO BE ______. THIS IS DEFINED AS THE WATER SURFACE ELEVATION FOR THE MEAN ANNUAL FLOOD, WHICH IS THE FLOOD THAT HAS A RECURRENCE INTERVAL OF 2.33 YEARS.

ORDINARY WATER IS ESTIMATED TO BE_______. THIS IS DEFINED AS THE HIGHEST SURFACE WATER ELEVATION LIKELY TO BE ENCOUNTERED DURING ONE CONSTRUCTION SEASON (OTHER THAN MAJOR FLOODS). IT IS ALWAYS LESS THAN THE ORDINARY HIGH WATER ELEVATION AND IT IS USUALLY AN OBSERVED ELEVATION RATHER THAN A COMPUTED ONE.

LOW WATER IS ESTIMATED TO BE ______. THIS WATER ELEVATION IS THE NORMAL LOW WATER ELEVATION PREVALENT DURING ONE CONSTRUCTION SEASON FOR MORE THAN 25% OF THE TIME. IT IS AN OBSERVED ELEVATION RATHER THAN A COMPUTED ONE.

50. SUPERSTRUCTURE NOTES

Use the following note to designate the type of structural steel to be used. Use ASTM designations. If different types of structural steel are used for different components, modify the note accordingly.

51. ALL STRUCTURAL STEEL SHALL CONFORM TO ASTM A709-GRADE ________.

52. THE CONTRACTOR'S ATTENTION IS DIRECTED TO THE PROVISIONS OF THE CURRENT SPECIFICATIONS FOR SUPERSTRUCTURE SLABS, WHICH ALLOW THE OPTION OF 3 FORMING SYSTEMS FOR THE UNDERSIDE OF THE SLABS.
However, if the designer believes that one or more of the form options is inappropriate for a given bridge, or if isotropic reinforcement is used in the deck, the following shall be added to the previous note:

53. **HOWEVER, ON THIS BRIDGE, ONLY THE FOLLOWING OPTION(S) WILL BE PERMITTED:** (List options. In the case of isotropic deck reinforcement, only permanent corrugated metal and removable wooden forms are allowed).

Use the following note when a structural slab is to be placed on steel girders if the depth of the girder web exceeds 4 ft.

54. **IN ORDER TO PREVENT MOVEMENT OF THE BRIDGE OVERHANG BRACKET DURING THE DECK CONCRETE PLACEMENT, AS WELL AS TO PREVENT LATERAL DISTORTION OF THE GIRDER WEB, A DEEP OVERHANG BRACKET THAT IS BRACED BY THE BOTTOM FLANGE SHALL BE USED.**

55. **NO DEVIATIONS FROM THE HAUNCH DETAILS SHOWN ON THESE PLANS MAY BE MADE WITHOUT THE PERMISSION OF THE D.C.E.S.**

56. **CLEANING CONTROLLED OXIDIZING STRUCTURAL STEEL ASTM A709 GRADE 50W.**

   A. **IN THE FABRICATION SHOP**

   GIRDERS SHALL BE BLAST CLEANED IN ACCORDANCE WITH SSPC-SP6 (COMMERCIAL BLAST CLEANING). HEAVY COATINGS OF OIL OR GREASE SHALL BE REMOVED BEFORE BLASTING IN ACCORDANCE WITH SSPC-SP1 (SOLVENT CLEANING).

   B. **IN THE FIELD**

   THE OUTSIDE SURFACE OF THE FASCIA STRINGERS SHALL BE CLEANED SO THAT ALL DIRT, GREASE, PAINT OR OTHER FOREIGN MATERIAL IS REMOVED AT THE COMPLETION OF THE BRIDGE CONSTRUCTION. THE PURPOSE OF THE CLEANING IS TO RETURN THE FASCIA SURFACES TO THE CONDITION IN WHICH THEY LEFT THE FABRICATION SHOP.

57. **THE COST OF CLEANING THIS STEEL IN THE FABRICATION SHOP AND THE FIELD SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE VARIOUS ITEMS IN THE CONTRACT.**

One of the following special notes shall be included with the superstructure for all steel bridges:

58. **THE STRUCTURAL STEEL FOR THIS BRIDGE SHALL NOT BE PAINTED.**

   or

59. **THE STRUCTURAL STEEL FOR THIS BRIDGE SHALL BE COMPLETELY PAINTED. FINISH COAT COLOR SHALL BE_________. THE COLOR SHALL CONFORM TO __________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT.**

   (Designer shall designate color and either Federal Color Standard No. 595 number or Munsell Book Notation number to which color conforms.) **THERE ARE__________ SQUARE FEET OF PAINTED STRUCTURAL STEEL ON THIS BRIDGE.** (Designer shall indicate area to nearest 100 ft².)
60. **THE STRUCTURAL STEEL FOR THIS BRIDGE SHALL BE PARTIALLY PAINTED.** FINISH COAT COLOR SHALL BE __________. THE COLOR SHALL CONFORM TO __________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT. THE FOLLOWING PORTIONS OF THE STEEL SHALL BE PAINTED: ALL EXPOSED SURFACES OF THE FASCIA STRINGERS INCLUDING ANY STIFFENERS OR CONNECTION PLATES, AND __________. (Designer shall designate color and either Federal Color Standard No. 595 number or Munsell Book Notation number to which color conforms, and any additional surfaces that are to be painted). THERE ARE _________ SQUARE FEET OF PAINTED STRUCTURAL STEEL ON THIS BRIDGE.

or

61. **THE STRUCTURAL STEEL FOR THIS BRIDGE SHALL BE PARTIALLY PAINTED.** FINISH COAT COLOR SHALL BE __________. THE COLOR SHALL CONFORM TO __________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT. THE FOLLOWING PORTIONS OF THE STEEL SHALL BE PAINTED: ALL EXPOSED SURFACES OF THE STRINGERS THAT ARE WITHIN A DISTANCE OF 1.5 TIMES THE DEPTH OF THE GIRDER FROM THE BRIDGE JOINTS INCLUDING ANY STIFFENERS OR CONNECTION PLATES. (Designer shall designate color and either Federal color Standard No. 595 number or Munsell Book Notation number to which color conforms, and any additional surfaces that are to be painted.) THERE ARE _________ SQUARE FEET OF PAINTED STRUCTURAL STEEL ON THIS BRIDGE. (This note shall be used on jointed bridges constructed of weathering steel where the steel is to be painted within a distance of 1.5 times the depth of the girder from the joint. The designer shall indicate the painting limits on the plans.)

or

62. **THE STRUCTURAL STEEL FOR THIS BRIDGE SHALL BE PARTIALLY PAINTED.** FINISH COAT COLOR SHALL BE __________. THE COLOR SHALL CONFORM TO __________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT. THE FOLLOWING PORTIONS OF THE STEEL SHALL BE PAINTED: ALL EXPOSED SURFACES OF THE STRINGERS THAT ARE WITHIN A DISTANCE OF 1.5 TIMES THE DEPTH OF THE GIRDER FROM THE BRIDGE JOINTS INCLUDING ANY STIFFENERS OR CONNECTION PLATES AND ALL EXPOSED SURFACES OF THE FASCIA STRINGERS INCLUDING ANY STIFFENERS OR CONNECTION PLATES. (Designer shall designate color and either Federal color Standard No. 595 number or Munsell Book Notation number to which color conforms, and any additional surfaces that are to be painted.) THERE ARE _________ SQUARE FEET OF PAINTED STRUCTURAL STEEL ON THIS BRIDGE. (This note shall be used on jointed bridges constructed of weathering steel where the steel is to be painted within a distance of 1.5 times the depth of the girder from the joint and the fascia girders are to be painted. The designer shall indicate the painting limits on the plans.)

The following note shall be placed on the plans when Galvanized Surfaces are to be painted:

63. **ANY GALVANIZED SURFACES REQUIRED TO BE PAINTED SHALL BE PAINTED IN ACCORDANCE WITH THE REQUIREMENTS OF SECTION 657 OF THE STANDARD SPECIFICATIONS.** FINISH COAT COLOR SHALL BE __________. THE COLOR SHALL CONFORM TO __________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT. THE COST OF THIS WORK SHALL BE INCLUDED IN THE UNIT PRICE BID FOR ITEM __________. (The designer shall designate color from Section 708-05 or either Federal Color Standard No. 595 number or Munsell Book Notation number to which color conforms. In addition, the designer shall choose the proper pay item to cover painting work of this type. Generally, if the project employs structural steel painting items, galvanized surface
painting costs can be included in those items. If not, the designer should choose the most appropriate item; e.g., railing, downspout etc.)

64. FOR THE VARIOUS LUMP SUM STRUCTURAL STEEL ITEMS IN THE CONTRACT, THE "TOTAL WEIGHT FOR PROGRESS PAYMENT" IS AS FOLLOWS:

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These weights shall be used in determining partial payments and progress. Under no circumstances shall the "TOTAL WEIGHT FOR PROGRESS PAYMENT" be used for final payment purposes. The contractor is advised not to use the "TOTAL WEIGHT FOR PROGRESS PAYMENT" as a bidding tool. Discrepancies which may occur between the total weight shipped and "TOTAL WEIGHT FOR PROGRESS PAYMENT" shall not be a basis for additional compensation.

One of the following notes shall be included with the superstructure for all steel bridges with straight girders that do not have integral abutments:

65. Diaphragms for skewed straight girder superstructures shall be fabricated to fit girders erected with their webs laid over (out of plumb) under the steel dead load condition. Girder webs shall be vertical after application of full dead load.

or

66. Diaphragms for nonskewed straight girder superstructures shall be fabricated to fit girders erected with their webs vertical under steel and full dead load conditions.

The following note shall be included with the superstructure for all steel bridges with curved girders that do not have integral abutments:

67. Diaphragms for all curved girder superstructures shall be fabricated to fit girders erected with their webs laid over (out of plumb) under the steel dead load condition. Girder webs shall be vertical after application of full dead load.

The following notes A1 or A2 and note B shall be used when structural steel is to be erected. Use note A1 if the girder fails the stability check or A2 if the girder passes the stability check based on the results required by NYSDOT LRFD Blue Page 6.10.3.1a.

68. STEEL ERECTION NOTES:

A1. The contractor shall provide for the stability of structural steel during all phases of erection and construction, as provided in subsection 204 of the New York State Steel Construction Manual (SCM). The girders on this bridge shall be stabilized during erection by use of falsework, temporary
BRACING, COMPRESSION FLANGE STIFFENING TRUSSES, CHOOSING ALTERNATE PICKING POINTS, OR BY USE OF A HOLDING CRANE UNTIL A SUFFICIENT NUMBER OF GIRDERS HAVE BEEN ERECTED AND CROSS FRAMES INSTALLED. THE METHODS USED BY THE CONTRACTOR SHALL BE DOCUMENTED ON THE ERECTION DRAWINGS WITH ALL SUPPORTING STABILITY CALCULATIONS SUBMITTED AND STAMPED BY A LICENSED NEW YORK STATE PROFESSIONAL ENGINEER AND SUBMITTED TO THE DCES IN ACCORDANCE WITH THE SCM.

