To reflect the findings in research, new products or changes in design concepts, revised portions of the manual, or addendum, will be issued to maintain a current Bridge Design Manual. The agency or its representative using this manual is responsible for compliance with not only the Manual but any addendums.

The Office of Structural Engineering offers a web page, the Manual and addenda at:

http://www.dot.state.oh.us/se/

Any comments or suggestions you may have to better the manual should be addressed to the Ohio Department of Transportation, Office of Structural Engineering, 1980 W. Broad Street, Columbus, Ohio 43223.

In this manual “Department” shall refer to the “Office of Structural Engineering” or “its representatives” including any district office production Administrator responsible for plan review or any consultant contracted by the Department to perform bridge plan review. Where the “Office of Structural Engineering” is specifically referenced in the manual only the ODOT Office of Structural Engineering shall be the governing authority. The user of this manual (i.e. Consulting engineers, ODOT District Personnel, County and other governmental agencies) will be referred to as the “Design Agency”. The owner of the project will be referred to as the “Appointing Authority”.

Updates and Revisions to this manual will be released on a quarterly basis as needed. The revisions will be available in Adobe Acrobat Reader (.pdf) format from the Department’s publication website:

http://www.dot.state.oh.us/drrc/

Subscribers to the ODOT Bridge Design Manual from this website will receive an email notification when revisions are released.

The format of the revisions will be in the form of replacement or insert pages to the existing manual. The replacement pages will include the date of the revision in the upper right corner of the page. The revised or new text will be marked with a single vertical line in the right hand margin of the page.
AASHTO  American Association of State Highway and Transportation Officials

ADT     Average Daily Traffic

ADTT    Average Daily Truck Traffic

AISC    American Institute of Steel Construction

AREMA   American Railway Engineering And Maintenance-of-way Association

ASTM    American Society for Testing and Materials

AWS     American Welding Society

CMS     Construction and Materials Specifications

CVN     Charpy V-Notch

FHWA    Federal Highway Administration

IZEU    Inorganic Zinc Epoxy Urethane

OZEU    Organic Zinc Epoxy Urethane

MSE     Mechanically Stabilized Earth

NCHRP   National Cooperative Highway Research Program

NDT     Non Destructive Testing

NFIP    National Flood Insurance Program

ODOT    Ohio Department of Transportation

OZEU    Organic Zinc Epoxy Urethane

USGS    United States Geological Survey
<table>
<thead>
<tr>
<th>SECTION 100 – GENERAL INFORMATION</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>101 INTRODUCTION</td>
<td>1-1</td>
</tr>
<tr>
<td>101.1 GENERAL CONSIDERATIONS</td>
<td>1-1</td>
</tr>
<tr>
<td>101.2 TABLE OF ORGANIZATION</td>
<td>1-1</td>
</tr>
<tr>
<td>102 PREPARATION OF PLANS</td>
<td>1-1</td>
</tr>
<tr>
<td>102.1 BRIDGE DESIGN, CHECK AND REVIEW REQUIREMENTS</td>
<td>1-3</td>
</tr>
<tr>
<td>102.2 MANUAL DRAFTING STANDARDS</td>
<td>1-4</td>
</tr>
<tr>
<td>102.2.1 GENERAL</td>
<td>1-4</td>
</tr>
<tr>
<td>102.2.2 LETTERING STANDARDS</td>
<td>1-4</td>
</tr>
<tr>
<td>102.2.3 MANUAL DRAFTING LINE STANDARDS</td>
<td>1-4</td>
</tr>
<tr>
<td>102.3 COMPUTER AIDED DRAFTING STANDARDS</td>
<td>1-4</td>
</tr>
<tr>
<td>102.4 DESIGNER, CHECKER, REVIEWER INITIALS BLOCK</td>
<td>1-4</td>
</tr>
<tr>
<td>102.5 TITLE BLOCK</td>
<td>1-4</td>
</tr>
<tr>
<td>102.6 ESTIMATED QUANTITIES</td>
<td>1-10</td>
</tr>
<tr>
<td>102.7 STANDARD BRIDGE DRAWINGS</td>
<td>1-11</td>
</tr>
<tr>
<td>102.8 SUPPLEMENTAL SPECIFICATIONS</td>
<td>1-12</td>
</tr>
<tr>
<td>102.9 PROPOSAL NOTES</td>
<td>1-12</td>
</tr>
<tr>
<td>103 COMPUTER PROGRAMS</td>
<td>1-12</td>
</tr>
<tr>
<td>103.1 GEOMETRIC PROGRAMS</td>
<td>1-13</td>
</tr>
<tr>
<td>103.2 DESIGN PROGRAMS</td>
<td>1-13</td>
</tr>
<tr>
<td>103.3 HYDRAULIC ENGINEERING PROGRAMS</td>
<td>1-13</td>
</tr>
<tr>
<td>103.4 GEOTECHNICAL ENGINEERING PROGRAMS</td>
<td>1-13</td>
</tr>
<tr>
<td>103.5 BRIDGE RATING PROGRAM</td>
<td>1-14</td>
</tr>
<tr>
<td>104 OHIO REVISED CODE SUBMITTALS</td>
<td>1-14</td>
</tr>
<tr>
<td>105 BRIDGE PLAN SHEET ORDER</td>
<td>1-14</td>
</tr>
</tbody>
</table>
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>GENERAL</td>
<td>2-1</td>
</tr>
<tr>
<td>201.1</td>
<td>STRUCTURE TYPE STUDY SUBMISSION REQUIREMENTS</td>
<td>2-1</td>
</tr>
<tr>
<td>201.2</td>
<td>PROFILE FOR EACH BRIDGE ALTERNATIVE</td>
<td>2-2</td>
</tr>
<tr>
<td>201.2.1</td>
<td>PRELIMINARY STRUCTURE SITE PLAN</td>
<td>2-2</td>
</tr>
<tr>
<td>201.2.2</td>
<td>HYDRAULIC REPORT</td>
<td>2-4</td>
</tr>
<tr>
<td>201.2.3</td>
<td>NARRATIVE OF BRIDGE ALTERNATIVES</td>
<td>2-6</td>
</tr>
<tr>
<td>201.2.4</td>
<td>FOUNDATION RECOMMENDATION</td>
<td>2-7</td>
</tr>
<tr>
<td>201.2.5</td>
<td>PRELIMINARY MAINTENANCE OF TRAFFIC PLAN</td>
<td>2-7</td>
</tr>
<tr>
<td>201.3</td>
<td>UTILITIES</td>
<td>2-8</td>
</tr>
<tr>
<td>202</td>
<td>BRIDGE PRELIMINARY DESIGN REPORT</td>
<td>2-9</td>
</tr>
<tr>
<td>202.1</td>
<td>GENERAL</td>
<td>2-9</td>
</tr>
<tr>
<td>202.2</td>
<td>BRIDGE PRELIMINARY DESIGN REPORT SUBMISSION REQUIREMENTS</td>
<td>2-9</td>
</tr>
<tr>
<td>202.2.1</td>
<td>FINAL STRUCTURE SITE PLAN</td>
<td>2-9</td>
</tr>
<tr>
<td>202.2.2</td>
<td>FINAL MAINTENANCE OF TRAFFIC PLAN</td>
<td>2-10</td>
</tr>
<tr>
<td>202.2.3</td>
<td>FOUNDATION REPORT</td>
<td>2-11</td>
</tr>
<tr>
<td>202.2.3.1</td>
<td>SPREAD FOOTINGS</td>
<td>2-11</td>
</tr>
<tr>
<td>202.2.3.2</td>
<td>PILE FOUNDATIONS</td>
<td>2-12</td>
</tr>
<tr>
<td>202.2.3.2.a</td>
<td>STEEL ‘H’ PILES</td>
<td>2-12</td>
</tr>
<tr>
<td>202.2.3.2.b</td>
<td>CAST-IN-PLACE REINFORCED CONCRETE PILES</td>
<td>2-13</td>
</tr>
<tr>
<td>202.2.3.2.c</td>
<td>DOWN DRAG FORCES ON PILES</td>
<td>2-14</td>
</tr>
<tr>
<td>202.2.3.2.d</td>
<td>PILE WALL THICKNESS</td>
<td>2-15</td>
</tr>
<tr>
<td>202.2.3.2.e</td>
<td>PILE HAMMER SIZE</td>
<td>2-15</td>
</tr>
<tr>
<td>202.2.3.2.f</td>
<td>CONSTRUCTION CONSTRAINTS</td>
<td>2-15</td>
</tr>
<tr>
<td>202.2.3.2.g</td>
<td>PREBORED HOLES</td>
<td>2-15</td>
</tr>
<tr>
<td>202.2.3.3</td>
<td>DRILLED SHAFTS</td>
<td>2-15</td>
</tr>
<tr>
<td>202.2.4</td>
<td>SUPPLEMENTAL SITE PLAN FOR RAILWAY CROSSINGS</td>
<td>2-16</td>
</tr>
<tr>
<td>203</td>
<td>BRIDGE WATERWAY</td>
<td>2-17</td>
</tr>
<tr>
<td>203.1</td>
<td>HYDROLOGY</td>
<td>2-17</td>
</tr>
<tr>
<td>203.2</td>
<td>HYDRAULIC ANALYSIS</td>
<td>2-18</td>
</tr>
<tr>
<td>203.3</td>
<td>SCOUR</td>
<td>2-20</td>
</tr>
<tr>
<td>203.4</td>
<td>BRIDGE AND WATERWAY PERMITS</td>
<td>2-21</td>
</tr>
<tr>
<td>204</td>
<td>SUBSTRUCTURE INFORMATION</td>
<td>2-21.1</td>
</tr>
<tr>
<td>204.1</td>
<td>FOOTING ELEVATIONS</td>
<td>2-21.1</td>
</tr>
<tr>
<td>204.2</td>
<td>EARTH BENCHES AND SLOPES</td>
<td>2-22</td>
</tr>
<tr>
<td>204.3</td>
<td>ABUTMENT TYPES</td>
<td>2-22</td>
</tr>
<tr>
<td>204.4</td>
<td>ABUTMENTS SUPPORTED ON MSE WALLS</td>
<td>2-22</td>
</tr>
<tr>
<td>204.5</td>
<td>PIER TYPES</td>
<td>2-23</td>
</tr>
<tr>
<td>204.6</td>
<td>RETAINING WALLS</td>
<td>2-23</td>
</tr>
<tr>
<td>204.6.1</td>
<td>DESIGN CONSTRAINTS</td>
<td>2-23</td>
</tr>
<tr>
<td>204.6.2</td>
<td>STAGE 1 DETAIL DESIGN SUBMISSION FOR RETAINING WALLS</td>
<td>2-25</td>
</tr>
<tr>
<td>204.6.2.1</td>
<td>PROPRIETARY WALLS</td>
<td>2-25</td>
</tr>
<tr>
<td>204.6.2.2</td>
<td>CAST-IN-PLACE WALLS</td>
<td>2-27.1</td>
</tr>
<tr>
<td>204.6.2.3</td>
<td>OTHER WALLS</td>
<td>2-27.1</td>
</tr>
<tr>
<td>205</td>
<td>SUPERSTRUCTURE INFORMATION</td>
<td>2-27.1</td>
</tr>
<tr>
<td>205.1</td>
<td>TYPE OF STRUCTURES</td>
<td>2-27.1</td>
</tr>
<tr>
<td>205.2</td>
<td>SPAN ARRANGEMENTS</td>
<td>2-28</td>
</tr>
<tr>
<td>205.3</td>
<td>CONCRETE SLABS</td>
<td>2-28</td>
</tr>
<tr>
<td>205.4</td>
<td>Prestressed Concrete Box Beams</td>
<td>2-29</td>
</tr>
<tr>
<td>205.5</td>
<td>Prestressed Concrete I-Beams</td>
<td>2-29</td>
</tr>
<tr>
<td>205.6</td>
<td>Steel Beams and Girders</td>
<td>2-30</td>
</tr>
<tr>
<td>205.7</td>
<td>Composite Design</td>
<td>2-31</td>
</tr>
<tr>
<td>205.8</td>
<td>Integral Design</td>
<td>2-32</td>
</tr>
<tr>
<td>Section</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>------</td>
</tr>
<tr>
<td>205.9</td>
<td>SEMI-INTEGRAL DESIGN</td>
<td>2-32</td>
</tr>
<tr>
<td>206</td>
<td>MINIMAL BRIDGE PROJECTS</td>
<td>2-34</td>
</tr>
<tr>
<td>207</td>
<td>BRIDGE GEOMETRICS</td>
<td>2-34</td>
</tr>
<tr>
<td>207.1</td>
<td>VERTICAL CLEARANCE</td>
<td>2-34</td>
</tr>
<tr>
<td>207.2</td>
<td>BRIDGE SUPERSTRUCTURE</td>
<td>2-35</td>
</tr>
<tr>
<td>207.3</td>
<td>LATERAL CLEARANCE</td>
<td>2-35</td>
</tr>
<tr>
<td>207.4</td>
<td>INTERFERENCE DUE TO EXISTING SUBSTRUCTURE</td>
<td>2-35</td>
</tr>
<tr>
<td>207.5</td>
<td>BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION</td>
<td>2-35.1</td>
</tr>
<tr>
<td>208</td>
<td>TEMPORARY SHORING</td>
<td>2-35.1</td>
</tr>
<tr>
<td>208.1</td>
<td>SUPPORT OF EXCAVATIONS</td>
<td>2-35.1</td>
</tr>
<tr>
<td>208.2</td>
<td>SUPPORT OF EXISTING STRUCTURE</td>
<td>2-36.1</td>
</tr>
<tr>
<td>209</td>
<td>MISCELLANEOUS</td>
<td>2-37</td>
</tr>
<tr>
<td>209.1</td>
<td>TRANSVERSE DECK SECTION WITH SUPERELEVATION</td>
<td>2-37</td>
</tr>
<tr>
<td>209.1.1</td>
<td>SUPERELEVATION TRANSITIONS</td>
<td>2-37</td>
</tr>
<tr>
<td>209.2</td>
<td>BRIDGE RAILINGS</td>
<td>2-38</td>
</tr>
<tr>
<td>209.3</td>
<td>BRIDGE DECK DRAINAGE</td>
<td>2-38</td>
</tr>
<tr>
<td>209.4</td>
<td>SLOPE PROTECTION</td>
<td>2-39</td>
</tr>
<tr>
<td>209.5</td>
<td>APPROACH SLABS</td>
<td>2-40</td>
</tr>
<tr>
<td>209.6</td>
<td>PRESSURE RELIEF JOINTS</td>
<td>2-40</td>
</tr>
<tr>
<td>209.7</td>
<td>AESTHETICS</td>
<td>2-40</td>
</tr>
<tr>
<td>209.8</td>
<td>RAILWAY BRIDGES</td>
<td>2-41</td>
</tr>
<tr>
<td>209.9</td>
<td>BICYCLE BRIDGES</td>
<td>2-43</td>
</tr>
<tr>
<td>209.10</td>
<td>PEDESTRIAN BRIDGES</td>
<td>2-43</td>
</tr>
<tr>
<td>209.11</td>
<td>SIDEWALKS ON BRIDGES</td>
<td>2-43</td>
</tr>
<tr>
<td>209.12</td>
<td>MAINTENANCE AND INSPECTION ACCESS</td>
<td>2-44</td>
</tr>
<tr>
<td>209.13</td>
<td>SIGN SUPPORTS</td>
<td>2-44</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS

SECTION 300 – DETAIL DESIGN ......................................................................................................................... 3-1
  301 GENERAL .................................................................................................................................................. 3-1
    301.1 DESIGN PHILOSOPHY ......................................................................................................................... 3-1
    301.2 DETAIL DESIGN REVIEW SUBMISSIONS ............................................................................................ 3-1
    301.3 DESIGN METHODS ............................................................................................................................... 3-2
    301.4 LOADING REQUIREMENTS .................................................................................................................. 3-2.1
      301.4.1 PEDESTRIAN AND BIKEWAY BRIDGES ................................................................................ 3-2.1
      301.4.2 RAILROAD BRIDGES .............................................................................................................. 3-2.1
      301.4.3 SEISMIC DESIGN ...................................................................................................................... 3-3
    301.5 REINFORCING STEEL .......................................................................................................................... 3-4
      301.5.1 MAXIMUM LENGTH ....................................................................................................................... 3-4
      301.5.2 BAR MARKS ................................................................................................................................. 3-4
      301.5.3 LAP SPLICES ................................................................................................................................. 3-4
      301.5.4 CALCULATING LENGTHS AND WEIGHTS OF REINFORCING .................................................. 3-5
      301.5.5 BAR LIST ..................................................................................................................................... 3-6
      301.5.6 USE OF EPOXY COATED REINFORCING STEEL ............................................................... 3-7
      301.5.7 MINIMUM CONCRETE COVER FOR REINFORCING ............................................................ 3-7
      301.5.8 MINIMUM REINFORCING STEEL ............................................................................................ 3-7
    301.6 REFERENCE LINE ............................................................................................................................... 3-7
    301.7 UTILITIES ........................................................................................................................................... 3-8
      301.7.1 UTILITIES ATTACHED TO BEAMS AND GIRDERS .................................................................... 3-8
    301.8 CONSTRUCTION JOINTS, NEW CONSTRUCTION ........................................................................ 3-8.1

302 SUPERSTRUCTURE .................................................................................................................................... 3-9
  302.1 GENERAL CONCRETE REQUIREMENTS ............................................................................................. 3-9
    302.1.1 CONCRETE DESIGN ALLOWABLES .............................................................................................. 3-9
    302.1.2 SUPERSTRUCTURE CONCRETE TYPES .................................................................................... 3-9
      302.1.2.1 CLASS S & HP CONCRETE, QC/QA CONCRETE FOR STRUCTURES & CONCRETE WITH WARRANTY .................................................................................................................. 3-9
      302.1.2.2 SELECTION OF CONCRETE FOR BRIDGE STRUCTURES .................................................. 3-10
    302.1.3 WEARING SURFACE ......................................................................................................................... 3-10
      302.1.3.1 TYPES ....................................................................................................................................... 3-10
      302.1.3.2 FUTURE WEARING SURFACE ............................................................................................. 3-11
    302.1.4 CONCRETE DECK PROTECTION ..................................................................................................... 3-11
      302.1.4.1 TYPES ....................................................................................................................................... 3-11
      302.1.4.2 WHEN TO USE ............................................................................................................................. 3-11.1
      302.1.4.3 SEALING OF CONCRETE SURFACES SUPERSTRUCTURE ................................................ 3-12
  302.2 REINFORCED CONCRETE DECK ON STRINGERS .............................................................................. 3-13
    302.2.1 DECK THICKNESS ........................................................................................................................... 3-13
    302.2.2 CONCRETE DECK DESIGN ........................................................................................................... 3-14
    302.2.3 DECK ELEVATION REQUIREMENTS ............................................................................................... 3-14
      302.2.3.1 SCREED ELEVATIONS .............................................................................................................. 3-14
      302.2.3.2 TOP OF HAUNCH ELEVATIONS ............................................................................................ 3-15
      302.2.3.3 FINAL DECK SURFACE ELEVATIONS .................................................................................. 3-15
    302.2.4 REINFORCEMENT ............................................................................................................................. 3-15.1
      302.2.4.1 LONGITUDINAL ......................................................................................................................... 3-15.1
      302.2.4.2 TRANSVERSE ............................................................................................................................ 3-16
    302.2.5 HAUNCHED DECK REQUIREMENTS ............................................................................................... 3-16
    302.2.6 STAY IN PLACE FORMS .................................................................................................................. 3-16
    302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS .................................................................... 3-16
      302.2.7.1 FINISHING MACHINES ............................................................................................................. 3-17
      302.2.7.2 SOURCES OF GIRDER TWIST ............................................................................................... 3-17.2
        302.2.7.2.a GLOBAL SUPERSTRUCTURE DISTORTION ...................................................................... 3-17.3
        302.2.7.2.b OIL-CANNING ....................................................................................................................... 3-17.6
        302.2.7.2.c GIRDER WARping ................................................................................................................ 3-17.7
      302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST ........................................................................ 3-17.11
    302.2.8 SLAB DEPTH OF CURVED BRIDGES ............................................................................................. 3-17.12

3-i
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>302.4.1</td>
<td>GENERAL</td>
<td>3-18</td>
</tr>
<tr>
<td>302.4.1.1</td>
<td>MATERIAL REQUIREMENTS</td>
<td>3-19</td>
</tr>
<tr>
<td>302.4.1.2</td>
<td>ATTACHMENTS</td>
<td>3-19</td>
</tr>
<tr>
<td>302.4.1.3</td>
<td>STEEL FABRICATION QUALIFICATION</td>
<td>3-19</td>
</tr>
<tr>
<td>302.4.1.4</td>
<td>MAXIMUM AVAILABLE LENGTH OF STEEL MEMBER</td>
<td>3-20</td>
</tr>
<tr>
<td>302.4.1.5</td>
<td>STRUCTURAL STEEL COATINGS</td>
<td>3-20</td>
</tr>
<tr>
<td>302.4.1.5.a</td>
<td>PRIMARY COATING SYSTEMS</td>
<td>3-20</td>
</tr>
<tr>
<td>302.4.1.5.b</td>
<td>ALTERNATIVE COATING SYSTEMS</td>
<td>3-21</td>
</tr>
<tr>
<td>302.4.1.5.c</td>
<td>EXISTING STEEL COATING SYSTEMS</td>
<td>3-22</td>
</tr>
<tr>
<td>302.4.1.6</td>
<td>STEEL PIER CAP</td>
<td>3-23</td>
</tr>
<tr>
<td>302.4.1.7</td>
<td>OUTSIDE MEMBER CONSIDERATIONS</td>
<td>3-23</td>
</tr>
<tr>
<td>302.4.1.8</td>
<td>CAMBER AND DEFLECTIONS</td>
<td>3-23</td>
</tr>
<tr>
<td>302.4.1.9</td>
<td>FATIGUE</td>
<td>3-24</td>
</tr>
<tr>
<td>302.4.1.9.a</td>
<td>LOADING</td>
<td>3-24</td>
</tr>
<tr>
<td>302.4.1.9.b</td>
<td>STRESS CATEGORY</td>
<td>3-25</td>
</tr>
<tr>
<td>302.4.1.10</td>
<td>TOUGHNESS TESTS</td>
<td>3-25</td>
</tr>
<tr>
<td>302.4.1.11</td>
<td>STANDARD END CROSS FRAMES</td>
<td>3-25</td>
</tr>
<tr>
<td>302.4.1.12</td>
<td>BASELINE REQUIREMENTS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES</td>
<td>3-25</td>
</tr>
<tr>
<td>302.4.1.13</td>
<td>INTERMEDIATE EXPANSION DEVICES</td>
<td>3-25</td>
</tr>
<tr>
<td>302.4.1.14</td>
<td>BOLTED SPLICES</td>
<td>3-26</td>
</tr>
<tr>
<td>302.4.1.14.a</td>
<td>BOLTS</td>
<td>3-27</td>
</tr>
<tr>
<td>302.4.1.14.b</td>
<td>EDGE DISTANCES</td>
<td>3-27</td>
</tr>
<tr>
<td>302.4.1.14.c</td>
<td>LOCATION OF FIELD SPLICES</td>
<td>3-27</td>
</tr>
<tr>
<td>302.4.1.15</td>
<td>SHEAR CONNECTORS</td>
<td>3-27</td>
</tr>
<tr>
<td>302.4.2</td>
<td>ROLLED BEAMS</td>
<td>3-28</td>
</tr>
<tr>
<td>302.4.2.1</td>
<td>GALVANIZED BEAM STRUCTURES</td>
<td>3-28</td>
</tr>
<tr>
<td>302.4.2.2</td>
<td>STIFFENERS</td>
<td>3-28.1</td>
</tr>
<tr>
<td>302.4.2.3</td>
<td>INTERMEDIATE CROSS FRAMES</td>
<td>3-29</td>
</tr>
<tr>
<td>302.4.2.4</td>
<td>WELDS</td>
<td>3-31</td>
</tr>
<tr>
<td>302.4.2.4.a</td>
<td>MINIMUM SIZE OF FILLET WELD</td>
<td>3-31</td>
</tr>
<tr>
<td>302.4.2.4.b</td>
<td>NON-DESTRUCTIVE INSPECTION OF WELDS</td>
<td>3-31</td>
</tr>
<tr>
<td>302.4.2.5</td>
<td>MOMENT PLATES</td>
<td>3-31</td>
</tr>
<tr>
<td>302.4.3</td>
<td>GIRDER S</td>
<td>3-32</td>
</tr>
<tr>
<td>302.4.3.1</td>
<td>GENERAL</td>
<td>3-32</td>
</tr>
<tr>
<td>302.4.3.2</td>
<td>FRACTURE CRITICAL</td>
<td>3-32</td>
</tr>
<tr>
<td>302.4.3.3</td>
<td>WIDTH &amp; THICKNESS REQUIREMENTS</td>
<td>3-33</td>
</tr>
<tr>
<td>302.4.3.3.a</td>
<td>FLANGES</td>
<td>3-33</td>
</tr>
<tr>
<td>302.4.3.3.b</td>
<td>WEBS</td>
<td>3-34</td>
</tr>
<tr>
<td>302.4.3.4</td>
<td>INTERMEDIATE STIFFENERS</td>
<td>3-34</td>
</tr>
<tr>
<td>302.4.3.5</td>
<td>INTERMEDIATE CROSS FRAMES</td>
<td>3-34</td>
</tr>
<tr>
<td>302.4.3.6</td>
<td>WELDS</td>
<td>3-35</td>
</tr>
<tr>
<td>302.4.3.6.a</td>
<td>TYPES</td>
<td>3-35</td>
</tr>
<tr>
<td>302.4.3.6.b</td>
<td>MINIMUM SIZE OF FILLET AND COMPLETE PENETRATION WELDS, PLAN REQUIREMENTS</td>
<td>3-35</td>
</tr>
<tr>
<td>302.4.3.6.c</td>
<td>INSPECTION OF WELDS, WHAT TO SHOW ON PLANS</td>
<td>3-36</td>
</tr>
<tr>
<td>302.4.3.7</td>
<td>CURVED GIRDER DESIGN REQUIREMENTS</td>
<td>3-36</td>
</tr>
<tr>
<td>302.5</td>
<td>PRESTRESSED CONCRETE BEAMS</td>
<td>3-37</td>
</tr>
<tr>
<td>302.5.1</td>
<td>BOX BEAMS</td>
<td>3-37</td>
</tr>
<tr>
<td>302.5.1.1</td>
<td>DESIGN REQUIREMENTS</td>
<td>3-38</td>
</tr>
<tr>
<td>302.5.1.2</td>
<td>STRANDS</td>
<td>3-39</td>
</tr>
<tr>
<td>302.5.1.2.a</td>
<td>TYPE, SIZE OF STRANDS</td>
<td>3-39</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS

302.5.1.2.b  SPACING ................................................................. 3-39
302.5.1.2.c  STRESSES .............................................................. 3-40
302.5.1.3  COMPOSITE .............................................................. 3-40
302.5.1.4  NON-COMPOSITE WEARING SURFACE ....................... 3-41
302.5.1.5  CAMBER .................................................................. 3-41
302.5.1.6  ANCHORAGE ............................................................. 3-42
302.5.1.7  CONCRETE MATERIALS FOR BOX BEAMS ................... 3-42
302.5.1.8  REINFORCING ........................................................... 3-43
302.5.1.9  TIE RODS ................................................................. 3-43
302.5.2  I-BEAMS .................................................................. 3-43
302.5.2.1  DESIGN REQUIREMENTS .......................................... 3-44
302.5.2.2  STRANDS ................................................................. 3-44
302.5.2.2.a  TYPE, SIZE ........................................................... 3-44
302.5.2.2.b  SPACING ............................................................... 3-45
302.5.2.2.c  STRESSES ............................................................. 3-45
302.5.2.2.d  DEBONDING .......................................................... 3-46
302.5.2.2.e  DRAPEING ............................................................ 3-46
302.5.2.3  CAMBER .................................................................. 3-47
302.5.2.4  ANCHORAGE ............................................................. 3-48
302.5.2.5  DECK SUPERSTRUCTURE AND PRECAST DECK PANEL .... 3-49
302.5.2.6  DIAPHRAGMS ........................................................... 3-49
302.5.2.7  DECK POURING SEQUENCE ....................................... 3-49
302.5.2.8  CONCRETE MATERIALS FOR I-BEAMS ....................... 3-50
302.5.2.9  REINFORCING .......................................................... 3-50
302.5.2.10  TRANSPORTATION & HANDLING CONSIDERATIONS ....... 3-50
303  SUBSTRUCTURE ................................................................. 3-51
303.1  GENERAL .................................................................. 3-51
303.1.1  SEALING OF CONCRETE SURFACES, SUBSTRUCTURE ....... 3-51
303.2  ABUTMENTS ................................................................. 3-52
303.2.1  GENERAL ................................................................. 3-52
303.2.1.1  PRESSURE RELIEF JOINTS FOR RIGID PAVEMENT ........... 3-52.1
303.2.1.2  BEARING SEAT WIDTH .............................................. 3-53
303.2.1.3  BEARING SEAT REINFORCEMENT ............................... 3-53
303.2.1.4  PHASED CONSTRUCTION JOINTS ................................. 3-53
303.2.2  TYPES OF ABUTMENTS ............................................... 3-54
303.2.2.1  FULL HEIGHT ABUTMENTS ......................................... 3-54
303.2.2.1.a  COUNTERFORTS FOR FULL HEIGHT ABUTMENTS ....... 3-55
303.2.2.1.b  SEALING STRIP FOR FULL HEIGHT ABUTMENTS ......... 3-55
303.2.2.2  CONCRETE SLAB BRIDGES ON RIGID ABUTMENTS ....... 3-55
303.2.2.3  STUB ABUTMENTS WITH SPILL THRU SLOPES ............... 3-55
303.2.2.4  CAPPED PILE STUB ABUTMENTS ................................. 3-56
303.2.2.5  SPREAD FOOTING TYPE ABUTMENTS ........................... 3-56
303.2.2.6  INTEGRAL ABUTMENTS .............................................. 3-56
303.2.2.7  SEMI-INTEGRAL ABUTMENTS ..................................... 3-57
303.2.3  ABUTMENT DRAINAGE ................................................. 3-58
303.2.3.1  BACKWALL DRAINAGE .............................................. 3-58
303.2.3.2  BRIDGE SEAT DRAINAGE .......................................... 3-59
303.2.3.3  WEEP HOLES IN WALL TYPE ABUTMENTS AND RETAINING WALLS ........ 3-59
303.2.4  WINGWALLS .............................................................. 3-59
303.2.5  EXPANSION AND CONTRACTION JOINTS ......................... 3-60
303.2.6  REINFORCING, “U” AND CANTILEVER WINGS .................. 3-60
303.2.7  FILLS AT ABUTMENTS .................................................. 3-60
303.3  PIERS ........................................................................ 3-61
303.3.1  GENERAL ................................................................. 3-61
303.3.1.1  BEARING SEAT WIDTHS ............................................ 3-61
303.3.1.2  PIER PROTECTION IN WATERWAYS ............................... 3-62
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>303.3.2</td>
<td>TYPES OF PIERS</td>
<td>3-62</td>
</tr>
<tr>
<td>303.3.2.1</td>
<td>CAP AND COLUMN PIERS</td>
<td>3-62</td>
</tr>
<tr>
<td>303.3.2.2</td>
<td>CAP AND COLUMN PIERS ON PILES</td>
<td>3-63</td>
</tr>
<tr>
<td>303.3.2.3</td>
<td>CAP AND COLUMN PIERS ON DRILLED SHAFTS</td>
<td>3-63</td>
</tr>
<tr>
<td>303.3.2.4</td>
<td>CAP AND COLUMN PIERS ON SPREAD FOOTINGS</td>
<td>3-63</td>
</tr>
<tr>
<td>303.3.2.5</td>
<td>CAPPED PILE PIERS</td>
<td>3-64</td>
</tr>
<tr>
<td>303.3.2.6</td>
<td>STEEL CAP PIERS</td>
<td>3-64</td>
</tr>
<tr>
<td>303.3.2.7</td>
<td>POST-TENSIONED CONCRETE PIER CAPS</td>
<td>3-65</td>
</tr>
<tr>
<td>303.3.2.8</td>
<td>T-TYPE PIERS</td>
<td>3-65</td>
</tr>
<tr>
<td>303.3.2.9</td>
<td>PIER USE ON RAILWAY STRUCTURES</td>
<td>3-66</td>
</tr>
<tr>
<td>303.3.10</td>
<td>PIERS ON NAVIGABLE WATERWAYS</td>
<td>3-66</td>
</tr>
<tr>
<td>303.3.11</td>
<td>PIER CAP REINFORCING STEEL STIRRUPS</td>
<td>3-66</td>
</tr>
<tr>
<td>303.3.3</td>
<td>FOOTING ON PILES</td>
<td>3-66</td>
</tr>
<tr>
<td>303.4</td>
<td>FOUNDATIONS</td>
<td>3-66</td>
</tr>
<tr>
<td>303.4.1</td>
<td>MINIMUM DEPTH OF FOOTINGS</td>
<td>3-66</td>
</tr>
<tr>
<td>303.4.1.1</td>
<td>FOOTING, RESISTANCE TO HORIZONTAL FORCES</td>
<td>3-67</td>
</tr>
<tr>
<td>303.4.1.2</td>
<td>LOCATION OF RESULTANT FORCES ON FOOTINGS</td>
<td>3-69</td>
</tr>
<tr>
<td>303.4.1.3</td>
<td>REINFORCING STEEL IN FOOTINGS</td>
<td>3-70</td>
</tr>
<tr>
<td>303.4.2</td>
<td>PILE FOUNDATIONS</td>
<td>3-70</td>
</tr>
<tr>
<td>303.4.2.1</td>
<td>PILES, PLAN SHEET REQUIREMENTS</td>
<td>3-70</td>
</tr>
<tr>
<td>303.4.2.2</td>
<td>PILES, NUMBER &amp; SPACING</td>
<td>3-71</td>
</tr>
<tr>
<td>303.4.2.3</td>
<td>PILES BATTERED</td>
<td>3-71</td>
</tr>
<tr>
<td>303.4.2.4</td>
<td>PILES, DESIGN LOADS</td>
<td>3-71</td>
</tr>
<tr>
<td>303.4.2.5</td>
<td>PILES, STATIC LOAD TEST</td>
<td>3-72</td>
</tr>
<tr>
<td>303.4.2.6</td>
<td>PILES, DYNAMIC LOAD TEST</td>
<td>3-73</td>
</tr>
<tr>
<td>303.4.2.7</td>
<td>PILE FOUNDATION – DESIGN EXAMPLE</td>
<td>3-73</td>
</tr>
<tr>
<td>303.4.3</td>
<td>DRILLED SHAFTS</td>
<td>3-75</td>
</tr>
<tr>
<td>303.5</td>
<td>DETAIL DESIGN REQUIREMENTS FOR PROPRIETARY RETAINING WALLS</td>
<td>3-76</td>
</tr>
<tr>
<td>303.5.1</td>
<td>WORK PERFORMED BY THE DESIGN AGENCY</td>
<td>3-77</td>
</tr>
<tr>
<td>303.5.2</td>
<td>WORK PERFORMED BY THE PROPRIETARY WALL COMPANIES</td>
<td>3-80</td>
</tr>
<tr>
<td>304</td>
<td>RAILING</td>
<td>3-82</td>
</tr>
<tr>
<td>304.1</td>
<td>GENERAL</td>
<td>3-82</td>
</tr>
<tr>
<td>304.2</td>
<td>STANDARD RAILING TYPES</td>
<td>3-84</td>
</tr>
<tr>
<td>304.3</td>
<td>WHEN TO USE</td>
<td>3-84</td>
</tr>
<tr>
<td>304.4</td>
<td>PARAPET TYPE (BR-1 &amp; SBR-1-99)</td>
<td>3-84</td>
</tr>
<tr>
<td>304.4.2</td>
<td>DEEP BEAM BRIDGE GUARDRAIL (DBR-2-73)</td>
<td>3-83</td>
</tr>
<tr>
<td>304.4.3</td>
<td>TWIN STEL TUBE BRIDGE RAILING (TST-1-99)</td>
<td>3-84</td>
</tr>
<tr>
<td>304.4.4</td>
<td>BRIDGE RETRO-FIT RAILING, THRIE BEAM BRIDGE RAILING FOR BRIDGES WITH SAFETY CURBS (TBR-91)</td>
<td>3-84</td>
</tr>
<tr>
<td>304.4.5</td>
<td>PORTABLE CONCRETE BARRIER (PCB-91)</td>
<td>3-84</td>
</tr>
<tr>
<td>304.4.6</td>
<td>BRIDGE SIDEWALK RAILING WITH CONCRETE PARAPETS (BR-2-98)</td>
<td>3-85</td>
</tr>
<tr>
<td>305</td>
<td>FENCING</td>
<td>3-86</td>
</tr>
<tr>
<td>305.1</td>
<td>GENERAL</td>
<td>3-86</td>
</tr>
<tr>
<td>305.2</td>
<td>WHEN TO USE</td>
<td>3-86</td>
</tr>
<tr>
<td>305.3</td>
<td>FENCING CONFIGURATIONS</td>
<td>3-87</td>
</tr>
<tr>
<td>305.4</td>
<td>SPECIAL DESIGNS</td>
<td>3-88</td>
</tr>
<tr>
<td>305.5</td>
<td>FENCE DESIGN GENERAL REQUIREMENTS</td>
<td>3-88</td>
</tr>
<tr>
<td>305.5.1</td>
<td>WIND LOADS</td>
<td>3-89</td>
</tr>
<tr>
<td>306</td>
<td>EXPANSION DEVICES</td>
<td>3-90</td>
</tr>
<tr>
<td>306.1</td>
<td>GENERAL</td>
<td>3-90</td>
</tr>
<tr>
<td>306.1.1</td>
<td>PAY ITEM</td>
<td>3-90</td>
</tr>
<tr>
<td>306.1.2</td>
<td>EXPANSION DEVICES WITH SIDEWALKS</td>
<td>3-90</td>
</tr>
<tr>
<td>306.1.3</td>
<td>EXPANSION DEVICES WITH STAGE CONSTRUCTION</td>
<td>3-91</td>
</tr>
<tr>
<td>306.2</td>
<td>EXPANSION DEVICES WITH SIDEWALKS</td>
<td>3-91</td>
</tr>
<tr>
<td>306.2.1</td>
<td>ABUTMENT JOINTS IN BITUMINOUS CONCRETE BOX BEAM BRIDGES</td>
<td>3-91</td>
</tr>
<tr>
<td>306.2.2</td>
<td>ABUTMENT JOINTS AS PER AS-1-81</td>
<td>3-91</td>
</tr>
<tr>
<td>Section</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>306.2.3</td>
<td>EXPANSION JOINTS USING POLYMER MODIFIED ASPHALT BINDER</td>
<td></td>
</tr>
<tr>
<td>306.2.4</td>
<td>STRIP SEAL EXPANSION DEVICES</td>
<td></td>
</tr>
<tr>
<td>306.2.5</td>
<td>COMPRESSION SEAL EXPANSION DEVICES</td>
<td></td>
</tr>
<tr>
<td>306.2.6</td>
<td>STEEL SLIDING PLATE ENDDAMS, RETIRED STANDARD DRAWING SD-1-69</td>
<td></td>
</tr>
<tr>
<td>306.2.7</td>
<td>MODULAR EXPANSION DEVICES</td>
<td></td>
</tr>
<tr>
<td>306.2.8</td>
<td>TOOTH TYPE, FINGER TYPE OR NON-STANDARD SLIDING PLATE EXPANSION DEVICES</td>
<td></td>
</tr>
<tr>
<td>306.2.9</td>
<td>EXPANSION DEVICE USES – BRIDGE OR ABUTMENT TYPE</td>
<td></td>
</tr>
<tr>
<td>306.3.1</td>
<td>INTEGRAL OR SEMI-INTEGRAL TYPE ABUTMENTS</td>
<td></td>
</tr>
<tr>
<td>306.3.2</td>
<td>REINFORCED CONCRETE SLAB BRIDGES</td>
<td></td>
</tr>
<tr>
<td>306.3.3</td>
<td>STEEL STRINGER BRIDGES</td>
<td></td>
</tr>
<tr>
<td>306.3.4</td>
<td>PRESTRESSED CONCRETE I-BEAM BRIDGES</td>
<td></td>
</tr>
<tr>
<td>306.3.5</td>
<td>NON-COMPOSITE PRESTRESSED BOX BEAM BRIDGES</td>
<td></td>
</tr>
<tr>
<td>306.3.6</td>
<td>COMPOSITE PRESTRESSED CONCRETE BOX BEAM BRIDGES</td>
<td></td>
</tr>
<tr>
<td>306.3.7</td>
<td>ALL TIMBER STRUCTURES</td>
<td></td>
</tr>
<tr>
<td>307.1</td>
<td>GENERAL</td>
<td></td>
</tr>
<tr>
<td>307.2</td>
<td>BEARING TYPES</td>
<td></td>
</tr>
<tr>
<td>307.2.1</td>
<td>ELASTOMERIC BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.2.2</td>
<td>STEEL ROCKER &amp; BOLSTER BEARINGS, RB-1-55</td>
<td></td>
</tr>
<tr>
<td>307.2.3</td>
<td>SLIDING BRONZE TYPE &amp; FIXED TYPE STEEL BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.2.4</td>
<td>SPECIALIZED BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.2.4.1</td>
<td>POT TYPE BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.2.4.2</td>
<td>DISC TYPE BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.2.4.3</td>
<td>SPHERICAL TYPE BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3</td>
<td>GUIDELINES FOR USE</td>
<td></td>
</tr>
<tr>
<td>307.3.1</td>
<td>FIXED BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.1.1</td>
<td>FIXED TYPE STEEL BEARINGS (RB-1-55 OR FB-1-82)</td>
<td></td>
</tr>
<tr>
<td>307.3.1.2</td>
<td>FIXED LAMINATED ELASTOMERIC BEARINGS FOR STEEL BEAM BRIDGES</td>
<td></td>
</tr>
<tr>
<td>307.3.1.3</td>
<td>FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS</td>
<td></td>
</tr>
<tr>
<td>307.3.1.4</td>
<td>FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS</td>
<td></td>
</tr>
<tr>
<td>307.3.2</td>
<td>EXPANSION BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.2.1</td>
<td>ROCKER BEARINGS (RB-1-55)</td>
<td></td>
</tr>
<tr>
<td>307.3.2.2</td>
<td>BRONZE TYPE STEEL EXPANSION BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.2.3</td>
<td>EXPANSION ELASTOMERIC BEARINGS FOR BEAM AND GIRDGER BRIDGES</td>
<td></td>
</tr>
<tr>
<td>307.3.2.4</td>
<td>EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS</td>
<td></td>
</tr>
<tr>
<td>307.3.2.5</td>
<td>EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS</td>
<td></td>
</tr>
<tr>
<td>307.3.3</td>
<td>SPECIALIZED BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.3.1</td>
<td>POT BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.3.2</td>
<td>DISC TYPE BEARINGS</td>
<td></td>
</tr>
<tr>
<td>307.3.3.3</td>
<td>SPHERICAL BEARINGS</td>
<td></td>
</tr>
</tbody>
</table>
SECTION 400 – REHABILITATION & REPAIR................................................................. 4-1