A2. THE CONTRACTOR SHALL PROVIDE FOR THE STABILITY OF STRUCTURAL STEEL DURING ALL PHASES OF ERECTION AND CONSTRUCTION, AS PROVIDED IN SUBSECTION 204 OF THE NEW YORK STATE STEEL CONSTRUCTION MANUAL (SCM). THE METHODS USED BY THE CONTRACTOR SHALL BE DOCUMENTED ON THE ERECTION DRAWINGS WITH ALL SUPPORTING STABILITY CALCULATIONS SUBMITTED AND STAMPED BY A LICENSED NEW YORK STATE PROFESSIONAL ENGINEER AND SUBMITTED TO THE DCES IN ACCORDANCE WITH THE SCM.

B. THE DESIGN OF THIS STRUCTURE ASSUMES THAT THE STRUCTURAL STEEL IS COMPLETELY ERECTED BEFORE IT IS ALLOWED TO DEFLECT UNDER ITS OWN DEAD LOAD. DEFLECTIONS INCURRED DURING THE VARIOUS STAGES OF THE ERECTION METHOD ARE NOT CONSIDERED. THEREFORE, THE ACTUAL ERECTION METHODS AND SEQUENCES EMPLOYED BY THE CONTRACTOR MAY HAVE A SUBSTANTIAL EFFECT ON THE FINAL STEEL PROFILE. THE CONTRACTOR SHALL BE RESPONSIBLE FOR TAKING ALL NECESSARY COMPENSATORY ACTION TO ENSURE THAT THE FINAL ALIGNMENT AND PROFILE OF THE ERECTED STEEL CONFORMS TO SUBSECTION 1213, 1214, AND 1215 OF THE NEW YORK STATE STEEL CONSTRUCTION MANUAL (SCM). ANY CORRECTIVE WORK NECESSARY TO RE-POSITION PREVIOUSLY ERECTED STEEL TO ACHIEVE ACCEPTABLE ALIGNMENT AND PROFILE MUST BE APPROVED BY THE D.C.E.S., AND SHALL BE PERFORMED AT NO ADDITIONAL COST TO THE STATE.

69. IF THE CONTRACTOR ELECTS TO MOVE THE SPLICE LOCATION SHOWN ON THE PLANS, IT IS THE CONTRACTOR’S RESPONSIBILITY TO HAVE A NEW YORK STATE PROFESSIONAL ENGINEER REDESIGN THE SPLICE. COST OF REDESIGN TO BE INCLUDED IN THE STEEL BID ITEM.

Use one of the following two notes if a Concrete Barrier, the payment for which includes its reinforcement, is used on the bridge:

70. THE DETAILS FOR THE BARRIER REINFORCEMENT ARE FOR THE SLIP-FORMED OR CAST-IN-PLACE OPTION ONLY. COST OF BARRIER AND ANCHORAGE REINFORCEMENT ORIGINATING IN THE SLAB SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE BARRIER ITEM.

or

71. THE DETAILS FOR THE BARRIER REINFORCEMENT ARE FOR THE SLIP-FORMED OR CAST-IN-PLACE OPTION ONLY. COST OF BARRIER AND ANCHORAGE REINFORCEMENT ORIGINATING IN THE SLAB SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE BARRIER ITEM. COST OF BARRIER ANCHORAGE REINFORCEMENT ORIGINATING IN THE PRESTRESSED UNIT SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE PRESTRESSED UNIT ITEM.
Use the following note if single slope concrete barrier is specified and service level TL-5 is required:

72. THE CONTRACTOR'S ATTENTION IS DIRECTED TO THE PROVISIONS OF THE CURRENT SPECIFICATIONS FOR PERMANENT CONCRETE TRAFFIC BARRIER FOR STRUCTURES, WHICH ALLOWS THE OPTION OF THREE CONSTRUCTION METHODS: CAST-IN-PLACE, SLIP FORMED, OR PRECAST. HOWEVER, ON THIS BRIDGE, ONLY CAST-IN-PLACE AND SLIP FORMING ARE ALLOWED.

Use the following note if steel bridge railing is used on the bridge and any of the situations described in section 6.9 of this manual occur:

73. FOR BIN XXXXXXXX, SHOP DRAWING SUBMITTALS ARE REQUIRED FOR THE FOLLOWING BRIDGE RAIL/TRANSITION ITEMS: 568.XX,...

Use the following note when Protective Sealer is to be applied to new bridge decks and approach slabs:

74. TOP SURFACES OF NEW BRIDGE DECKS AND APPROACH SLABS SHALL BE SEALED ACCORDING TO ITEM 559.1896 18–PROTECTIVE SEALING OF STRUCTURAL CONCRETE ON NEW BRIDGE DECKS AND BRIDGE DECK OVERLAYS.

Use the following note whenever Open Steel Floor Grating is used on structures. If the grating is specified to be painted, Note 63 shall also be used.

75. OPEN STEEL FLOOR GRATING SHALL BE GALVANIZED IN ACCORDANCE WITH THE REQUIREMENTS OF 719-01 GALVANIZED COATINGS AND REPAIR METHODS OF THE STANDARD SPECIFICATIONS.

76. REMOVAL NOTES

The following two notes shall be used when the project is replacing an existing structure. The preliminary bridge plans must indicate location on General Plan or Location Plan:

77. EXISTING SUBSTRUCTURE SHALL BE REMOVED WITHIN THE LIMITS SHOWN ON THE PLANS UNDER ITEM 202.19 IN THE BRIDGE ESTIMATE.

78. EXISTING SUPERSTRUCTURE SHALL BE REMOVED UNDER ITEM 202.12nnnn IN THE BRIDGE ESTIMATE.

Use one of the following two notes when structures longer than 20 ft. are being removed. Refer to Appendix 17A for guidance on determining whether a removal plan prepared by a Professional Engineer is required.

79. THE CONTRACTOR'S ATTENTION IS DIRECTED TO THE REQUIREMENTS OF SUBSECTION 202-3.01 GENERAL AND SAFETY REQUIREMENTS. A REMOVAL PLAN, SIGNED BY A REGISTERED PROFESSIONAL ENGINEER IN THE STATE OF NEW YORK, SHALL BE SUBMITTED TO THE ENGINEER THIRTY (30) DAYS PRIOR TO BEGINNING THE DEMOLITION.
or

80. THE CONTRACTOR'S ATTENTION IS DIRECTED TO THE REQUIREMENTS OF SUBSECTION 202-3.01 GENERAL AND SAFETY REQUIREMENTS. A REMOVAL PLAN SHALL BE SUBMITTED TO THE ENGINEER FIFTEEN (15) DAYS PRIOR TO BEGINNING THE DEMOLITION. THE REQUIREMENT THAT IT BE SIGNED BY A REGISTERED PROFESSIONAL ENGINEER IS WAIVED.

In addition to one of the above notes, either of the following should also be placed on the Contract Plans:

81. RECORD PLANS FOR THIS STRUCTURE ARE AVAILABLE AT THE REGIONAL OFFICE OF THE DEPARTMENT OF TRANSPORTATION.  

or

82. RECORD PLANS FOR THIS STRUCTURE ARE NOT AVAILABLE.

Use Note 83, and, if applicable, Note 84 and Note 85 if a steel superstructure containing lead-based paint is being removed:

83. SUPERSTRUCTURE (OR SUBSTRUCTURE) REMOVAL NOTES:

LIMITS AND METHODS FOR REMOVAL OF PAINT AT LOCATIONS OF FASTENER REMOVAL OR FLAME CUTTING SHALL BE AS DESCRIBED IN SUBSECTIONS 202-3.05 AND 574 OF THE STANDARD SPECIFICATIONS. THE COST OF PAINT REMOVAL SHALL BE INCLUDED IN THE LUMP SUM PRICE(S) BID FOR THE SUPERSTRUCTURE REMOVAL ITEM(S) (OR THE UNIT PRICE BID FOR THE SUBSTRUCTURE REMOVAL ITEM). PAINT WASTE NOT COLLECTED BY VACUUM METHODS SHALL BE COLLECTED USING THE ENVIRONMENTAL GROUND AND/OR WATERWAY PROTECTION ITEM(S). WASTE SHALL BE DISPOSED OF USING THE TREATMENT AND DISPOSAL OF PAINT REMOVAL WASTE ITEM.

In addition to paint removal described above at locations of dismantling and removal operations, there may exist areas of loose or peeling paint on various steel surfaces which are likely to become dislodged during removal operations or during transportation from the site. If this condition is confirmed, either by referring to the latest Bridge Inspection Report, by observation by the designer or by Regional personnel at the request of the designer, the following note should be placed on either the General Notes sheet or the Superstructure (or Substructure) Removal sheet:

84. LOOSE AND/OR PEELING PAINT ON STEEL SURFACES MAY BECOME DISLODGED DURING REMOVAL OPERATIONS OR DURING TRANSPORTATION FROM THE SITE UNLESS APPROPRIATE MEASURES ARE TAKEN. THE CONTRACTOR SHALL FORMULATE AND SUBMIT A METHOD OF REMEDIATING THE CONDITION FOR APPROVAL BY THE ENGINEER. WORKER LEAD PROTECTION IN ACCORDANCE WITH OSHA 1926.62 MUST BE SATISFIED. ALTERNATIVES COULD INCLUDE TRANSPORTING AFFECTED MEMBERS IN CLOSED TRUCKS, WRAPPING AFFECTED MEMBERS PRIOR TO REMOVAL, ENCAPSULATING THE LOOSE PAINT OR REMOVAL OF LOOSE PAINT PRIOR TO DISMANTLING OPERATIONS. THE COST OF REMEDIATING THIS CONDITION SHALL BE INCLUDED IN THE LUMP SUM PRICE(S) BID FOR THE SUPERSTRUCTURE REMOVAL ITEM(S) (OR THE UNIT PRICE BID FOR THE SUBSTRUCTURE REMOVAL ITEM.) THE USE OF ENVIRONMENTAL GROUND AND/OR

85. REFER TO SUBSECTION 107-05 OF THE STANDARD SPECIFICATIONS FOR SAFETY AND HEALTH REQUIREMENTS.

86. RECONSTRUCTION NOTES

Use Notes 87-93 on all reconstruction projects.

87. THE CONTRACTOR’S ATTENTION IS DIRECTED TO THE FACT THAT, DUE TO THE NATURE OF RECONSTRUCTION PROJECTS, THE EXACT EXTENT OF RECONSTRUCTION WORK CANNOT ALWAYS BE ACCURATELY DETERMINED PRIOR TO THE COMMENCEMENT OF WORK. THESE CONTRACT DOCUMENTS HAVE BEEN PREPARED BASED ON FIELD INSPECTION AND OTHER INFORMATION AVAILABLE AT THE TIME. ACTUAL FIELD CONDITIONS MAY REQUIRE MODIFICATIONS TO CONSTRUCTION DETAILS AND WORK QUANTITIES. THE CONTRACTOR SHALL PERFORM THE WORK IN ACCORDANCE WITH FIELD CONDITIONS.

88. THE CONTRACTOR SHALL VERIFY DIMENSIONS NECESSARY FOR THE PROPER FIT OF STEEL PIECES PRIOR TO THE FABRICATION OF THE STEEL. THE COST OF FIELD VERIFYING DIMENSIONS SHALL BE INCLUDED IN THE PRICE BID FOR STRUCTURAL STEEL ITEMS.