401 GENERAL ................................................................................................................. 4-1
  401.1 DESIGN CONSIDERATIONS .............................................................................. 4-1
  401.2 STRENGTH ANALYSIS .................................................................................. 4-2

402 STRUCTURAL STEEL .............................................................................................. 4-2
  402.1 DAMAGE OR SECTION LOSS ......................................................................... 4-2
  402.2 FATIGUE ANALYSIS ...................................................................................... 4-2
  402.3 FATIGUE RETROFIT ...................................................................................... 4-3
    402.3.1 END BOLTED COVER PLATES ................................................................. 4-3
    402.3.2 BOX GIRDER PIER CAPS ................................................................. 4-4
    402.3.3 MISCELLANEOUS FATIGUE RETROFITS ............................................. 4-4
  402.4 STRENGTHENING OF STRUCTURAL STEEL MEMBERS ......................... 4-4
  402.5 TRIMMING BEAM ENDS .............................................................................. 4-5
  402.6 HEAT STRAIGHTENING ................................................................................. 4-5
  402.7 HINGE ASSEMBLIES ...................................................................................... 4-5
  402.8 BOLTS ........................................................................................................... 4-5

403 CONCRETE REPAIR/RESTORATION (OTHER THAN DECK REPAIR) .............. 4-6
  403.1 GENERAL ...................................................................................................... 4-6
  403.2 PATCHING .................................................................................................... 4-6
  403.3 CRACK REPAIR ............................................................................................ 4-6

404 BRIDGE DECK REPAIR ....................................................................................... 4-7
  404.1 OVERLAYS ON AN OVERLAY .................................................................... 4-7
  404.2 OVERLAYS ................................................................................................... 4-7
  404.3 UNDER DECK REPAIR .............................................................................. 4-8

405 BRIDGE DECK REPLACEMENT ........................................................................... 4-9
  405.1 ELIMINATION OF LONGITUDINAL DECK JOINT ..................................... 4-9
  405.2 DECK HAUNCH ............................................................................................. 4-9
  405.3 CLOSURE POUR ........................................................................................... 4-10
  405.4 CONCRETE PLACEMENT SEQUENCE ...................................................... 4-10
    405.4.1 STANDARD BRIDGES .......................................................................... 4-10
    405.4.2 STRUCTURES WITH INTERMEDIATE HINGES ..................................... 4-10

406 EXPANSION JOINT RETROFIT ......................................................................... 4-11

407 RAISING AND JACKING BRIDGES ..................................................................... 4-12

408 BRIDGE DRAINAGE .............................................................................................. 4-13

409 WIDENING ........................................................................................................... 4-13
  409.1 CLOSURE POUR ............................................................................................ 4-13
  409.2 SUPERSTRUCTURE DEFLECTIONS ............................................................. 4-14
  409.3 FOUNDATIONS ............................................................................................ 4-15
    409.3.1 WIDENED STRUCTURES .................................................................. 4-15
    409.3.2 SCOUR CONSIDERATIONS .............................................................. 4-15
  409.4 CONCRETE SLAB BRIDGES ....................................................................... 4-15
  409.5 PIER COLUMNS ............................................................................................ 4-16

410 RAILING .............................................................................................................. 4-16
  410.1 FACING ......................................................................................................... 4-16
  410.2 REMOVAL FLUSH WITH THE TOP OF THE DECK .................................... 4-16
  410.3 THRIE BEAM RETROFIT ............................................................................ 4-17

411 BEARINGS ........................................................................................................... 4-17

412 CONCRETE BRIDGE DECK REPAIR QUANTITY ESTIMATING ..................... 4-17
  412.1 SPECIAL REQUIREMENTS FOR QUANTITY ESTIMATING FOR BRIDGES 500 FEET [150 m] OR GREATER IN LENGTH .............................................. 4-17
  412.2 ACTUAL QUANTITIES, ESTIMATING FACTORS ....................................... 4-19

413 REFERENCES ....................................................................................................... 4-21
<table>
<thead>
<tr>
<th>SECTION 500 – TEMPORARY STRUCTURES</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>501 GENERAL</td>
<td>5-1</td>
</tr>
<tr>
<td>502 PRELIMINARY DESIGN</td>
<td>5-1</td>
</tr>
<tr>
<td>502.1 HYDRAULICS</td>
<td>5-1</td>
</tr>
<tr>
<td>503 DETAIL DESIGN</td>
<td>5-1</td>
</tr>
<tr>
<td>504 GENERAL NOTES</td>
<td>5-3</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS

January 2010

<table>
<thead>
<tr>
<th>SECTION 600 – TYPICAL GENERAL NOTES</th>
<th>6-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>601 DESIGN REFERENCES</td>
<td>6-1</td>
</tr>
<tr>
<td>601.1 GENERAL</td>
<td>6-1</td>
</tr>
<tr>
<td>601.2 STANDARD DRAWINGS AND SUPPLEMENTAL SPECIFICATIONS</td>
<td>6-1</td>
</tr>
<tr>
<td>601.3 DESIGN SPECIFICATIONS</td>
<td>6-1</td>
</tr>
<tr>
<td>602 DESIGN DATA</td>
<td>6-2</td>
</tr>
<tr>
<td>602.1 DESIGN LOADING</td>
<td>6-2</td>
</tr>
<tr>
<td>602.2 DESIGN STRESSES</td>
<td>6-3</td>
</tr>
<tr>
<td>602.3 FOR RAILWAY PROJECTS</td>
<td>6-7</td>
</tr>
<tr>
<td>602.4 DECK PROTECTION METHOD</td>
<td>6-8</td>
</tr>
<tr>
<td>602.5 MONOLITHIC WEARING SURFACE</td>
<td>6-9</td>
</tr>
<tr>
<td>602.6 SEALING OF CONCRETE SURFACES</td>
<td>6-9</td>
</tr>
<tr>
<td>603 EXISTING STRUCTURE REMOVAL NOTES</td>
<td>6-9</td>
</tr>
<tr>
<td>603.1 CONCRETE DECK REMOVAL PROJECTS</td>
<td>6-10</td>
</tr>
<tr>
<td>604 TEMPORARY STRUCTURE CONSTRUCTION</td>
<td>6-13</td>
</tr>
<tr>
<td>605 EMBANKMENT CONSTRUCTION</td>
<td>6-13</td>
</tr>
<tr>
<td>605.1 FOUNDATIONS ON PILES IN NEW EMBANKMENTS</td>
<td>6-13</td>
</tr>
<tr>
<td>605.2 FOUNDATIONS ON SPREAD FOOTINGS IN NEW EMBANKMENTS</td>
<td>6-14</td>
</tr>
<tr>
<td>605.3 EMBANKMENT CONSTRUCTION NOTE</td>
<td>6-15</td>
</tr>
<tr>
<td>605.4 UNCLASSIFIED EXCAVATION</td>
<td>6-15</td>
</tr>
<tr>
<td>605.5 PROPRIETARY RETAINING WALLS</td>
<td>6-16</td>
</tr>
<tr>
<td>606 FOUNDATIONS</td>
<td>6-16.1</td>
</tr>
<tr>
<td>606.1 PILES DRIVEN TO BEDROCK</td>
<td>6-16.1</td>
</tr>
<tr>
<td>606.2 FRICTION TYPE PILES</td>
<td>6-17</td>
</tr>
<tr>
<td>606.3 STEEL PILE POINTS</td>
<td>6-19</td>
</tr>
<tr>
<td>606.4 PILE SPLICES</td>
<td>6-20</td>
</tr>
<tr>
<td>606.5 MINIMUM HAMMER SIZE</td>
<td>6-20</td>
</tr>
<tr>
<td>606.6 PILE ENCASEMENT</td>
<td>6-20</td>
</tr>
<tr>
<td>606.7 FOUNDATION BEARING PRESSURE</td>
<td>6-20.1</td>
</tr>
<tr>
<td>606.7.1 SPREAD FOOTINGS NOT ON BEDROCK</td>
<td>6-21</td>
</tr>
<tr>
<td>606.8 FOOTINGS</td>
<td>6-22</td>
</tr>
<tr>
<td>606.9 DRILLED SHAFTS</td>
<td>6-22</td>
</tr>
<tr>
<td>607 MAINTENANCE OF TRAFFIC</td>
<td>6-23</td>
</tr>
<tr>
<td>608 RAILROAD GRADE SEPARATION PROJECTS</td>
<td>6-23</td>
</tr>
<tr>
<td>608.1 CONSTRUCTION CLEARANCE</td>
<td>6-23</td>
</tr>
<tr>
<td>608.2 RAILROAD AERIAL LINES</td>
<td>6-23</td>
</tr>
<tr>
<td>608.3 RAILROAD STRUCTURAL STEEL</td>
<td>6-24</td>
</tr>
<tr>
<td>609 UTILITY LINES</td>
<td>6-24</td>
</tr>
<tr>
<td>610 REHABILITATION OF EXISTING STRUCTURES</td>
<td>6-24</td>
</tr>
<tr>
<td>610.1 EXISTING STRUCTURE VERIFICATION</td>
<td>6-24</td>
</tr>
<tr>
<td>610.2 REINFORCING STEEL REPLACEMENT</td>
<td>6-25</td>
</tr>
<tr>
<td>610.3 REHABILITATION – STRUCTURAL STEEL</td>
<td>6-25</td>
</tr>
<tr>
<td>610.4 REFURBISHED BEARINGS</td>
<td>6-26</td>
</tr>
<tr>
<td>610.5 JACKING BRIDGE SUPERSTRUCTURES</td>
<td>6-26</td>
</tr>
<tr>
<td>610.6 FATIGUE MEMBER INSPECTION</td>
<td>6-27</td>
</tr>
<tr>
<td>610.7 RAILING</td>
<td>6-29</td>
</tr>
<tr>
<td>611 MISCELLANEOUS GENERAL NOTES</td>
<td>6-30</td>
</tr>
<tr>
<td>611.1 DOWEL HOLES</td>
<td>6-30</td>
</tr>
<tr>
<td>611.2 APPROACH SLABS</td>
<td>6-30</td>
</tr>
<tr>
<td>611.3 INTEGRAL AND SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEALS</td>
<td>6-31</td>
</tr>
<tr>
<td>611.4 BACKWALL DRAINAGE</td>
<td>6-32</td>
</tr>
<tr>
<td>611.5 CONCRETE PARAPET SAWCUT JOINTS</td>
<td>6-32</td>
</tr>
<tr>
<td>611.6 BEARING PAD SHIMS, PRESTRESSED</td>
<td>6-32</td>
</tr>
<tr>
<td>611.7 CLEANING STEEL IN PATCHES</td>
<td>6-33</td>
</tr>
<tr>
<td>611.8 CONVERSION OF STANDARD BRIDGE DRAWINGS</td>
<td>6-33</td>
</tr>
<tr>
<td>611.9 COFFERDAMS AND EXCAVATION BRACING</td>
<td>6-33</td>
</tr>
</tbody>
</table>
### TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section Reference</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>611.10</td>
<td>DECK PLACEMENT NOTES</td>
<td>6-34</td>
</tr>
<tr>
<td>611.10.1</td>
<td>FALSEWORK AND FORMS</td>
<td>6-34</td>
</tr>
<tr>
<td>611.10.2</td>
<td>DECK PLACEMENT DESIGN ASSUMPTIONS</td>
<td>6-34</td>
</tr>
</tbody>
</table>
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Page</th>
<th>Section</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-1</td>
<td>701</td>
<td>Substructure Details</td>
</tr>
<tr>
<td>7-1</td>
<td>701.1</td>
<td>Steel Sheet Piling</td>
</tr>
<tr>
<td>7-1</td>
<td>701.2</td>
<td>Porous Backfill</td>
</tr>
<tr>
<td>7-1</td>
<td>701.3</td>
<td>Bridge Seat Reinforcing</td>
</tr>
<tr>
<td>7-2</td>
<td>701.4</td>
<td>Bridge Seat Elevations for Elastomeric Bearings</td>
</tr>
<tr>
<td>7-2</td>
<td>701.5</td>
<td>Proper Seating of Steel Beams at Abutments</td>
</tr>
<tr>
<td>7-2</td>
<td>701.6</td>
<td>Backwall Concrete Placement for Prestressed Box Beams</td>
</tr>
<tr>
<td>7-3</td>
<td>701.7</td>
<td>Sealing of Beam Seats</td>
</tr>
<tr>
<td>7-3</td>
<td>702</td>
<td>Superstructure Details</td>
</tr>
<tr>
<td>7-3</td>
<td>702.1</td>
<td>Steel Beam Deflection and Camber</td>
</tr>
<tr>
<td>7-3</td>
<td>702.2</td>
<td>Steel Notch Toughness Requirement (Charpy V-Notch)</td>
</tr>
<tr>
<td>7-4</td>
<td>702.3</td>
<td>High Strength Bolts</td>
</tr>
<tr>
<td>7-4</td>
<td>702.4</td>
<td>Scuppers</td>
</tr>
<tr>
<td>7-4</td>
<td>702.5</td>
<td>Elastomeric Bearing Load Plate</td>
</tr>
<tr>
<td>7-4</td>
<td>702.6</td>
<td>Bearing Repositioning</td>
</tr>
<tr>
<td>7-4</td>
<td>702.7</td>
<td>Concrete Placement Sequence Notes</td>
</tr>
<tr>
<td>7-4</td>
<td>702.7.1</td>
<td>Concrete Intermediate Diaphragm for Prestressed Concrete I-Beams</td>
</tr>
<tr>
<td>7-5</td>
<td>702.7.2</td>
<td>Semi-Integral or Integral Abutment Concrete Placement for Diaphragms</td>
</tr>
<tr>
<td>7-7</td>
<td>702.8</td>
<td>Concrete Deck Slab Depth and Pay Quantities</td>
</tr>
<tr>
<td>7-7</td>
<td>702.9</td>
<td>Concrete Deck Haunch Widths</td>
</tr>
<tr>
<td>7-7</td>
<td>702.10</td>
<td>Prestressed Concrete I-Beam Bridges</td>
</tr>
<tr>
<td>7-8</td>
<td>702.11</td>
<td>Prestressed Concrete Box Beam Bridge</td>
</tr>
<tr>
<td>7-10</td>
<td>702.12</td>
<td>Asphalt Concrete Surface Course</td>
</tr>
<tr>
<td>7-11</td>
<td>702.13</td>
<td>Painting of A588/A709 Grade 50 Steel</td>
</tr>
<tr>
<td>7-11</td>
<td>702.14</td>
<td>Erection Bolts</td>
</tr>
<tr>
<td>7-12</td>
<td>702.15</td>
<td>Welded Attachments</td>
</tr>
<tr>
<td>7-12</td>
<td>702.16</td>
<td>Deck Elevation Tables</td>
</tr>
<tr>
<td>7-12</td>
<td>702.16.1</td>
<td>Screed Elevation Tables</td>
</tr>
<tr>
<td>7-12</td>
<td>702.16.2</td>
<td>Top of Haunch Elevation Tables</td>
</tr>
<tr>
<td>7-12</td>
<td>702.16.3</td>
<td>Final Deck Surface Elevation Tables</td>
</tr>
<tr>
<td>7-13</td>
<td>702.17</td>
<td>Steel Drip Strip</td>
</tr>
<tr>
<td>7-13</td>
<td>702.18</td>
<td>Reinforcing Steel for Rehabilitation</td>
</tr>
<tr>
<td>7-13</td>
<td>702.19</td>
<td>Elastomeric Bearing Material Requirements</td>
</tr>
<tr>
<td>7-13</td>
<td>702.20</td>
<td>Bearing Seat Adjustments for Special Bearings</td>
</tr>
<tr>
<td>7-14</td>
<td>702.21</td>
<td>Haunched Girder Fabrication Note</td>
</tr>
<tr>
<td>7-14</td>
<td>702.22</td>
<td>Fracture Critical Fabrication Note</td>
</tr>
<tr>
<td>7-14</td>
<td>702.23</td>
<td>Welded Shear Connectors on Galvanized Structures</td>
</tr>
<tr>
<td>7-14</td>
<td>703</td>
<td>Site Plan Requirements for Section 401 and 404 of the Clean Water Act</td>
</tr>
</tbody>
</table>
SECTION 800 – NOISE BARRIERS

8-1
SECTION 900 – BRIDGE LOAD RATING ............................................................................................................. 9-1

901 PURPOSE .................................................................................................................................................. 9-1
902 SCOPE ...................................................................................................................................................... 9-1
903 APPLICABILITY ......................................................................................................................................... 9-1
904 QUALITY MEASURES ............................................................................................................................... 9-1
905 DEFINITIONS AND TERMINOLOGY ........................................................................................................ 9-1
906 REFERENCES FROM OHIO REVISED CODE .......................................................................................... 9-4
907 BRIDGE FILES (RECORDS) ...................................................................................................................... 9-5

907.1 CONSTRUCTION PLANS .......................................................................................................................... 9-6
907.2 CONSTRUCTION & MATERIAL SPECIFICATIONS .................................................................................. 9-6
907.3 SHOP AND WORKING DRAWINGS ......................................................................................................... 9-6
907.4 AS-BUILT DRAWINGS ............................................................................................................................. 9-6
907.5 CORRESPONDENCE ............................................................................................................................... 9-6
907.6 INVENTORY DATA ................................................................................................................................. 9-6
907.7 INSPECTION HISTORY .......................................................................................................................... 9-7
907.8 PHOTOGRAPHS .................................................................................................................................... 9-7
907.9 RATING RECORDS ................................................................................................................................. 9-7
907.10 ACCIDENT DATA ................................................................................................................................. 9-7
907.11 MAINTENANCE AND REPAIR HISTORY ............................................................................................ 9-7
907.12 POSTING HISTORY ................................................................................................................................ 9-7

908 GENERAL ................................................................................................................................................... 9-7
909 UNIT WEIGHTS & DENSITIES ...................................................................................................................... 9-8
910 STRUCTURES EXEMPT FROM LOAD RATING ........................................................................................... 9-8
911 WHICH PORTION OF BRIDGES SHALL BE LOAD RATED ........................................................................ 9-8
912 PROCEDURE FOR RATING ........................................................................................................................ 9-8
913 WHEN LOAD RATING SHALL BE REVISED ............................................................................................. 9-9
914 ANALYSIS OF BRIDGES WITH SIDEWALKS ............................................................................................ 9-9
915 ANALYSIS OF MULTILANE LOADING ......................................................................................................... 9-10
916 ANALYSIS FOR SPECIAL LOAD OR SUPERLOAD ..................................................................................... 9-10
917 LOAD RATING OF LONG SPAN BRIDGES .................................................................................................. 9-10