89. THE CONTRACTOR SHALL PERFORM ALL WORK WITH CARE SO THAT ANY MATERIALS WHICH ARE TO REMAIN IN PLACE, OR WHICH ARE TO REMAIN THE PROPERTY OF THE STATE, WILL NOT BE DAMAGED. IF THE CONTRACTOR DAMAGES ANY MATERIALS WHICH ARE TO REMAIN IN PLACE OR WHICH ARE TO REMAIN THE PROPERTY OF THE STATE, THE DAMAGED MATERIALS SHALL BE REPAIRED OR REPLACED IN A MANNER SATISFACTORY TO THE ENGINEER AT THE EXPENSE OF THE CONTRACTOR.

90. WHenever ITEMS IN THE CONTRACT REQUIRE MATERIALS TO BE REMOVED AND DISPOSED OF, THE COST OF SUPPLYING A DISPOSAL AREA AND TRANSPORTATION TO THAT AREA SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THOSE ITEMS.

91. DURING REMOVAL OPERATIONS, THE CONTRACTOR SHALL NOT BE ALLOWED TO DROP WASTE CONCRETE, DEBRIS AND OTHER MATERIAL TO THE AREA BELOW THE BRIDGE EXCEPT WHERE THE PLANS SPECIFICALLY PERMIT THE DROPPING OF MATERIAL. PLATFORMS, NETS, SCREENS OR OTHER PROTECTIVE DEVICES SHALL BE USED TO CATCH THE MATERIAL. IF THE ENGINEER DETERMINES THAT ADEQUATE PROTECTIVE DEVICES ARE NOT BEING EMPLOYED, THE WORK SHALL BE SUSPENDED UNTIL ADEQUATE PROTECTION IS PROVIDED.

92. ALL MATERIAL FALLING ON THE AREA BELOW AND ADJACENT TO THE BRIDGE SHALL BE REMOVED AND DISPOSED OF BY THE CONTRACTOR AT NO COST TO THE STATE.
93. THE COST OF FURNISHING, INSTALLING, MAINTAINING, REMOVING AND DISPOSING OF ALL PLATFORMS, NETS, SCREENS OR OTHER PROTECTIVE DEVICES SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE APPROPRIATE ITEMS OF THE CONTRACT.

Use Notes 94-104 as needed on reconstruction contracts:

94. THE DETAILS ON DRAWING NO. ___ INDICATE THE SPALLS, SCALES AND CRACKS NOTED ON A FIELD INSPECTION BY THE DESIGNER. ALL OF THE MAJOR AREAS OF SPALLING, SCALING AND CRACKING KNOWN TO EXIST AT THE TIME OF CONTRACT PREPARATION HAVE BEEN SHOWN TO INDICATE THE APPROXIMATE EXTENT OF DETERIORATION THAT WILL HAVE TO BE REPAIRED BY THE CONTRACTOR.

95. THE CONTRACTOR SHALL PROVIDE THE EIC ACCESS TO ALL PIER SURFACES FOR SOUNDING TO DETERMINE AND MARK OUT REMOVAL LIMITS. THE EIC SHALL SUBMIT LABELED DIGITAL PHOTOS OF THE PIER TO THE REGIONAL STRUCTURES ENGINEER ALONG WITH THE CONTRACTOR’S PROPOSED REMOVAL AND REPLACEMENT SEQUENCE FOR APPROVAL. 10 CALENDAR DAYS SHALL BE ALLOWED FOR REVIEW AND APPROVAL. REMOVAL OF CONCRETE SHALL NOT COMMENCE WITHOUT AN APPROVED REMOVAL PLAN. THE COST FOR THESE ITEMS SHALL BE INCLUDED IN THE REMOVAL AND REPLACEMENT CONCRETE ITEMS.

If the designer determines there is sufficient volume of concrete repair work required to justify the use of Shotcrete (40 - 60 bags of cement minimum), the following note should be used:

96. AREAS OF CONCRETE DETERIORATION THAT ARE GENERALLY 5 INCHES OR LESS IN DEPTH (LOCALIZED POCKETS MAY BE UP TO 12 INCHES DEEP) SHALL BE REPAIRED USING ITEM 583.02 – REMOVAL OF STRUCTURAL CONCRETE – REPLACEMENT WITH SHOTCRETE, NO REINFORCEMENT BAR ENCASEMENT OR ITEM 583.03 – REMOVAL OF STRUCTURAL CONCRETE-REPLACEMENT WITH SHOTCRETE, REINFORCEMENT BAR ENCASEMENT, AS APPROPRIATE.

AREAS THAT ARE GREATER IN DEPTH SHALL BE REPAIRED USING ITEM 582.05 – REMOVAL OF STRUCTURAL CONCRETE REPLACEMENT WITH CLASS A CONCRETE. THESE GUIDELINES ARE APPROXIMATE, AND THE FINAL DETERMINATION OF WHICH ITEM TO USE SHALL BE MADE BY THE ENGINEER.

If the designer determines there is not sufficient volume of concrete repair work required to justify the use of Shotcrete, the following note should be used:

97. AREAS OF CONCRETE DETERIORATION SHALL BE REPAIRED USING ITEM 582.05 - REMOVAL OF STRUCTURAL CONCRETE - REPLACEMENT WITH CLASS A CONCRETE, ITEM 582.06 - REMOVAL OF STRUCTURAL CONCRETE - REPLACEMENT WITH CLASS D CONCRETE, OR ITEM 582.07 - REMOVAL OF STRUCTURAL CONCRETE - REPLACEMENT WITH VERTICAL AND OVERHEAD PATCHING MATERIAL AS SHOWN ON THE PLANS OR AS ORDERED BY THE ENGINEER.
98. ALL CONCRETE SURFACES RECEIVING NEW CONCRETE SHALL BE SANDBLASTED. PRIOR TO THE APPLICATION OF NEW CONCRETE, THE SURFACES SHALL BE AIR CLEANED THEN PRE-WET FOR 12 HOURS. THERE WILL BE NO SEPARATE PAYMENT FOR THIS WORK. THE COST SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE VARIOUS CONCRETE ITEMS IN THE CONTRACT.

The following two Notes shall be used with caution, as this work is normally covered in the specifications. They should be used only if a special weight of hammer is necessary for limited areas. Use Note 99 for partial removals if the concrete to be removed is unsound. Use Note 100 for partial removals if the concrete to be removed is sound. Generally for concrete to be considered sound, the aggregate must fracture when struck with a hammer.

99. CHIPPING HAMMERS USED TO REMOVE CONCRETE FROM THE FOLLOWING STRUCTURAL COMPONENTS SHALL NOT EXCEED 25 LB. IN WEIGHT WITH THE BIT REMOVED.

or

100. CHIPPING HAMMERS USED TO REMOVE CONCRETE FROM THE FOLLOWING STRUCTURAL COMPONENTS SHALL NOT EXCEED 40 LB. IN WEIGHT WITH THE BIT REMOVED.

Use the following note when Protective Sealer is to be applied to existing bridge decks.

101. TOP SURFACES OF EXISTING BRIDGE DECKS SHALL BE SEALED ACCORDING TO ITEM 559.1796 18 – PROTECTIVE SEALING OF STRUCTURAL CONCRETE FOR EXISTING BRIDGE DECKS.

The following note may be used when a significant number of anchor bolts need to be proof-load tested.

102. IT IS RECOMMENDED THAT THE CONTRACTOR INSTALL AND TEST SEVERAL BOLTS PRIOR TO GROUTING ALL THE BOLTS IN CASE OF A CONCRETE/GROUT INCOMPATIBILITY. THIS TEST IS FOR THE CONTRACTOR’S CONVENIENCE AND IS NOT PART OF THE ACCEPTANCE TESTING FOR THIS ITEM.


Use the following note when Protective Sealer is to be applied to existing concrete elements, other than bridge deck surfaces, containing uncoated bar reinforcement or having less than 3 inches of concrete cover (refer to 5.1.10 for additional guidelines). Complete the note so as to list the appropriate concrete elements for the particular bridge, and whether a penetrating type or coating type sealer is to be used on that element.

104. THE FOLLOWING CONCRETE ELEMENTS SHALL BE SEALED ACCORDING TO ITEM 559.1696 18 - PROTECTIVE SEALING OF STRUCTURAL CONCRETE:
Use the following note whenever a structural steel or prestressed concrete superstructure is to be replaced utilizing the existing substructures:

105. **IT SHALL BE THE CONTRACTOR’S RESPONSIBILITY TO CONFIRM THE FOLLOWING DIMENSIONS IN THE FIELD PRIOR TO THE FABRICATION OF NEW SUPERSTRUCTURE COMPONENTS:**

   A. **EXISTING SPAN LENGTHS (CHECK AT MULTIPLE APPROPRIATE POINTS IF SUBSTRUCTURES ARE NONPARALLEL).**

   B. **EXISTING LENGTHS OF INDIVIDUAL STRINGERS (IF ONLY CERTAIN STRINGERS ARE TO BE REPLACED).**

Use the following note whenever individual structural steel components are to be replaced:

106. **IT SHALL BE THE CONTRACTOR’S RESPONSIBILITY TO CONFIRM THE LENGTHS OF EXISTING STRUCTURAL STEEL COMPONENTS TO BE REPLACED PRIOR TO THE FABRICATION OF THE REPLACEMENT COMPONENTS.**

Use the following note whenever bearings are to be replaced on new or reconstructed pedestals:

107. **IT SHALL BE THE CONTRACTOR’S RESPONSIBILITY TO CONFIRM THE TOP OF PEDESTAL ELEVATIONS PRIOR TO CASTING THE NEW PEDESTALS AND INSTALLING THE NEW BEARINGS.**

Use the following note whenever bearings are to be replaced on the existing pedestals:

108. **THE COST OF FURNISHING AND INSTALLING SHIM PLATES FOR ADJUSTING BEARING HEIGHTS SHALL BE INCLUDED IN ITEM 564.51.nnnn, STRUCTURAL STEEL. THIS IS A CONTINGENCY ITEM THAT WILL ONLY BE PAID IF ACTUAL FIELD CONDITIONS CONFLICT WITH THE CONTRACT DOCUMENTS. THE SHIM PLATE MATERIAL SHALL BE CONSIDERED A MINOR ITEM AND THE UNIT PRICE BID IS NOT SUBJECT TO RENEGOTIATION IF THE QUANTITY VARIES FROM THE ESTIMATED QUANTITY.**

   **OFF SITE FABRICATION INSPECTION WILL USUALLY BE WAIVED FOR THE SHIM MATERIAL. MATERIAL CERTIFICATION AND DOCUMENTATION SHALL BE SUBMITTED TO THE EIC FOR ACCEPTANCE OF THE MATERIAL.**

See Section 8.2.7 to determine which items and when to use the following two notes:

109. **SHOP DRAWINGS SHALL BE SUBMITTED TO THE D.C.E.S. FOR APPROVAL FOR THE FOLLOWING STRUCTURAL STEEL REPLACEMENT ITEMS:** (List the items.)

110. **SHOP DRAWINGS SHALL BE SUBMITTED TO THE ENGINEER FOR APPROVAL FOR THE FOLLOWING STRUCTURAL STEEL REPLACEMENT ITEMS:** (List the items.)
Use the following note for rehabilitation contracts:

111. **IF THE STRUCTURE HAS A BRIDGE IDENTIFICATION NUMBER (B.I.N.) PLATE ATTACHED, IT SHALL BE THE CONTRACTOR’S RESPONSIBILITY TO PROTECT IT DURING CONSTRUCTION OR REMOVE AND REMOUNT IT AFTER CONSTRUCTION IS COMPLETED.**

The following special note shall be included in the PS&E for each structure unless the field inspection indicates that less than 30% of the steel will require Near White Metal Blast Cleaning: (The designer should arrange for this field inspection not more than 1 year before the PS&E date. SSPC specifications should be used to determine the percent of steel surface area requiring Near White Metal Blast Cleaning to the nearest 20 percent, i.e., 0-20-40-60-80-100.)

112. **CLEANING STRUCTURAL STEEL ON EXISTING BRIDGES:**

   **BIN # __________**

   IT IS ANTICIPATED THAT A SIGNIFICANT PORTION OF THE STRUCTURAL STEEL IN THE BRIDGE(S) IDENTIFIED ABOVE WILL REQUIRE NEAR WHITE METAL BLAST CLEANING IN ACCORDANCE WITH SECTION 573 OF THE STANDARD SPECIFICATIONS. THEREFORE, BIDDERS SHOULD INSPECT THE BRIDGE(S) CAREFULLY PRIOR TO SUBMITTING BIDS.

113. **THE CONTRACTOR SHALL KEEP ALL BRIDGE DRAINS CLEAN AND FREE FLOWING DURING THE LIFE OF THE CONTRACT. THE COST SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE VARIOUS SUPERSTRUCTURE ITEMS IN THE CONTRACT.**

Use the following note, if applicable, whenever Structural Lifting is part of the contract:

114. **VEHICULAR TRAFFIC OR CONSTRUCTION EQUIPMENT SHALL NOT BE PERMITTED ON THE LIFTED SPAN UNTIL SHIMS, CRIBBING, BOLSTERS OR OTHER SUITABLE SUPPORTS ARE IN THEIR REQUIRED POSITION.**

Use the following note if Conduits are encased, or are suspected to be encased, in the superstructure of a bridge undergoing rehabilitation:

115. **CONDUIT CAUTION NOTE:**

   THE CONTRACTOR’S ATTENTION IS DIRECTED TO THE FACT THAT CONDUITS MAY BE PRESENT IN THE STRUCTURAL SLABS, PARAPETS OR SIDEWALKS OF BRIDGES RECEIVING NEW OR REWORKED JOINT SYSTEMS. THEIR EXISTENCE AND LOCATIONS SHALL BE FIELD VERIFIED. IF CONDUITS ARE PRESENT AND ARE ENCOUNTERED DURING CONSTRUCTION OPERATIONS, CARE SHALL BE EXERCISED NOT TO DAMAGE CONDUITS, EXPANSION COUPLINGS, OR CONTENTS OF CONDUITS. ANY DAMAGE SHALL BE REPAIRED TO THE SATISFACTION OF THE ENGINEER, AT NO COST TO THE STATE.

116. **STRUCTURAL SLAB CONCRETE OVERLAY NOTES.**

117. **THE MINIMUM THICKNESS OF THE MICRO SILICA CONCRETE OVERLAY SHALL BE 1½ INCHES.**
118. THE MINIMUM THICKNESS OF THE DP CONCRETE OVERLAY SHALL BE 1½ INCHES.

119. THE MINIMUM TOTAL COVER (EXISTING CONCRETE OR SLAB RECONSTRUCTION CONCRETE PLUS THICKNESS OF DP OR MICRO-SILICA OVERLAY) SHALL BE 2 1/4 INCHES.

120. THE TRANSITION LENGTHS BETWEEN THE EXISTING PROFILE AND REVISED FINISHED PROFILE SHALL BE THE SAME AS THOSE SHOWN ON THE PLANS.

121. SHOULD THE TYPE OF RECONSTRUCTION WORK REQUIRED TO BE PERFORMED ON THE STRUCTURAL SLAB, TOGETHER WITH APPLICATION OF THE CONTRACTOR’S CHOICE OF SPECIALIZED CONCRETE OVERLAY, RESULT IN A REVISED FINISHED PROFILE HIGHER THAN THAT SHOWN ON THE PLANS, THE CONTRACTOR SHALL SUBMIT THE REVISED PROFILE TO THE REGIONAL DIRECTOR FOR APPROVAL AT LEAST TWO WEEKS PRIOR TO PLACEMENT OF THE CONCRETE OVERLAY.

122. THE CONTRACTOR’S PROPOSAL MAY INCLUDE ADDITIONAL GRADE TRANSITIONS SUBJECT TO THE FOLLOWING:

   A. THE MINIMUM LENGTH BETWEEN GRADE TRANSITIONS SHALL BE ______* FEET.


   C. THE SLOPE CHANGES DO NOT CREATE DRAINAGE PROBLEMS ON THE BRIDGE DECK.

   NO OVERLAY MATERIAL SHALL BE PLACED UNTIL THE REGIONAL DIRECTOR HAS APPROVED THE CONTRACTOR’S PROPOSED REVISIONS.

* The designer should select values for the length between and the difference in slope of the grade transitions considering design speed, rider comfort, and bridge geometry. Suggested values are 60 ft. and 0.5 percent.

123. ALL ROADWAY SURFACES RECEIVING A SPECIALIZED CONCRETE OVERLAY SHALL BE GROOVED UNDER THE SAWCUT GROOVING OF STRUCTURAL SLAB SURFACE ITEM AND SEALED UNDER THE PROTECTIVE SEALING OF STRUCTURAL CONCRETE ON NEW BRIDGE DECKS AND BRIDGE DECK OVERLAYS ITEM.

124. MISCELLANEOUS NOTES

125. LUMBER AND TIMBER NOTES

126. STRESS GRADED LUMBER AND TIMBER HAVE BEEN DESIGNED FOR THE FOLLOWING ALLOWABLE STRESSES, AND THE TYPE USED MUST MEET THESE MINIMUM REQUIREMENTS:
127. **EXTREME FIBER IN BENDING AND TENSION PARALLEL TO GRAIN**
**COMPRESSION PERPENDICULAR TO GRAIN**
**MODULUS OF ELASTICITY**

128. **STREAM PROTECTION NOTE**

Use the following note only if requested by Dept. of Environmental Conservation or the Regional Office.

129. **DURING THE COURSE OF CONSTRUCTION, THE CONTRACTOR SHALL CONDUCT OPERATIONS IN SUCH A MANNER AS TO PREVENT OR REDUCE TO A MINIMUM ANY DAMAGE TO ANY STREAM FROM POLLUTION BY DEBRIS, SEDIMENT, OR OTHER FOREIGN MATERIAL, OR FROM MANIPULATION OF EQUIPMENT AND/OR MATERIALS IN OR NEAR SUCH STREAMS. THE CONTRACTOR SHALL NOT RETURN DIRECTLY TO A STREAM ANY WATER WHICH HAS BEEN USED FOR WASH PURPOSES OR OTHER SIMILAR OPERATIONS WHICH CAUSE THIS WATER TO BECOME POLLUTED WITH SAND, SILT, CEMENT, OIL, OR OTHER IMPURITIES. IF THE CONTRACTOR USES WATER FROM A STREAM, THE CONTRACTOR SHALL CONSTRUCT AN INTAKE OR TEMPORARY DAM REQUIRED TO PROTECT AND MAINTAIN WATER RIGHTS AND TO SUSTAIN FISH LIFE DOWNSTREAM.**

130. **CONCRETE ANCHOR STUD NOTE**

Use the following note when pier nosing is used.

131. **ALL CONCRETE ANCHOR STUDS WHICH ARE ATTACHED TO THE PIER NOSING SHALL MEET THE REQUIREMENTS LISTED IN MATERIAL SUBSECTION 709-05, STUD SHEAR CONNECTORS. PAYMENT FOR FURNISHING AND PLACING THE CONCRETE ANCHORS AND ANGLE WILL BE INCLUDED IN THE UNIT PRICE BID FOR THE CONCRETE ITEM TO WHICH THE ANCHORS ARE ATTACHED.**

Use one of the following notes when concrete box culverts are used.


or

134. **STONE MASONRY**

135. JOINTS FOR STONE MASONRY MAY VARY FROM ½ INCH TO 1 INCH THICKNESS.

136. FINISH OF STONE MASONRY SHALL BE AS FOLLOWS:

137. **DIMENSION MASONRY**

138. RINGSTONE, QUOINS, COPINGS AND OTHER STONES, IF SO DESIGNATED, SHALL BE DIMENSION MASONRY.

139. ALL JOINTS FOR DIMENSION MASONRY SHALL BE ½ INCH THICKNESS.

140. FINISH OF DIMENSION MASONRY SHALL BE AS FOLLOWS:

141. **PRECAST PRECOMPRESSED CONCRETE/STEEL COMPOSITE SUPERSTRUCTURE NOTES**

When a Precast Precompressed Concrete/Steel Composite Superstructure (Inverset™) bridge is specified, the designer shall place the following applicable notes in the contract plans.

142. CONCRETE IN THE DECK SLAB SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF _________ PSI AT 28 DAYS. THE UNITS SHALL NOT BE HANDLED UNTIL CONCRETE STRENGTH REACHES A MINIMUM OF 3000 PSI.

143. ASTM A709 GRADE 50W STEEL SHALL BE USED AS STRUCTURAL STEEL.

or

144. ASTM A709 GRADE 50 STEEL SHALL BE USED AS STRUCTURAL STEEL.

145. HIGH STRENGTH BOLTS USED IN DIAPHRAGM CONNECTIONS SHALL BE ASTM A325. (USE TYPE 1 FOR PAINTED STEEL, TYPE 3 FOR WEATHERING STEEL).

146. TO ENSURE FULL AND EVEN BEARING BETWEEN BOTTOM OF BEAMS AND MASONRY PLATES, THE BOTTOM SURFACES OF BEAMS IN THE BEARING AREAS SHALL, WITHIN EACH PANEL, BE FABRICATED TO BE TRULY IN ONE PLANE.

147. ALL REINFORCEMENT SHALL HAVE A COVER OF 2 INCHES (TO BOTTOM OF LONGITUDINAL GROOVES) UNLESS SHOWN OTHERWISE. THE TOP BARS IN THE DECK AND APPROACH SLAB SHALL BE EPOXY COATED. NO CHAIRS, BOLSTERS OR OTHER SUPPORT DEVICES SHALL BE IN PLACE AGAINST THE BOTTOM SURFACE OF THE FORM (TOP OF DECK IN FIELD) DURING CASTING.

148. ANCHOR BOLTS MAY BE CAST INTO THE BRIDGE SEATS, OR AT THE CONTRACTOR’S OPTION, DRILLED AND GROUTED INTO THE ABUTMENTS AT NO ADDITIONAL COST TO THE STATE.
149. THE PRECAST PRECOMPRESSED CONCRETE/STEEL COMPOSITE SUPERSTRUCTURE UNITS MAY BE CONSTRUCTED WITHOUT DIAPHRAGMS. HOWEVER, PRIOR TO TRANSPORTATION TO THE BRIDGE SITE, ALL DIAPHRAGMS INTEGRAL TO ANY ONE UNIT SHALL BE INSTALLED.