917.1 WHEN THE LOAD RATING SHALL BE DONE .......................................................................................... 9-10
917.2 HOW THE LOAD RATING SHALL BE DONE .......................................................................................... 9-10
917.3.1 INVENTORY & OPERATING LEVEL RATING USING HL-93 LOADING ........................................... 9-10
917.3.2 INVENTORY & OPERATING LEVEL RATING USING HS20 TRUCK .............................................. 9-10
917.3.3 OPERATING LEVEL RATING USING OHIO LEGAL LOADS ............................................................. 9-11
917.2.3.1 BRIDGES WITH THREE OR MORE LANES .................................................................................. 9-11
917.2.3.2 BRIDGES WITH TWO LANES ....................................................................................................... 9-12
917.2.3.3 BRIDGES WITH A SINGLE LANE .................................................................................................. 9-12

918 BRIDGE POSTING FOR REDUCED LOAD LIMITS ......................................................................................... 9-13

918.1 PURPOSE ................................................................................................................................................ 9-13
918.2 REFERENCE .......................................................................................................................................... 9-13
918.3 PROCEDURE FOR POSTING .................................................................................................................. 9-13
918.4 PROCEDURE FOR RESCINDING POSTING ......................................................................................... 9-14
918.5 PROCEDURE FOR CHANGING POSTING ............................................................................................. 9-15
918.6 REQUIRED INFORMATION FOR POST, RESCIND AND CHANGE REQUESTS ............................. 9-15

919 SOFTWARE TO BE USED FOR LOAD RATING ......................................................................................... 9-16

920 LOAD RATING REPORT SUBMISSION ....................................................................................................... 9-16

921 LOAD ANALYSIS USING AASHTO VIRTIS PROGRAM .............................................................................. 9-17

921.1 GENERAL ............................................................................................................................................... 9-17
921.2 VIRTIS LOAD RATING REPORT SUBMISSION ....................................................................................... 9-17
921.3 VIRTIS COMPUTER INPUT AND OUTPUT FILES .................................................................................. 9-18

922 LOAD ANALYSIS USING BRASS-CULVERT PROGRAM .............................................................................. 9-18

922.1 GENERAL ............................................................................................................................................... 9-18
922.2 BRASS CAPABILITIES ........................................................................................................................... 9-18
922.3 BRASS COMPUTER INPUT AND OUTPUT FILES .................................................................................. 9-19

923 LOAD RATING ANALYSIS USING BARS-PC ................................................................................................. 9-19
923.1 GENERAL........................................................................................................... 9-19
923.2 BARS-PC ANALYSIS – GENERAL GUIDELINES...................................................... 9-19
923.3 BARS-PC LOAD RATING REPORT SUBMISSION.................................................... 9-21
923.4 BARS-PC COMPUTER INPUT AND OUTPUT FILES............................................ 9-21
924 LOAD RATING OF BRIDGES USING LRFR SPECIFICATIONS.................................. 9-21
924.1 APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS................................... 9-21
924.2 GENERAL LOAD RATING EQUATION....................................................................... 9-21
924.3 LIMIT STATES AND LOAD FACTORS FOR LOAD RATING...................................... 9-22
924.4 DYNAMIC LOAD ALLOWANCE (IM)........................................................................ 9-24
924.5 CONDITION FACTOR (φc)....................................................................................... 9-25
924.6 SYSTEM FACTOR (φs)............................................................................................ 9-25
924.7 RESISTANCE FACTOR (φr).................................................................................... 9-25
924.8 EFFECT OF SKEW................................................................................................. 9-26
925 LOAD RATING OF NEW BRIDGES............................................................................ 9-26
925.1 LOADS TO BE USED FOR LOAD RATING................................................................. 9-26
925.2 LOAD RATING OF NEW BURIED BRIDGES............................................................ 9-27
925.2.1 CAST-IN-PLACE CONCRETE BOX & FRAME STRUCTURES.............................. 9-27
925.2.2 PRECAST CONCRETE BOXES............................................................................. 9-27
925.2.2.1 PRECAST CONCRETE BOXES OF SPAN GREATER THAN 12-FT.................. 9-27
925.2.2.2 PRECAST BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT...................... 9-27
925.2.3 PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES... 9-28
925.2.4 ANALYSIS OF CONCRETE BOX SECTIONS & FRAMES.................................... 9-28
925.3 LOAD RATING OF NON-BURIED STRUCTURES..................................................... 9-28
925.3.1 GENERAL......................................................................................................... 9-28
925.3.2 HOW THE LOAD RATING SHALL BE DONE....................................................... 9-28
925.3.3 WHEN THE BRIDGE LOAD RATING SHALL BE DONE....................................... 9-29
925.3.3.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS 9-29
925.3.3.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS........ 9-29
925.3.3.3 BRIDGES DESIGNED UNDER MINOR DESIGN-BUILD PROCESS.................. 9-29
925.3.3.4 BRIDGES DESIGNED UNDER MINIMAL DESIGN-BUILD PROCESS.............. 9-30
925.3.3.5 BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL........ 9-30
926 LOAD RATING OF EXISTING BRIDGES................................................................. 9-30
926.1 APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS................................... 9-30
926.2 LOADS TO BE USED FOR LOAD RATING................................................................. 9-30
926.3 LOAD RATING OF BRIDGES TO BE REHABILITATED............................................ 9-31
926.3.1 HOW THE LOAD RATING SHALL BE DONE....................................................... 9-31
926.3.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE....................................... 9-32
926.3.2.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS 9-32
926.3.2.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS........ 9-32
926.3.2.3 BRIDGES DESIGNED UNDER MINOR DESIGN-BUILD PROCESS.................. 9-33
926.3.2.4 BRIDGES DESIGNED UNDER MINIMAL DESIGN-BUILD PROCESS.............. 9-33
926.3.2.5 BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL........ 9-33
926.3.3 LOAD RATING OF BURIED BRIDGES................................................................. 9-33
926.3.3.1 CAST-IN-PLACE STRUCTURES......................................................................... 9-33
926.3.3.2 PRECAST BOXES OF SPAN GREATER THAN 12-FT........................................ 9-33
926.3.3.3 PRECAST BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT......................... 9-33
926.3.3.4 PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES... 9-34
926.3.3.5 ANALYSIS OF CONCRETE BOX SECTIONS & FRAMES.................................... 9-34
926.3.4 LOAD RATING OF NON-BURIED STRUCTURES.................................................. 9-34
926.4 LOAD RATING OF EXISTING BRIDGES WITH NO REPAIR PLANS....................... 9-34
926.4.1 HOW THE LOAD RATING SHALL BE DONE....................................................... 9-34
926.4.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE....................................... 9-35
926.4.3 LOAD RATING OF EXISTING BURIED BRIDGES................................................. 9-35
926.4.4 LOAD RATING OF NON-BURIED STRUCTURES.................................................. 9-35
(THIS PAGE INTENTIONALLY LEFT BLANK)
# APPENDIX – MISC. BRIDGE INFORMATION

## APPENDIX PURPOSE

1. ACCREDITATION PROCEDURE FOR MSE WALLS ................................................................. 2
2. 3 COAT SHOP PAINT SYSTEM IZEU .................................................................................. 38
3. STEEL POT BEARINGS ....................................................................................................... 56
4. METALLIC COATING SYSTEM FOR SHOP APPLICATION .................................................. 70
5. GALVANIZED COATING SYSTEM FOR STRUCTURAL STEEL BRIDGES ....................... 82
6. FINGER JOINTS FOR BRIDGES .......................................................................................... 91
7. HIGH PERFORMANCE STEEL (GRADE 70) ....................................................................... 95
8. CORRUGATED STEEL BRIDGE FLOORING ................................................................. 97
9. CLASS S CONCRETE USING SHRINKAGE COMPENSATING CEMENT ......................... 99
10. RETIRED NOTE 13 ............................................................................................................. 102
11. RETIRED NOTE 32 ............................................................................................................. 102
12. RETIRED NOTE 33 ............................................................................................................. 102
13. RETIRED NOTE 40A ........................................................................................................... 103
14. RETIRED NOTE 44A ........................................................................................................... 104
15. RETIRED NOTE 49 ............................................................................................................. 104
16. RETIRED NOTE 50 ............................................................................................................. 105
17. RETIRED NOTE 50A ......................................................................................................... 105
18. RETIRED NOTE 52 ............................................................................................................. 106
19. RETIRED NOTE 53 ............................................................................................................. 106
20. RETIRED NOTE 67 ............................................................................................................. 106
21. RETIRED NOTE 68 ............................................................................................................. 107
22. RETIRED NOTE 73 ............................................................................................................. 107
23. RETIRED NOTE 74 ............................................................................................................. 107
24. RETIRED NOTE 78 ............................................................................................................. 108
25. RETIRED NOTE 84 ............................................................................................................. 108
26. RETIRED NOTE 85 ............................................................................................................. 109
27. RETIRED NOTE 86 ............................................................................................................. 109
28. RETIRED NOTE 28 ............................................................................................................. 109
29. 6 mm (1/4") EPOXY WATERPROOFING OVERLAY FOR BRIDGE DECKS .................... 109.2
30. CONCRETE REPAIR USING PREPLACED AGGREGATE CONCRETE ......................... 109.8
31. STRUCTURAL SURVEY AND MONITORING OF VIBRATIONS .................................. 109.12
32. RETIRED NOTE 17 ........................................................................................................... 109.13
33. RETIRED NOTE 81 .......................................................................................................... 109.14
34. RATING OF BRIDGES AND POSTED LOADS ............................................................... 110

**APPENDIX-i**
SECTION 100 – GENERAL INFORMATION

101 INTRODUCTION

101.1 GENERAL CONSIDERATIONS

Unless otherwise noted, the design values, policies, practices, etc. that are established in this Manual are considered guidelines to promote uniform, safe and sound designs for bridges and structures in the State of Ohio. Deviation from these guidelines does not require formal approval from the Department; however, during the normal staged review process, the appointing authority may require the design agency to justify or otherwise seek recommendation from the Office of Structural Engineering when deviation is necessary.

The user of this Manual should be fully familiar with the AASHTO Standard Design Specifications For Highway Bridges including all issued Interim Specifications, the ODOT Construction and Material Specifications, and Office of Structural Engineering Standard Drawings and Design Data Sheets, along with the contents of this Manual.

The practicability of construction should be considered with reference to each detail of design. This applies particularly as new ideas are considered.

Where complete description or instruction is not provided in the Construction and Material Specifications, the description or instruction should be shown on the plans, but care should be taken to insure clarity both from a structural and contractual viewpoint.

101.2 TABLE OF ORGANIZATION

An organizational chart for the various sections in the Office of Structural Engineering and a list of bridge contacts is available on our website at http://www.dot.state.oh.us/se/.

102 PREPARATION OF PLANS

Drawings should be so planned that all details will fall within the prescribed borderlines. All detail views should be carefully drawn to a scale large enough to be easily read when reduced to half size. Views should not be crowded on the sheet.

The scale of the views on the drawings should not be stated because in making reproductions of the drawing the prints may be either the same size as the drawing or half-size.

A North Arrow symbol should be placed on the Site Plan, General Plan and all plan views.

Elevation views of piers and the forward abutment should be shown looking forward along the
stationing of the project. The rear abutment should be viewed in the reverse direction. Rear and forward abutments should be detailed on separate plan sheets for staged construction projects or for other geometric conditions that produce asymmetry between abutments.

When describing directions or locations of various elements of a highway project, the centerline of construction (survey) and stationing should be used as a basis for these directions and locations. Elements are located either left or right of the centerline and to the rear and forward with respect to station progression. [e.g. rear abutment; forward pier; left side; right railing; left forward corner] Sheets in the bridge plans should be numbered in accordance with Figure 109.

For each substructure unit, the skew angle should be shown with respect to the centerline of construction or, for curved structures, to a reference chord. The skew angle is the angle of deviation of the substructure unit from perpendicular to the centerline of construction or reference chord. The angle shall be measured from the centerline of construction or reference chord to a line perpendicular to the centerline of the substructure unit or from a line perpendicular to the centerline of construction or reference chord to the centerline of the substructure unit.

In placing dimensions on the drawings, sufficient overall dimensions will be given so that it will not be necessary for a person reading the drawings to add up dimensions in order to determine the length, width or height of an abutment, pier or other element of a structure.

In general, the designer should avoid showing a detail or dimension in more than one place on the plans. Such duplication is usually unnecessary and always increases the risk of errors, particularly where revisions are made at a later date.

If, because of lack of space on a particular sheet, it is necessary to place a view or a section on another sheet, both sheets should be clearly cross-referenced.

Abbreviation of words generally should be avoided. Abbreviations, unless they are in common use, may cause delay and uncertainty in interpreting the drawings. If abbreviations are used, a legend should be provided to explain the abbreviation.

Plan sheet size to be used is 22" x 34" [559 mm x 864 mm]. Margins shall be 2" [50 mm] on the left edge and 1/2" [15 to 20 mm] on all other edges.