150. GRIND ALL EDGES OF STEEL AS NEEDED TO REMOVE SHARP EDGES PRIOR TO CLEANING FOR PAINTING.

151. STRUCTURAL STEEL SHALL BE CLEANED AND PAINTED UNDER THE SHOP APPLIED STRUCTURAL STEEL PAINT SYSTEM ITEM. AFTER CLEANING, MILL SCALE SHALL NOT BE PRESENT. AT THE TIME OF SHIPMENT OF THE UNITS TO THE JOB SITE, THE 3 COATS OF PAINT SHALL HAVE BEEN APPLIED. THE COLOR OF THE FINISH COATING SHALL BE__________. THE COLOR SHALL CONFORM TO ________. VIEWING SHALL BE DONE UNDER NORTH STANDARD DAYLIGHT.

(Designer shall designate color and either Federal Color Standard Number 595 Number or Munsell Book Notation Number to which color conforms).


or

153. CLEANING CONTROLLED OXIDIZING STRUCTURAL STEEL ASTM A709 GRADE 50W.

A. IN THE FABRICATION SHOP

GIRDERS SHALL BE BLAST CLEANED IN ACCORDANCE WITH SSPC-SP6 (COMMERCIAL BLAST CLEANING). HEAVY COATINGS OF OIL OR GREASE SHALL BE REMOVED BEFORE BLASTING IN ACCORDANCE WITH SSPC-SP1 (SOLVENT CLEANING).

B. IN THE FIELD

THE OUTSIDE SURFACE OF THE FASCIA STRINGERS SHALL BE CLEANED SO THAT ALL DIRT, GREASE, PAINT OR OTHER FOREIGN MATERIAL IS REMOVED AT THE COMPLETION OF THE BRIDGE CONSTRUCTION. THE PURPOSE OF THE CLEANING IS TO RETURN THE FASCIA SURFACES TO THE CONDITION IN WHICH THEY LEFT THE FABRICATION SHOP.

THE COST OF CLEANING THIS STEEL IN THE FABRICATION SHOP AND THE FIELD SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE VARIOUS ITEMS IN THE CONTRACT.

154. BEARING ANCHOR BOLT NUTS SHALL BE SNUG TIGHT AS PER THE NYS STEEL CONSTRUCTION MANUAL.

155. THIS IS A NON-MATCH CAST SEGMENTAL CONSTRUCTION. HENCE, ALL PROVISIONS OF ‘SECTION 2.3 INSTALLATION DRAWINGS AND SUPPORTING DOCUMENTS’ OF THE PCCM, EXCEPT PROVISIONS RELATED TO POST-TENSIONING, SHALL APPLY.
156. **PROVISIONS OF SECTION 8.4.5. SHEAR KEY JOINTS OF THE PCCM SHALL NOT APPLY.** THE CONTRACTOR SHALL PROPOSE A LEAK PROOF LONGITUDINAL JOINT SYSTEM BETWEEN THE UNITS. ALL NECESSARY INFORMATION SUCH AS PREPARATION OF SHEAR KEY SURFACE, MATERIAL FOR SHEAR KEY GROUT, PLACEMENT AND CURING OF SHEAR KEYS AND PLACEMENT OF LEAK PROOFING SYSTEM SHALL BE SHOWN ON THE INSTALLATION DRAWINGS.

157. **PROCEDURE FOR PREPARING BLOCKOUT SURFACES, PLACING AND CURING BACKFILL, ETC. SHALL BE SHOWN ON INSTALLATION DRAWINGS.**

158. **THE COST OF FURNISHING AND INSTALLING SHIM PLATES UNDER THE BEARINGS SHALL BE INCLUDED IN THE UNIT PRICE BID FOR THE BEARINGS.**

Note: Designers shall allow ¾ inch thickness for shim plates when setting pedestal elevations for reinforced concrete three-sided structures.

**SUPERSTRUCTURE SLAB SHEET**

The following notes shall be placed in the contract plans on the superstructure slab sheet. The slab placement sequence diagram from Section 5.1.8 shall also be included.

159. **DECK PLACEMENT NOTES**

The following notes shall be shown on the plans for all simple and continuous span structures:

160. **CONCRETE PLACEMENT AND FINISHING OPERATIONS SHALL BE PERFORMED AS RAPIDLY AS POSSIBLE. THE ENGINEER MAY ORDER THE CONTRACTOR TO STOP PLACEMENT OPERATIONS AT ANY TIME IF, IN THE ENGINEER’S OPINION, CONCRETE PLACED DURING THE PLACEMENT HAS STARTED TO SET, OR IS ABOUT TO SET, AND FURTHER PLACEMENT OF CONCRETE WILL CAUSE DEFLECTION CRACKING.**

161. **LONGITUDINAL CONSTRUCTION JOINTS WILL NOT BE PERMITTED.**

162. **FINISHING MACHINE(S) SHALL BE OPERATED AS CLOSE TO THE SKEW ANGLE AS PRACTICABLE FOR SKEW ANGLES BETWEEN 0° AND 50°. WHEN THE SKEW ANGLE IS GREATER THAN 50° THE FINISHING MACHINE(S) SHALL BE OPERATED AT AN ANGLE OF 50°.**

163. **WET BURLAP CURING BLANKETS ARE REQUIRED TO BE PLACED ON THE CONCRETE DECK WITHIN 30 MINUTES OF THE CONCRETE BEING DEPOSITED INTO THE FORMS OR 5 MINUTES AFTER FINISHING, WHICHEREVER COMES FIRST. THE PLACEMENT OF THE TURF DRAG TEXTURE SHALL NOT INTERFERE WITH THESE REQUIREMENTS.**
164. IN THE EVENT THE CONTRACTOR'S DECK PLACEMENT OPERATION IS STOPPED PRIOR TO COMPLETION, WHETHER BY THE CONTRACTOR'S OWN DECISION OR BY ORDER OF THE ENGINEER, THE CONTRACTOR SHALL BE RESPONSIBLE FOR PROVIDING A FINISHED DECK GRADE WHICH MATCHES THE PLANNED PROFILE. ANY SUBSEQUENT REVISIONS TO DECK FORMS MADE NECESSARY BY SUCH ACTION SHALL BE AT THE CONTRACTOR'S EXPENSE.

Include the following note when the structure has a cross slope transition:

165. SINCE THIS STRUCTURE HAS A CROSS SLOPE TRANSITION, IT MAY BE ADVISABLE TO PLACE THE FINISHING MACHINE PERPENDICULAR TO THE STATION LINE.

Include the following note when two finishing machines are required:

166. (Insert “PLACEMENT 1” or “THE CONTINUOUS”) PLACEMENT SHALL BE ACCOMPLISHED BY THE SIMULTANEOUS OPERATION OF TWO FINISHING MACHINES AND CREWS. A MINIMUM RATE OF 30 CUBIC YARDS PER HOUR SHALL BE MAINTAINED BY EACH MACHINE.

The following notes shall be shown on the plans for all continuous spans (whether the deck is placed in one or multiple placements) as appropriate.

When there will be no exceptions to the pouring sequence allowed, use the following Note.

167. THERE WILL BE NO EXCEPTIONS MADE TO THE POURING SEQUENCE AS SHOWN ON THE CONTRACT PLANS.

When exceptions to the pouring sequence are possible pending review by the Department (refer to Department’s “Procedure for Approval of Alternate Deck Pouring Sequence on Continuous Bridges”) use the following three notes:

168. THE CONCRETE DECK SLAB FOR THIS STRUCTURE SHALL BE PLACED ACCORDING TO THE POURING SEQUENCE SHOWN ON THE CONTRACT PLANS. REQUESTS FOR ANY ALTERNATE DECK POURING SEQUENCE SHALL BE SUBMITTED TO THE EIC. THE SUBMITTAL REQUIREMENTS ARE PROVIDED IN THE DEPARTMENT’S “PROCEDURE FOR APPROVAL OF ALTERNATE DECK POURING SEQUENCE ON CONTINUOUS BRIDGES” IN THE CONSTRUCTION INSPECTION MANUAL. NO RELATED WORK MAY BE PROGRESSED BY THE CONTRACTOR UNTIL THE WRITTEN APPROVAL OF THE ALTERNATE PROCEDURE IS RECEIVED FROM THE DEPARTMENT (REGIONAL OFFICE). THE DEPARTMENT WILL REVIEW THE REQUEST AND REPLY WITHIN (15) WORK DAYS AFTER RECEIPT OF ALL THE REQUIRED SUBMITTAL DOCUMENTS FROM THE CONTRACTOR.

169. THE CONTRACTOR SHALL PROVIDE TO THE ENGINEER THE PROPOSED SET RETARDING WATER ADMIXTURE (ASTM TYPE D, SRWR) AND A COPY OF THE MANUFACTURER’S LITERATURE SPECIFYING THE RECOMMENDED RANGE TO PROVIDE SUFFICIENT RETARDATION. THIS SRWR DOSAGE SHALL NOT BE REDUCED AS THE PLACEMENT PROGRESSES. THE ENGINEER WILL REJECT ANY CONCRETE TRUCK THAT CALLS FOR AN ADMIXTURE DOSAGE RATE BEYOND THE
170. THE VALUES SHOWN IN THE CAMBER AND HAUNCH TABLES ARE BASED ON THE DECK PLACEMENT SEQUENCE SHOWN ON THE PLANS. IF THE DECK PLACEMENT SEQUENCE IS ALTERED, THE CAMBER AND HAUNCH TABLES NEED TO BE RECOMPUTED. THE CONTRACTOR IS RESPONSIBLE TO HAVE A PROFESSIONAL ENGINEER RECOMPUTE THESE TABLES AND SUBMIT THEM TO THE D.C.E.S FOR APPROVAL.

The following notes shall be shown on the plans for continuous spans when a two placement sequence is used:

171. CONSTRUCTION JOINTS SHALL BE PLACED PARALLEL TO THE SKEW ANGLE. DECK CONCRETE SHALL BE PLACED SO THAT THE LEADING EDGE PARALLELS THE SKEW. FINISHING MACHINE(S) SHALL BE OPERATED AS CLOSE TO THE SKEW ANGLE AS PRACTICABLE. TEXTURING MAY BE DONE LONGITUDINAL, TRANSVERSE OR PARALLEL TO THE ALIGNMENT OF THE FINISHING MACHINE.

172. ALL AREAS SHOWN ON THE PLANS AS “PLACEMENT 1” MUST BE PLACED DURING THE INITIAL CONTINUOUS WORK PERIOD. SUBSEQUENT PLACEMENTS (CONTINUOUS PLACEMENTS) WILL NOT BE PERMITTED UNTIL 72 HOURS OF ACCEPTABLE CURING AFTER THE COMPLETION OF THE PREVIOUS PLACEMENT.

Include the following note when the structure contains three or more spans.

173. THE CONTRACTOR MAY DIVIDE PLACEMENT 2 INTO SEPARATE SEGMENTS PROVIDED THE 72 HOUR WAITING PERIOD BETWEEN PLACEMENTS IS OBSERVED.

Add the following note to contract plans for projects involving prestressed concrete beams on integral abutment bridges and for continuous for live load designs:

174. ALL PRESTRESSED CONCRETE BRIDGE BEAMS SHALL HAVE A MINIMUM AGE OF 60 DAYS AT THE TIME OF CONCRETE DECK PLACEMENT.

Stage Construction Notes

The following notes shall be used, where applicable, for stage construction projects on bridges with steel superstructures.