Where a project includes more than one bridge, plan preparation economies may be obtained by coordination of the individual plans. Where general notes are numerous and extensive, time can be saved by using a sheet of notes common to all bridges, or by including all of the common notes on one bridge plan and referring to them on the other bridge plans. The same applies to common details.
102.1 BRIDGE DESIGN, CHECK AND REVIEW REQUIREMENTS

The Department requires bridge design computations and bridge plans be made and prepared by an experienced bridge design engineer, the designer; all bridge computations be independently verified by an experienced engineer, the checker; and all bridge plans be reviewed by an experienced engineer, the reviewer. The design agency shall perform the required checks and reviews prior to submitting prints to the Department for review.

All outside agencies performing bridge design work for the Department shall be pre-qualified according to the requirements contained in the Departmental document “Consultant Prequalification Requirements and Procedures” which is on file with the Office of Contracts. Those individuals upon whose experience the classification level of the design agency is based shall complete work. The initials of these same individuals shall be placed in the appropriate spaces in the title block signifying that they performed the work.

The designer shall be responsible for preparing a design that follows sound engineering practice and conforms to AASHTO, ODOT and other specifications and manuals. The designer shall also be responsible for preparing an accurate and complete set of final bridge construction plans.

The checker shall be responsible for ensuring correctness, constructability and completeness of the plans and calculations and adherence to pertinent specifications and manuals. The checker shall perform and prepare a set of separate, independent calculations verifying all stations, dimensions, elevations and estimated quantities.

The checker shall independently check all structural calculations to assure that the structural theory, design formulae and mathematics used by the designer are correct. The intent is not to produce two separate sets of structural calculations. However, for atypical designs, fracture critical components, and situations where the designer’s theory is unclear or questionable, the checker shall perform and prepare a set of separate, independent calculations. The checker and designer shall resolve all discrepancies and the final product shall reflect mutual agreement that the design is correct.

The checker shall verify all structural calculations performed by computer analysis by preparing independent input for comparison with the designer’s input. The checker shall perform an independent analysis of the output and agree with the designer on the final design.

The design agency’s reviewer is responsible for the overall evaluation of the plans for completeness, consistency, continuity, constructability, general design logic and quality.

Design and check computations shall be kept neat and orderly so they may be easily followed and understood by a person other than the preparer.
102.2 MANUAL DRAFTING STANDARDS

102.2.1 GENERAL

A. All lines and lettering shall be dark and opaque. All lines and lettering shall be on the front face of the drawing, whether original or reproduced.
B. Plan sheets submitted to the Department shall be of extremely good quality on reproducible mylar.

102.2.2 LETTERING STANDARDS

A. All lettering shall be Braddock No. 5 size (upper case 5/32" [4 mm] in height), or larger.
B. Lettering within lined areas, such as quantity box, should at no time come in contact with any of these lines.
C. Letters should be properly spaced so that a crowded condition does not exist.

102.2.3 MANUAL DRAFTING LINE STANDARDS

A. "0" (Rapidograph pen size) (decimal width of 0.4 mm) is minimum and can only be used for dimension lines, X-hatching and index map.
B. All other lines and lettering shall be a minimum of "1" (Rapidograph pen size) (decimal width of 0.5 mm).
C. Individual lines shall be of uniform weight and density.
D. 1/16" [1.5 mm] is the minimum distance between two or more adjacent lines, even though an out of scale condition might exist.

102.3 COMPUTER AIDED DRAFTING STANDARDS

Refer to the Ohio Department of Transportation CADD – Engineering Standards for all CADD drafting standards.

102.4 DESIGNER, CHECKER, REVIEWER INITIALS BLOCK

The design agency's designer, checker and reviewer's initials and the date of the final review shall be shown in the title block of each sheet.

102.5 TITLE BLOCK

See Figure 109 for example title blocks for 22" x 34" [559 mm x 864 mm] sheets.

Straight Line Mileage (SLM) shall be shown to the nearest 1/100 of a mile. (Example: MER-707-16.92)

Straight Line Kilometers (SLK) shall be shown to the nearest 1/1000 of a kilometer (nearest 10 meters). (Example: MER-707-27.310)
A bridge number is the SLM of the structure written without the decimal point. (Example: MER-707-1692)
A bridge number is SLK of the structure written without the decimal point. (Example: MER-707-27310)

A Station is defined as 100 feet. Stations shall be shown to the nearest 1/100 of a foot. (Example: Sta 895+08.75)

A Station is a kilometer. Roadway stations are shown to the nearest 1/1000 of a meter. (Example: Sta 8+282.273)

The correct Structure File Number (SFN) shall be shown in the Existing Structure Block and Title blocks for the existing and proposed structures respectively. The SFN for the existing structure should be included in the Scope of Services. If not, it may be obtained from the responsible office.

If a new SFN is required for a proposed structure, contact the appropriate office as follows:

A. Contact the Office of Structural Engineering, Inventory Section, for structures on the State System or special systems statewide.
B. Contact County Bridge Engineers for SFN’s to structures within their jurisdiction.
C. Contact the responsible District Office for SFN’s to structures in municipalities within the District’s jurisdiction.

It is the Designer’s responsibility to contact and confirm the correct SFN with the appropriate office. For more information on the Structure File Number, refer to the Structure Inventory website at http://www.dot.state.oh.us/sfn/inventory.

Provide the Project Identification Number (PID) below the project County, Route and Section number in the Title Block.

The wording “State of Ohio, Department of Transportation”, “Office of Structural Engineering” is used on plans prepared by the Office of Structural Engineering. The name and address of the consulting firm should replace “State of Ohio, Department of Transportation, Office of Structural Engineering” in the title block.

**102.6 ESTIMATED QUANTITIES**

Plan quantities shall be listed separately for each bridge structure. Incorporating common bid items between multiple bridge structures in a project is not acceptable. Summation of common items cannot be done due to computer tracking of quantities based on Structural File Number.

In order to avoid the re-calculating of pay quantities by the construction forces, a copy of the quantity calculations made, as the basis for the quantities shown on the plans, shall be furnished to the District Production Office. Therefore, it is important that the quantity calculations be
accurate and complete and prepared neatly on standard computation sheets. They should be arranged in an orderly fashion so that a person examining them will be able to follow the calculation sequence. Someone other than the original designer should independently check the calculations. The designer and checker shall each prepare separate sets of figures to minimize risk of error. The two sets of calculations then shall be reconciled, and one set (either the designer's or the checker's) shall be selected as the official set. The calculations shall be initialed and dated on each sheet by both the designer and checker. The results of this official set shall correspond to the quantities shown on the plans.

Each sheet of computations, notes, estimated quantities and steel list shall be marked with the Bridge Number, Structure File Number, the date, the writer's initials, and subject. Sheets that are accidentally misplaced are sometimes very difficult to identify if they are unmarked.

102.7 STANDARD BRIDGE DRAWINGS

Current standard bridge drawings should be followed and used whenever practicable. Reference to standard bridge drawings should be made by stating the Drawing Number and latest date of revision, or the approval date if there has been no revision. If reference is made to a standard drawing, details shown on such standard drawing generally should not be duplicated on the project plans. The designer shall be familiar with the standards and know if they are adequate for the particular design situation being addressed. If they are not, then standards shall be modified as necessary by supplying pertinent details, dimensions or material specifications in the plans.

The designer shall assure that standard drawings referenced on the General Notes sheet are also transferred to the Project Plans Title Sheet.

A standard drawing should not be referenced if only one or two small details on the standard are applicable. Details should be copied on the project plans. In general the call out of more than one standard drawing for a particular bridge component should be avoided.

Where possible, English standard bridge drawings should be referenced to English design plans and vice versa. Refer to Section 600 for unit conversion information and related plan note. Design Data drawings are not Standard Drawings and should not be referenced in the plans.

The quantities if shown on standard drawings are based upon average conditions and are only approximate. The quantities shall be computed from the actual plan dimensions.

In accordance with ODOT Policy 16-004(P), effective July 1, 2005, Standard Drawings will no longer be available for purchase through the Department’s Office of Contracts. Standard Drawings can be downloaded through the Office of Structural Engineering’s web page:

http://www.dot.state.oh.us/se/
102.8 SUPPLEMENTAL SPECIFICATIONS

The Department has many Supplemental Specifications that the designer needs to be familiar with and should use rather than developing their own individual specifications in the form of plan notes or Special bid items.

Supplemental Specifications may be obtained on the internet by accessing the Design Reference Resource Center (DRRC) home page from the Department website. The designer shall not modify Supplemental Specifications.

Supplemental Specifications, like standard bridge drawings are to be listed on the General Notes plan sheet. (See Section 600). The designer shall assure that standard drawings referenced on the General Notes sheet are also transferred to the Project Plans Title Sheet.

102.9 PROPOSAL NOTES

Proposal notes are similar to Supplemental Specifications. The Department’s numbered Proposal Notes were developed to assure uniform specifications for specific items that may not be required on every project or are either experimental or of an interim status. Proposal Notes, like Supplemental Specifications, are not to be revised by the designer.

The designer not only needs to know what bid item the Proposal Note applies to but also understand the Proposal Note so it is only applied where applicable.

Proposal Notes can also be obtained at the DRRC from the Department website. Proposal Notes are referenced on the bridge plan by adding a note to the end of the applicable bid item (See Proposal Note). If multiple Proposal Notes are being used with different bridge plan bid items a footnote method may be used at the end of each bid item with the footnote saying - “See Proposal Note”. Do not refer to Proposal Notes by number.

103 COMPUTER PROGRAMS

The following is a list of computer programs used by the Department. The Design Agency may want to consider these programs or other programs not listed. The Design Agency is responsible for obtaining any programs. It is the choice of the Design Agency as to which computer programs it uses.

Note: (MF) denotes a mainframe program
(PC) denotes a personal computer program
103.1 GEOMETRIC PROGRAMS

A. COGO - Coordinate Geometry (MF)
B. GEOPAK COGO - Roadway Design Software (PC)

103.2 DESIGN PROGRAMS

A. GAD - Girder Automated Design (MF)
B. BDS - Bridge Design Systems (MF) & (PC)
C. BRASS - Bridge Rating and Analysis of Structural Systems (MF) & (PC)
D. BOXCAR - Box Culvert Structural Analysis (PC)
E. MERLIN DASH - Beam and Girder Analysis and Design (PC)
F. DESCUS I - Curved Girder (PC)
G. CONSPAN – Prestressed Concrete Beams (PC)
H. PCA COLUMN - concrete column design (PC)
I. SIMON SYSTEMS (PC)
J. RISA3D - Structural Analysis (PC)
K. VANCK - curved steel bridge structures (PC)
L. RC-PIER - Concrete Substructure Analysis and Design (PC)
M. OPIS - Beam and Girder Analysis and Design (PC)
N. PSBEAM – Prestressed Concrete Beams (PC)

103.3 HYDRAULIC ENGINEERING PROGRAMS

A. HEC-2 or HEC-RAS - Computations of Water Surface Profiles in Open Channels (PC)
B. HY7 - (WSPRO) Water Surface Profiles (PC)
C. HY8 - Culvert Hydraulics (PC)
D. HEC-12 - Pavement Drainage (PC)
E. HYDRA V3.2 (PC)
   1. Universal Culvert
   2. Special Culvert
   3. Long Span Culvert
   4. Storm Sewer Design
   5. Inlet Spacing
   6. Ditch Analysis

103.4 GEOTECHNICAL ENGINEERING PROGRAMS

A. PICAP - Pile Capacity (PC)
B. SHAFT - Drilled Shafts (PC)
C. COM624P - Lateral Loading of Piles and Drilled Shafts (PC)
D. WEAP - Wave Equation Analysis of Pile Driving (PC)
E. STABL - Slope Stability Analysis (PC)
F. SPW911 - Sheet Pile Design and Analysis (PC)
G. Driven - Pile Capacity (PC)

103.5 BRIDGE RATING PROGRAM

Refer to Section 900 for Bridge Rating Programs.

104 OHIO REVISED CODE SUBMITTALS

The Ohio Revised Code has been changed so that Section 5543.02 no longer requires county financed bridge projects to be submitted to the Department for approval. The Code does require the Department to review and comment on the plans for conformance with State and Federal requirements if requested to do so by the County.