176. THE STRUCTURAL SLAB AND SLAB OVERHANG FOR EACH STAGE OF CONSTRUCTION HAVE BEEN DESIGNED FOR THE LOADING CONDITIONS SHOWN IN THE DETAILS.

177. THE COST OF FURNISHING AND PLACING MECHANICAL CONNECTORS, MATERIAL SPECIFICATION, SECTION 709-10, SHALL BE INCLUDED IN THE UNIT PRICES BID FOR THE SUPERSTRUCTURE SLAB ITEM.
In some instances, geometry may require the use of a large overhang during stage construction. Special temporary bracing may be required in order to prevent the rotation of the temporary fascia girder during the deck placement.

178. **THE CONTRACTOR'S ATTENTION IS DIRECTED TO THE UNUSUALLY LARGE TEMPORARY OVERHANG PRESENT DURING THE STAGE ___ STRUCTURAL SLAB PLACEMENT. THE CONTRACTOR SHALL PROVIDE ADEQUATE TEMPORARY SUPPORT AND BRACING TO PREVENT THE TEMPORARY FASCIA STRINGER FROM TWISTING UNDER THE LOADS OF THE CONCRETE DEAD LOAD AND THE CONSTRUCTION LOADS. THE CONTRACTOR MUST SUBMIT OVERHANG FORMING DESIGN AND DETAILS TO THE D.C.E.S. FOR APPROVAL.**

179. **DUE TO THE NATURE OF STAGE CONSTRUCTION AND THE PROBLEMS INHERENT WITH DIFFERENTIAL DEFLECTIONS, THE HOLES FOR ONE SIDE OF THE STAGE DIAPHRAGM CONNECTION PLATES SHALL BE FIELD DRILLED. NO ADDITIONAL COMPENSATION SHALL BE MADE FOR FIELD DRILLING.**

In cases where more than two stages are used, the following notes will have to be modified:

180. **THE INTERMEDIATE DIAPHRAGMS, END DIAPHRAGMS AND ANY OTHER CROSS FRAMES THAT MAY BE PRESENT BETWEEN STAGE 1 AND STAGE 2 STRINGERS SHALL NOT BE INSTALLED UNTIL 72 HOURS FOLLOWING THE PLACEMENT OF THE STAGE 2 STRUCTURAL SLAB.**

If a closure placement is called for, the following sentence must be added to this note:

181. **THE INTERMEDIATE DIAPHRAGMS AND END DIAPHRAGMS MUST BE IN PLACE AND BOLTS TIGHTENED PRIOR TO PROCEEDING WITH THE CLOSURE PLACEMENT.**

182. **THE CONTRACTOR SHALL WAIT A MINIMUM OF 72 HOURS FOLLOWING COMPLETION OF THE SECOND STAGE DECK PLACEMENT BEFORE BEGINNING THE CLOSURE PLACEMENT.**

183. **FORM WORK FOR THE STAGE 2 DECK PLACEMENT SHALL BE SUPPORTED ONLY BY THE STAGE 2 STRINGERS, NOT BY THE STAGE 1 STRINGER IMMEDIATELY ADJACENT.**

184. **PRIOR TO PLACING THE STAGE 2 DECK PLACEMENT AND FOR 72 HOURS FOLLOWING ITS COMPLETION, NO REINFORCING BAR WITHIN THE CLOSURE PLACEMENT SHALL BE WIRED.**

185. **THE TEMPORARY FASCIAS OF THE STAGE 1 AND STAGE 2 DECK SHALL BE THOROUGHLY WET FOR 12 HOURS IMMEDIATELY PRIOR TO PROCEEDING WITH THE CLOSURE PLACEMENT. THE CONTRACTOR SHALL REMOVE ALL STANDING WATER WITH OIL-FREE COMPRESSED AIR AND SHALL PROTECT THE FASCIA SURFACES FROM DRYING, SO THE EXISTING CONCRETE REMAINS IN A CLEAN, SATURATED SURFACE DRY CONDITION UNTIL PLACEMENT OF THE NEW CONCRETE.**
The following note shall be used for stage construction projects using adjacent precast prestressed beams when anticipated camber as per Section 9.14 is greater than 1 inch:

186. **STAGE 1 OF THE DECK HAS BEEN DETAILED WITH A 7 INCH MINIMUM DECK THICKNESS TO DEAL WITH A SMALL AMOUNT OF CAMBER GROWTH FOR STAGE 2 UNITS. IF THE CONTRACTOR’S SCHEDULE PLANS SIGNIFICANTLY MORE (14 DAYS) STORAGE TIME FOR STAGE 2 UNITS THAN STAGE 1 UNITS, CAMBER GROWTH CONTROL MEASURES SHALL BE PROPOSED BY THE CONTRACTOR IN THE SHOP DRAWINGS. SUGGESTED CAMBER GROWTH CONTROL MEASURES ARE:**

1. **BOTH STAGE 1 AND STAGE 2 UNITS SHALL BE STORED FOR A MINIMUM OF 60 DAYS (PRIOR TO SHIPMENT) TO ALLOW MOST OF THE CAMBER GROWTH TO OCCUR PRIOR TO SHIPMENT.**

2. **PRELOAD THE STAGE 2 BEAMS (IN STORAGE) TO RESTRAIN GROWTH. THE CONTRACTOR SHALL SUBMIT DESIGN CALCULATIONS ALONG WITH THE SHOP DRAWINGS.**


187. **PRESTRESSED CONCRETE BEAM NOTES**

188. **THE CONTRACTOR MAY PROPOSE DEBONDING OF PRETENSIONING STRANDS FOR 6 INCHES FROM ENDS OF BEAMS TO REDUCE THE TENDENCY FOR BEAM ENDS TO CRACK. TOTAL NUMBER OF DEBONDED STANDS (DESIGN BONDING SHOWN ON THE CONTRACT PLANS AND CRACK CONTROL DEBONDING COMBINED) SHALL NOT EXCEED 50% OF TOTAL NUMBER OF STRANDS.**

Add the following note to contract plans for projects that use PCEF Bulb Tee prestressed concrete girders:

189. **THE CONTRACTOR MAY PROPOSE TO SUBSTITUTE NEW ENGLAND BULB TEE GIRDERS OF EQUIVALENT SECTION PROPERTIES FOR THE PCEF GIRDERS SHOWN ON THE CONTRACT PLANS. ALL ADDITIONAL COSTS ASSOCIATED WITH THE SUBSTITUTION INCLUDING DESIGN, CONSTRUCTION, AND DETAILING CHANGES SHALL BE AT THE CONTRACTORS EXPENSE.**

Add the following note to contract plans for projects that use New England Bulb Tee prestressed concrete girders:

190. **THE CONTRACTOR MAY PROPOSE TO SUBSTITUTE PCEF BULB TEE GIRDERS OF EQUIVALENT SECTION PROPERTIES FOR THE NEBT GIRDERS SHOWN ON THE CONTRACT PLANS. ALL ADDITIONAL COSTS ASSOCIATED WITH THE SUBSTITUTION INCLUDING DESIGN, CONSTRUCTION, AND DETAILING CHANGES SHALL BE AT THE CONTRACTORS EXPENSE.**
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When designing connections, interference with other members should be considered. This is also true when making spans continuous for live load or full dead load. Welding of stiffeners is not allowed to the splice plates. Lateral gusset plates may have to be moved.

19.4.3 Painted vs. Unpainted

Special consideration should be given to blast cleaning requirements and the specifications governing the painting or coating of the new structure. Often, because of the nature of the work, the existing and new structures will be painted using different items. The limits of each controlling item should be clearly shown on the plans.

19.4.4 Fracture-Critical Member (FCM) Work

When dealing with FCMs, such as large floorbeams in a truss or column connections, the process of structure reassembly must be considered. The structure must be erected such that there are no unaccounted for internal stresses induced by the assembly sequence. To ensure this “zero stress” state, the system may require erection shoring, or the system may be assembled in the shop and transported to the site.

Steel available at the time of original construction will most likely not have the strength, toughness and fatigue life of the steels used today. Special inspection may be needed for the determination to reuse the existing steel. The extent of deterioration should be carefully considered for the lead time of the contract plans. If the project is not anticipated to begin for two years and to be completed for four years, the additional amount of corrosion should be anticipated and compensated for in the design. Steels used in main members should be ordered to the correct level of strength and toughness. For main members, the material should specify Charpy V Notch (CVN) requirements for FCM Zone 2 and reference the direction of rolling (See SCM Section 507).

19.4.5 Rehabilitation of Riveted Structures

The following considerations should be addressed during design of riveted structures that will be rehabilitated:

- It is important to consider the original construction, and the need to bring the structure up to the current AASHTO code requirements for strength, service life, and fatigue resistance. Riveted connections in structures are classified as Category D for fatigue resistance per the AASHTO Specifications. In order to upgrade this classification, removal of all rivets, reaming all holes, and installation of oversized bolts is necessary. In lieu of retrofitting, remaining fatigue life may be calculated using the Section 7 of the AASHTO Manual for Bridge Evaluation. If the calculated remaining safe life exceeds the remaining expected service life of the structure, further work is not required if the component is in good condition.
• An in-depth inspection of the steel and riveted connections should be performed during preliminary engineering. The extent of deterioration that is documented in the in-depth inspection shall be clearly identified on the contract plans. Pack rust should be noted in the in-depth inspection, as it is a critical issue for riveted structures. Even a very small amount of pack rust can increase substantially in period of only a few years and will have a major impact on the serviceability of the bridge. It is essential that any pack rust, no matter how slight, be identified.

• Use of bolted repairs is preferred on most riveted bridge rehabilitation projects, because of the difficulty and cost associated with welding older steels.

• Types of Repairs:

  **Removal of Rivets and Replacement with High Strength Bolts:**

  The determination as to when to replace individual rivets in built-up structural elements is based mainly on the section loss of the rivet head. An estimate based on a field survey should be used to determine the quantity of rivets to be replaced. This shall be detailed on the contract plans. It is recommended that this estimate be increased to take into account unknown or unforeseen field conditions, as it is not uncommon for this percentage to go to 100% where rivets are concealed by the deck.

  Designers should contact the Metals Engineering Unit for rivet removal and replacement notes and details to be placed on the contract plans.

  Total quantities shall be confirmed by the E.I.C., and paid for under Item 586.05.

  **Coverplated Repairs of Riveted Members:**

  Cover-plating should be considered for the repair of localized areas of deterioration, such as the ends of stringers under joint systems, when the deck and bearings are being replaced, when the remainder of the structural element is in good condition, and/or when the replacement of the entire element does not fall under the scope of the project. Since existing rivet holes are rarely available for these new connections, it is recommended that new cover-plates with full size shop drilled holes be provided for field use. After assembly and alignment, the holes in the new steel shall be used as a template to field drill new holes in the existing steel.

  It is important that designers clearly define all holes to be field drilled on the contract plans, as the Contractor will be paid for each designated hole location. Additionally, the following note should be placed on the associated contract drawings:

  “The Contractor shall be provided one payment for each hole location designated to be field drilled, regardless of the number of plies field drilled.”
Bridge Rehabilitation Projects

Field drilling existing steel to be included under Item 586.10; and installation of new steel coverplates and bolts to be paid for under Item 564.51n

Replacement of Members, Member Components, and Member Connections:

Based on the extent of deterioration, consideration should be given to replacing the existing member components and connections with new steel sections and high strength bolts.

Generally, new steel that will mate to existing steel shall be brought out to the field blank (without predrilled holes). Thereafter, rivet holes in the existing steel shall be used as a one time template to core drill full size holes in the new steel. This method is preferred by both Contractors and NYSDOT as it simplifies fabrication, expedites construction, and provides the hole quality required by the SCM.

Another less preferred and more time consuming approach is to use templates to fabricate the replacement element. This method requires the contractor to disassemble the connection, create a template of the existing rivet pattern, and then fabricate a replacement element. This method can only be used when the bridge can be closed to traffic and the use of temporary supports is possible.

Removal of existing steel and associated connections shall be paid for under Items 589.01nnnn or 589.52nnnn; drilling new steel to match existing holes shall be included under Item 564.xx; and installation of new steel and new bolts shall be paid under Item 564.xx.

A legend specifying payment items and sections showing the locations where existing holes are to be used for the repair shall be clearly defined on the contract plans.

Additional notes to designer:

1. Where deterioration is found in riveted primary members (multi-girder), designers should review the project scope. If the deck is being replaced as part of the same project, replacement of existing riveted members with new steel may be faster and more cost-effective.
2. For non-redundant two-girder structures with floor beams, replacement may not be an option.
3. As repairing riveted members tends to be very costly, an analysis shall be performed to determine the cost benefit of repairing rivet holes versus replacing the elements. Rivet replacement and field drilling both cost in the range of $50 per hole, and a fairly simple repair on a plate girder can have 100 holes.
4. Note that the cost of temporarily supporting a member during rivet removal may be higher than the cost of repairs.
5. Projects that involve riveted structures should have the required work determined after the In-depth Inspection is done. Biennial inspection data is usually not precise enough to accurately estimate the cost or scope of work on the rehabilitation of a riveted structure.
19.4.6 A7 Steel Retrofits or Replacement

It is recommended to replace existing A7 steel with ASTM A709 Grade 36 or Grade 50 whenever possible. FCM Zone 2 steel should be used for FCM members.

If A7 steel is to be retained and welding is to be considered, a chemical analysis should first be performed in order to determine the Carbon Equivalent (CE). This is because the ASTM A7 specification did not contain limits for carbon or other commonly used alloying elements and a higher CE negatively affects weldability. The CE shall be calculated using the following formula:

\[ CE = C + \frac{(Mn+Si)}{6} + \frac{(Cr+Mo+V)}{5} + \frac{(Ni+Cu)}{15}, \]

where

- \( C \) = Carbon percentage
- \( Mn \) = Manganese percentage
- \( Si \) = Silicon percentage
- \( Cr \) = Chromium percentage
- \( Mo \) = Molybdenum percentage
- \( V \) = Vanadium percentage
- \( Ni \) = Nickel percentage
- \( Cu \) = Copper percentage

(Source: AWS D1.5 Bridge Welding Code, 2010)

The CE value is used along with plate thickness to specify preheat and interpass temperatures for all welding onto A7 steel, including studs. Table 19-4 shall be used to specify the temperature values. For dissimilar plate thicknesses, the thicker value shall be used. If no chemical test is performed, a CE of 0.55 is to be assumed.

<table>
<thead>
<tr>
<th>CE, %</th>
<th>Plate Thickness, inches</th>
<th>Preheat and interpass Actual Temperature F°</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40 max</td>
<td>Any</td>
<td>50</td>
</tr>
<tr>
<td>0.41 to 0.45 inclusive</td>
<td>To 1 ¼ inclusive</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>Over 1 ¼</td>
<td>100</td>
</tr>
<tr>
<td>0.46 to 0.55 inclusive</td>
<td>To 1 ¼ inclusive</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td>1 ¼ - 3</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>Over 3</td>
<td>300</td>
</tr>
<tr>
<td>Over 0.75</td>
<td>Any</td>
<td>No Welding Permitted</td>
</tr>
</tbody>
</table>

Table 19-4

Preheat and Interpass Temperatures for A7 Steel

If A7 steel is to be retained and welded, the following note shall be added to the plans with the appropriate temperature from Table 9-4:

BEFORE AND DURING ANY WELDING OPERATION TO A7 (INCLUDING SHEAR STUDS), THE STEEL SHALL BE HEATED TO A TEMPERATURE OF XXX DEGREES FAHRENHEIT.
19.4.7 Fatigue

If fatigue sensitive details (AASHTO category D, E, or E’) fall within the scope of the work, they shall be analyzed for remaining life using accepted methods. Notch effects, such as rivet holes and nonradius cuts, cause stress increases. The designer should consider removing or retrofitting all poor details, fatigue sensitive details and stress risers of all types. Lateral connection plates should not be welded to tension flanges. Rivet holes should be made round by reaming to eliminate crack initiation sites. Often when widening or connecting two new structures, new load paths are created. The designer should carefully consider the stiffness of the new members and how the older adjacent members should be strengthened in order to carry the new loadings.

19.4.8 Partial Length Coverplate Retrofits

There are many existing highway bridges with steel beams constructed prior to the recognition of the low fatigue resistance of partial length cover plates.

When rehabilitating structures with partial length coverplates calculate the remaining fatigue life in accordance the Section 7 of the AASHTO Manual for Bridge Evaluation. If the remaining fatigue life is inadequate, the beam coverplates should be retrofitted using the end bolted detail shown in Fig. 10.3.1C in the NYSDOT Standard Specifications for Highway Bridges or Fig. 6.6.1.2.3-1 in the NYSDOT LRFD Bridge Design Specifications or use Ultrasonic Impact Treatment. When adding cover plate retrofits designers need to verify that the minimum allowable vertical clearance is not violated.

Designers can contact the Metals Engineering Unit for the cost information associated with this retrofit.

19.5 Continuity Retrofit

19.5.1 Feasibility

Continuity retrofits require DCES approval and should not be considered for structures with skews greater than 30 degrees.

During a rehabilitation project, the expansion joint at a pier can be eliminated by splicing the simple spans together to form a continuous girder. Benefits include reducing the possibility of deterioration of the girder and substructure due to a leaky joint, increasing resistance to seismic displacements, and slightly improving the load carrying capacity of the superstructure.

However, continuity retrofit can result in undesirable structural performance characteristics that must be addressed in the design. Increased vulnerability to fatigue may result due to areas of the existing beams being subjected to stress reversals and higher stress ranges compared to simple span behavior. The end regions of retrofitted girders originally designed for simple span positive moments of small magnitude are subjected to larger magnitude negative moments. While the deck joints over the interior supports are eliminated, the deck in this area is subjected to tension under service loads and crack control measures must be considered. Continuity can also increase seismic loads on individual piers depending on bearing fixity configurations.
The scope of a project may help determine when it is appropriate to retrofit two or more simple spans into one continuous span. For a rehabilitation that involves a deck overlay only, the extra cost of concrete removal required to retrofit the simple span may be beyond the project scope. However, if deck scarification, deep removal and joint replacement are also scheduled as part of the rehabilitation, a cost assessment should be done to determine if retrofitting the simple span girders to be continuous is reasonable. Complete deck replacement projects provide excellent opportunities to include girder retrofit since the girders will be readily accessible and the future costs of maintaining the joints will be eliminated. The cost of providing continuity retrofits for full deck replacement projects should be compared to the cost of replacing the girders. This is particularly relevant when the cost of cleaning and painting the existing steel is required for the retrofit alternate.

19.5.2 General Design Considerations

19.5.2.1 Full Continuity vs. Continuous for Live Load

When considering using a continuity retrofit, a decision must be made whether the girders will be made fully continuous or continuous for live load only. Representative details of a fully continuous and a continuous for live load retrofit splice are shown in Figure 19.1.

Compared to continuous for live load only designs, fully continuous retrofits require more complex splice and retrofit details. However, a retrofit that provides full continuity for both dead and live loads is advantageous because the combined girder should behave like a conventional continuous girder. Since this retrofit requires so much more of the girder to be exposed in the area of the splice, a fully continuous retrofit should only be done in conjunction with a full deck replacement. Another benefit is that the existing two lines of bearings at the pier can be replaced by a single bearing line.
Figure 19.1
Typical Retrofit Details
On the negative side, the existing beam sections adjacent to the pier may not be able to adequately resist the increased moments and shears associated with a continuous beam without supplemental cover plating. If a fully continuous retrofit proves to be structurally difficult or uneconomical, an alternative is to make the span continuous for live load only (see Design Guideline No. 16, below).

Continuous for live load retrofits adapt well structurally to situations where the deck is being retained. Although the splice details are simpler than those for fully continuous retrofit, two lines of bearings must be retained at each splice.

Continuous for live load designs in conjunction with complete deck replacements require a deck placement sequence consistent with the design assumptions. All such design assumptions, including a construction sequence, shall be clearly documented in the contract plans as well as reflected in the design load, moment and shear, and haunch tables. In some cases, the continuity splice for continuous for live load designs may not accommodate a future continuous deck replacement unless the deck removal and replacement follows the original continuous for live load design assumptions. Such sequences could require loosening and reinstalling the splice. Alternatively, continuous for live load retrofits can be designed to accommodate unrestricted full deck replacements. (See Design Guideline No. 17, below).

19.5.2.2 Fatigue Considerations

Continuity retrofits often put fatigue sensitive details originally intended to only be in compression into tension and/or stress reversal. All connection details in areas of tension or stress reversal should be analyzed for the stress ranges induced by the retrofit. Details of particular importance to check are butt welded splices, partial length cover plate ends, welded lateral gusset plate connections, connection plate/stiffener welds and shear connector welds in tension or reversal zones. Nondestructive testing should be performed on butt welded top flange splices to ensure weld soundness. Upgrading of fatigue sensitive details using bolted over-splicing of partial length cover plate ends should also be considered to meet the allowable fatigue stresses as per Article 10.3 of the Standard Specifications for Highway Bridges. Excessive fatigue stresses or unreasonable costs to upgrade fatigue sensitive details may dictate that a continuity retrofit not be performed. Riveted girders should not be retrofitted for continuity due to their uncertain fatigue performance and difficult splice detail requirements.

19.5.2.3 Detail Verification

As-built plans and/or shop drawings should be reviewed followed by a thorough site inspection making note of material condition, fatigue prone details, utilities, geometry, girder alignment, and possible paint removal and containment considerations.
19.5.3 Design Guidelines

1. The retrofitted span should be analyzed as fully continuous or continuous for live load to determine the new moments and increased shears induced over the interior support. The bolted flange splices shall be designed to carry the new moments, while the web splice shall be designed to carry the increased shear. The existing piers and bearings (if being retained) shall be analyzed for the increased reactions due to continuity. As a minimum, the splice is made the same section size as the beams.

2. Continuity retrofits should be designed for an HS 20 live load. Upgrading the superstructure to an HS 25 design is not required.

3. One method of increasing the design moment capacity of the continuous girder is to increase the girder’s section properties by adding bolted cover plates to the flanges of the existing girders.

4. The negative moment capacity of the girder may be enhanced by considering the girder over the pier as a composite section. Using this method, the longitudinal reinforcing steel in the deck is included in computing the composite section properties. If not damaged, the stud shear connectors for the simple span beams may be left in place during deck removal operations. In most cases, the existing shear connectors are adequate to provide composite action in the negative moment region between the girder and the longitudinal reinforcing steel in the deck. Spiral shear connectors should be replaced with stud shear connectors because of the difficulty of removing concrete around the spirals.

5. For both fully continuous and continuous for live load retrofits, additional longitudinal reinforcing steel must be installed in the tension regions of the continuous deck. If the full deck is not being replaced, a portion of the existing deck concrete over the interior supports shall be removed to install the additional reinforcement. The deck concrete shall then be replaced as a continuous placement after the girder continuity splices are installed. For fully continuous retrofits, the provisions of the Standard Specifications for Highway Bridges (Art. 10.38.4.3) should be applied. This negative moment deck reinforcement should extend to the points of dead load contraflexure plus the development length. For continuous for live load retrofits, the reinforcement needs only to extend to where the combined dead load plus the negative live load moments equal zero, plus the development length. The minimum continuous for live load reinforcement provided may be per Standard Specifications for Highway Bridges (Art. 10.38.4.3) or be based on concrete crack control requirements.

6. Filler plates may be used to make up differences in thickness between flanges or between webs to be spliced. The minimum filler plate thickness allowed is ⅛” (Note: This allowance of filler plates for continuity retrofit splices is an exception to the NYSDOT general prohibition of their use in girder bolted splices). Machined splice plates have been used to make up thickness differences, however, these plates are more expensive than filler plates and generally not necessary.
7. Bolts through the bottom flanges must be arranged to avoid interfering with the bearing(s). The use of countersunk bolts through the bottom splice plates in the area over the bearing may reduce this interference, as well as reduce the length of the splice.

8. Installing the splice may require removing the existing end diaphragms and bearing stiffeners. A new line of diaphragms and bearing stiffeners should be placed over the centerline of the new bearings. Rolled beams may not have bearing stiffeners. In this case, new bearing stiffeners should be designed and installed to provide support and stiffness. New bearing stiffeners shall be bolted to the web splice assembly.

9. The remaining expansion joints on the structure, if there are any, should be checked to verify that they can handle the thermal expansion of the continuous girders. If it is determined that new joints are required, they should be designed with the current design procedure for expansion joints.

10. The existing pier shall be analyzed for any increased longitudinal or seismic loading caused by the continuity retrofit. Current seismic retrofit criteria should be reviewed. Pedestals and capbeam repair or replacement may be required due to deterioration of pier concrete below the joint connecting the simple spans or due to new bearing and pedestal requirements.

11. The designer shall consider the constructability, variations in girder alignment and end gap differences between adjoining girders. The designer should consider larger splice plates to provide extra edge distance for field fit-up. Field confirmation of dimensions and steel condition is essential.

12. Caution is advised when using continuity retrofits with stage construction. The design must carefully consider construction sequencing. Each stage shall be structurally independent during the retrofitting process. In no case shall two simple spans be attached to a deck continuous over a pier. Diaphragms in the bay between the staging need to be temporarily disconnected.

13. Continuity retrofits have been installed on a few bridges with horizontally curved decks and straight girders set on chords. Such retrofitting should be considered only when the angle between beams to be spliced is small (i.e., less than 4°). Flange splice plates must be cut to fit the splice geometry or oversized plates may be used if dimensions permit. Bent plates are used for the web splice. Lateral force effects from the angled continuous beams must be considered in investigating the retrofit details and bearings.

14. For a retrofit made in conjunction with a full deck replacement, a new deck haunch table using continuous concrete dead load and super-imposed dead load deflections shall be provided. The haunch table shall be developed in conformance with the design assumptions (fully continuous or continuous for live load) and proposed deck placement sequence. Corresponding moment and shear tables shall also be provided.
15. A fully continuous retrofit includes replacing the existing two lines of simple-span bearings with one line of bearings for the continuous girder. When replacing the bearings, care must be taken to insure that the elevation of the superstructure remains the same. Tapered sole plates may be required to maintain proper grade and elevations. The pedestals may also have to be modified or replaced. If space constraints hamper work on the existing pedestals, height adjustments may have to be made in the bearing plates. A construction sequence for lifting girders and installing bearings shall be provided on the plans.

16. For continuous for live load retrofits, the two lines of bearings from the existing simple span configuration are retained. Only the girder flanges need to be spliced. The top flange splice shall be made using conventional bolted splice plates. The bottom (live load compression) flange splice may be bolted, or be achieved using a compression block fitted and welded between the flange ends. Continuous for live load retrofits require that the deck be in place, except for the areas over the piers, prior to installing the splice. Continuity closure pours over the pier are then placed after splicing the girders.

17. It is advisable to check the behavior of continuous for live load retrofits for a future full deck removal and replacement. Removing a continuous for live load deck will impart a positive moment into a continuity splice that was primarily designed for negative live load plus superimposed dead load moments only. Uplift at the pier bearings is also theoretically possible upon removing the deck. Normally, this load case should not be a problem since the reduced stiffness of a continuous for live load splice relative to the girders as a whole should minimize the magnitude of this moment redistribution. This behavior would only occur during a temporary construction condition, therefore some overstress allowance is reasonable. Similar behavior during a future deck replacement could also occur with fully continuous retrofits that were installed while the existing deck was retained.

19.6 **Truss Rehabilitation**

Early involvement with the Metals Engineering Unit is highly recommended on all truss rehabilitation projects. The following should be addressed during design:

- It is important to consider the original construction, and the need to bring the structure up to current AASHTO code requirements for strength, service life, and fatigue resistance.

- An in-depth inspection of the steel needs to be performed during the scoping phase and the extent of deterioration must be clearly identified on the contract plans.

- The steel used on many trusses fabricated before the advent of modern carbon steel does not have the weldability or the resistance to fatigue that the replacement steel adds to the structure. In some instances it is important to consider the retrofit or reconfiguration of the design connections because of the level of stress that the stronger steel will introduce. This may require the replacement of more steel in order to have a fatigue resistant load path.
Welded repairs for older steels are cost prohibitive due to the very rigorous controls required on the welding processes. Therefore, bolted repairs should generally be specified on most truss rehabilitation projects.

When trusses have pre-existing welded repairs to tension members, or other welded attachments to tension members, these welds shall be removed and ground flush. In the case of I-bar structures, these details can introduce serious defects in the fracture critical members. For these cases, repair procedures should be requested from the Metals Engineering Unit.

Contract plans shall identify all main members in tension.

Fracture critical truss members shall be called out in a separate listing. This callout requires the contractor to follow the fracture critical control plan during fabrication. Fracture critical members include the bottom chord of the truss (in tension areas), the vertical and diagonals in tension, and in some truss configurations may include the end portals. Additionally, floorbeams shall be considered fracture critical when the center-to-center spacing exceeds 12 feet.

19.7 Seismic Rehabilitation

All bridges that are scheduled for rehabilitation shall be evaluated with regard to seismic failure vulnerability. The purpose of this evaluation is to assess seismic retrofit measures and to incorporate into the rehabilitation plans those measures deemed warranted to eliminate or mitigate such failure vulnerability. (See the Bridge Safety Assurance Seismic Vulnerability Manual.) Policy and specifications for seismic design and rehabilitation are contained in the Standard Specifications for Highway Bridges.
Section 22
Maintenance

22.1 Introduction

The purpose of this section is to call the designer's attention to the importance of future maintenance considerations during the design process. The goal of all designers should be to design bridges that will require as little maintenance as possible and make it as easy as possible to do the maintenance that is necessary. Well-thought-out details at the design stage can often accomplish this with little or no increased initial cost or effort.

For details on recommended maintenance procedures and practices, the current edition of the AASHTO Maintenance Manual: The Maintenance and Management of Roadways and Bridges, should be consulted.

22.2 Geometrics

A significant factor in bridge maintenance cost is the skew angle of the bridge. It has been observed for a number of years that bridges with skews have more maintenance problems than square bridges. Additionally, it appears that the problems (steel fatigue, deck cracking, leaking joints, etc.) become larger as the skew angle of the bridge increases. It is understood that site conditions usually dictate skew angles, but anything that can mitigate this feature should be investigated.

A bridge skew can also cause a problem when the skew angle of deck joints matches the angle of snow plow blades. In this situation, a plow blade can catch on the joint, causing serious consequences for the joint, plow and driver. It is preferable to adjust the skew angle slightly to avoid this problem. Plow blades are usually set at an angle of approximately 37°. Designers should check with the Regional Office if this issue is of concern and verify the typical plow blade angle for that Region.

Other geometric factors that can influence maintenance costs are vertical clearances of under roadways (maximize as much as possible to avoid oversized vehicle impact) and the profile (avoid flat grades of less than 0.3% to prevent ponding of water on the deck). The placement of sag vertical curves on bridges should be avoided, if possible. Curbless bridges are preferred because of their superior drainage characteristics.

22.3 Deck Joints and Drainage

The most important single factor in increasing bridge maintenance costs is the presence of deck joints. It can be generally assumed that, in time, all joints will leak. Leaking joints are responsible for the majority of deterioration of underlying bridge components.
A designer should, therefore, do everything possible to eliminate deck joints. This means that continuous spans should be used in lieu of simple spans when possible. Integral and semi-integral or jointless abutments should be used, when possible, at the ends of bridges to eliminate joints at those locations. Designers should think of the deck as a roof for the structural elements below. A properly designed roof will be watertight and will effectively drain itself so as not to create dams which will inevitably leak and cause drainage to the elements below.

Scuppers and gratings should also be items of maintenance concern to designers. Scupper downspouts need to be designed to carry their effluent beyond the structural elements they are there to protect. Downspouts, other than short straight vertical outlets, should be designed with cleanouts. If diffusers are used on downspouts, care should be taken to avoid them spraying on substructure elements.

Scuppers are not used as frequently as they once were. The wider shoulders provided on new structures because of current geometric policy have a larger hydraulic capacity than older structures. This has eliminated the need for scuppers in many situations. Although they may not be necessary hydraulically, it may sometimes be a good idea to place scuppers near a joint of a curved bridge that has a flat or nearly flat grade to prevent water ponding over a deck joint.

Open steel grating should never be used in new construction as it exposes the underlying structure to salt laden water. In rehabilitation projects, consideration should be given to filling open steel grating with concrete. If this is not possible because of loading considerations, some benefit can be gained by filling only the ends of the spans to protect the substructures in those locations.

22.4 Approach Drainage

When the approach highway section has curbs, drainage inlets must be provided on both sides just off the high end of the bridge. This is necessary to prevent approach drainage from being carried onto the bridge. See Section 13 for more information.

22.5 Superstructure

22.5.1 Material Type

The two principal structural materials, steel and concrete, have very different characteristics relative to their need and ease of maintenance. Steel tends to need more maintenance than concrete, but it is relatively easy to repair. Concrete, especially prestressed concrete, does not need maintenance as frequently as steel, but it may be difficult or impossible to repair.

One of the best ways to reduce maintenance on steel structures is the use of weathering steel. When used in the proper locations (See Section 8), the elimination of periodic painting is a significant benefit.