105 BRIDGE PLAN SHEET ORDER

A set of completed bridge plans should conform to the following order:

A. Site Plan
B. General Plan & General Notes
C. Estimated Quantities Phase Construction Details
D. Abutments
E. Piers
F. Superstructure
G. Railing Details
H. Expansion device details
I. Non-standard approach slab details
J. Reinforcing Steel List

The General Plan sheet no longer requires an elevation view. The General Plan sheet is only required for:

A. Deck overlay projects
B. Deck replacement projects where the bridge deck is variable width or curved.
C. New bridge of variable width or curved alignment.
D. New or rehabilitated structure requiring staged construction

If no General Plan sheet is furnished, the bridge plans may require a line diagram to show stationing and bridge layout dimensions that would not be practical to show on the site plan due to the site plan’s scale. Other details may be required to adequately present information needed to construct the bridge.
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-17-03)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
(THIS PAGE INTENTIONALLY LEFT BLANK)
STANDARD TITLE BLOCKS

TX=0.125 IN., WT=1 (CADD) OR NO. (0) RAPIDOGRAPH (HAND)

1"  1 1/2"  2"  3"  6 1/2"  2 1/2"  2"  3 1/2"

EXAMPLE SITE PLAN TITLE BLOCK

1"  1 1/2"  2"  3"  9"  2"  3 1/2"

EXAMPLE PLAN SHEET TITLE BLOCK

TX=0.125 IN., WT=1 (CADD) OR NO. (0) RAPIDOGRAPH (HAND)

TX=0.175 IN., WT=2 (CADD) OR NO. (1) RAPIDOGRAPH (HAND)

* THIS BLOCK SHOULD BE FILLED IN WITH THE NAME OF THE ACTUAL DESIGN AGENCY.
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-17-03)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
STANDARD TITLE BLOCKS

TX = 3.2 mm, WT = 1 (CADD) OR NO. (0) RAPIDOGRAPHT (HAND)

15 mm

25 mm

75 mm

165 mm

65 mm

25 mm

25 mm

50 mm

90 mm

MER-707-27.230
SITE PLAN
BRIDGE NO. MER-707-27310
OVER ST. MARYS RIVER

MERCE COUNTY
STA. 27+282.267
STA. 27+351.000

REZA
REZA
CHECKED
REVISED
LMW 12-22-92
STRUCTURAL FILE NUMBER 1234567

DESIGN AGENCY
OFFICE OF
STRUCTURAL ENGINEERING

EXAMPLE SITE PLAN TITLE BLOCK

TX = 3.2 mm, WT = 1 (CADD) OR NO. (0) RAPIDOGRAPHT (HAND)

TX = 4.4 mm, WT = 2 (CADD) OR NO. (1) RAPIDOGRAPHT (HAND)

EXAMPLE PLAN SHEET TITLE BLOCK

THIS BLOCK SHOULD BE FILLED IN WITH THE NAME OF THE ACTUAL DESIGN AGENCY.
SECTION 200 - PRELIMINARY DESIGN

201 STRUCTURE TYPE STUDY

201.1 GENERAL

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Structure Type Study shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. The Structure Type Study is not required for projects classified as Minimal.

The project site should be studied in detail and evaluated to determine the best structure alternative. A site visit should be made. In many cases, it can be readily determined whether a particular bridge or culvert should be chosen for a particular site. If a bridge is the most appropriate structure for a particular site, then the Structure Type Study needs to be performed to determine the appropriate bridge type.

201.2 STRUCTURE TYPE STUDY SUBMISSION REQUIREMENTS

A Structure Type Study submission should include the following:

A. Profile for each bridge alternative ................................................................. Section 201.2.1
B. Preliminary Structure Site Plan (for preferred bridge alternative) ............. Section 201.2.2
C. Hydraulic Report ......................................................................................... Section 201.2.3
D. Narrative of Bridge Alternatives ................................................................. Section 201.2.4
E. Cost Analysis .............................................................................................. Section 201.2.5
F. Foundation Recommendation ................................................................. Section 201.2.6
G. Preliminary Maintenance of Traffic Plan .................................................. Section 201.2.7

The Structure Type Study shall be included in the review submission made directly to the District Office. A concurrent review submission shall be made to the Office of Structural Engineering if the proposed structure type contains non-standard bridge railing types, non-redundant designs, or fracture critical designs. The Office of Structural Engineering will forward review comments for these items to the responsible District Office.

Additional structural related items that are required at this stage of the review process include:

A. Retaining Wall Justification (L&D Section 1404.2)
B. Noise Wall Justification (ODOT Policy #21-001 and Procedure #417-001)
C. Pedestrian Overpass Justification (L&D Section 1404.4)
201.2.1 PROFILE FOR EACH BRIDGE ALTERNATIVE

The profile for each bridge alternative considered shall generally be drawn to a scale of 1” = 20’ [1 to 200] and shall generally be taken along the proposed centerline of survey for the full length of the bridge. The profiles shall include: the existing and proposed profile grade lines; existing ground line; the cross-section of channel; an outline of the structure; highest known high water mark; normal water elevation; Ordinary High Water Mark (OHWM); flow line elevation (thalweg); design and 100 year highwater elevations including backwater; overtopping flood elevation and frequency; existing and proposed profile grade elevations at 25 ft [10 m] increments; and minimum and required vertical and horizontal clearances. Note: normal water elevation is the water elevation in the stream which has not been affected by a recent heavy rain runoff and could be found in the stream most of the year. Refer to BDM Section 203.4 for OHWM definition.

201.2.2 PRELIMINARY STRUCTURE SITE PLAN

The Site Plan scale generally should be 1” = 20’ [1 to 200]. For some cases to get the entire bridge on one sheet a smaller scale may be provided, if all details can be clearly shown. For bridges where the 1” = 20’ [1 to 200] scale is too small to clearly show the Site Plan details, a 1” = 10’ [1 to 100] scale may be considered. The following general information should be shown on the Preliminary Structure Site Plan:

A. The plan view should show the existing structures (use dashed lines); contours at 2 foot [0.5 meter] intervals showing the existing surface of the ground (for steep slopes contours at 5 foot [2.0 meter] or greater intervals may be used); existing utility lines and their disposition; proposed structure; proposed temporary bridge; proposed channel improvements; a north arrow; and other pertinent features concerning the existing topography and proposed work in an assembled form.

In case of a highway grade separation or a highway-railway grade separation, the required minimum and actual minimum horizontal and vertical clearances and their locations shall be shown in the plan and profile views.

For a bridge over a railway, the vertical clearance shall be measured from a point level with the top of the highest rail and 6 feet [2 meters] from the centerline of those tracks, or greater if specified by the individual railroad. Reference shall be made to Chapter 15, Section 1.2.6(a), AREMA Specifications for increased lateral clearances required when tracks are on a horizontal curve.

B. A profile as described in Section 201.2.1. The profile scale shall be the same as the plan view.

C. Horizontal and vertical curve data.

D. Size of drainage area. The elevation, discharge and stream velocity through the structure and the backwater elevation for the 100-year frequency base flood, the design year flood and if necessary the overtopping flood. The clearance from the lowest elevation of the bottom of the superstructure to the design year water surface elevation (freeboard) should be provided.
E. In the existing structure block, provide a brief description of existing bridge. This should include type, length of spans and how measured (c/c of bearings, f/f of abutments), roadway width (t/t of barrier, t/t of curb, or f/f of railing), skew angle, original design loading or upgraded loading, type of deck and type of substructure, date when built, Structure File Number (SFN), approach slabs and wearing surface.

F. In the proposed structure block provide a brief description of proposed bridge. This should include type, length of spans and how measured (c/c of bearings), roadway width (t/t of barrier, t/t of curb, or f/f of railing), width of sidewalks, design loading, future wearing surface loading, skew angle, wearing surface, approach slabs, alignment, superelevation or crown and latitude and longitude bridge coordinates.

G. A cross section of the proposed superstructure, including an elevation of the proposed pier type(s) if applicable.

H. The design and current average daily traffic (ADT) and the design average daily truck traffic (ADTT).

I. For each substructure unit where a bearing is to be used, the bearing condition (fixed or expansion) shall be designated in the profile view (FIX or EXP). Semi-integral substructures shall be designated as expansion (EXP) and integral shall be designated as integral (INT).

J. Horizontal and vertical clearances and their locations shall be provided for navigable waterway crossings.

K. A cross section sketch at the abutments shall be submitted to provide information to help verify bridge limits.
201.2.3 HYDRAULIC REPORT

The Structure Type Study shall include a Hydraulic Report that includes the following information:

A. Supplemental Site Plan showing information necessary for the determination of the waterway opening. Information shown on the Supplemental Site Plan should not be repeated on the Structure Site Plan. The following information should be included on the Supplemental Site Plan:

1. A small scale area plan showing: approximate location of all stream cross sections used for the hydraulic analysis; an accurate waterway alignment at least 500 feet [150 meters] each way from the structure; and the alignment of the proposed and present highways, taken from actual surveys. Note location of dams or other regulatory work on the waterway above the site, and the pool level, if the bridge is in a pool area above a dam.

2. A stream profile at least 500 feet [150 meters] each way from the bridge showing waterway flow line elevations and low water profile (where materially different) and high water profile if such is obtainable. If a high water profile cannot be obtained, high water elevations, with their locations marked or described, should be shown both above and below the bridge. Show high water elevations with dates and location of reading with relation to the existing structure. The source of high water data should be noted on the Supplemental Site Plan. High water data should preferably be collected from at least two locations and preferably verified by interviewing two local residents.

3. A profile along the centerline of highway so that the overflow section may be computed. This profile should extend along the approach fill to an elevation well above high water. If
there are bridges or large culverts located within 1000 feet [300 meters] upstream or down-
stream from the proposed bridge, show stream cross sections including the structure and road-
way profiles of the overflow sections of the structures. These may be used as a guide in estab-
lishing the waterway requirements of the proposed structure.

4. The nature of the waterway should be described as to condition of channel, banks, drift, ice
conditions, flow of channel during low or high water, etc.

5. In the areas where agricultural or other drainage improvements are proposed, local
authorities should be consulted as to the nature of the improvement and the probability of
future lowering of the flow line, and appropriate provisions should be made.

B. Hydraulic Analysis that includes:

1. Drainage area determination in mi² [km²].

2. One set of computations for the design year and 100 year frequency discharges, including
calculations used for the watershed’s main channel slope.

3. Stream cross-section plots representing the natural stream conditions. The cross-section plots
should indicate the Manning’s “n” values chosen for the channel sub-sections.

4. One set of color photographs of the upstream channel, downstream channel and bridge
opening should be included as assistance in determining the Manning’s “n” values.

5. A stream cross-section at the existing bridge, including the bridge structure, for bridge
replacement projects. All computer input shall be substantiated by existing ground contours
or additional cross sections.

6. Hydraulic calculations for the computation of backwater and mean velocities at the proposed
bridge for both the design year and 100 year frequency discharges. All computer input data
should be provided on a disk and included with the submission. HEC-RAS submissions
shall conform to the file structure shown on Figure 207.

   a. For bridge replacement projects, the computations should be made for both the existing
      and proposed bridges, using like analysis methods.

   b. If the proposed roadway is overtopped by a discharge less than the 100 year frequency
discharge, the elevation and approximate frequency of the overtopping discharge shall be
shown on the Preliminary Structure Site Plan.

C. Bridge Scour Analysis that includes: (See Section 203.3 for specific scour information)

1. A narrative of findings and recommended scour counter-measures. Include a statement
regarding the susceptibility of the stream banks and flow line to scour, and also the
susceptibility of the piers and abutments to scour.

2. A copy of the scour calculations (if necessary). These calculations shall be provided to the ODOT District Engineer responsible for bridge inspection.

D. Hydraulic Narrative that briefly discusses the hydraulic effects of the bridge. The narrative should include:

1. A discussion of the hydraulic adequacy for both the design year and 100 year frequency discharges.

2. High water data from local residents and observed high water marks including their locations.

3. A description of the bridge deck drainage collection system. Indicate how the surface water will be collected and discharged and include proposed scupper (if necessary) and catch basin locations. Refer to Section 209.3 for more information.

E. Flood Hazard Evaluation. This is a condition statement regarding the nature of the upstream area, the extent of upstream flooding and whether buildings are in the 100 year frequency flood plain.

201.2.4  NARRATIVE OF BRIDGE ALTERNATIVES

The Structure Type Study shall include a brief narrative identifying the structure alternatives and their costs. The narrative should provide insight into why the particular proposed structure was chosen. Factors that need to be considered in selecting a structure for a particular site include geometry, economics, maintainability, constructability, right-of-way constraints, disruption to the traveling public, waterway crossing requirements or grade separations requirements, clearances for railway and highway crossings, foundation considerations, historical and environmental concerns, debris and ice flow problems and appearance.

For rehabilitations, include color photographs of the portions of the existing structure to be salvaged. To substantiate the proposed salvage decision, all areas of rehabilitation shall be identified by field investigation.

For rehabilitation of steel beam or girder superstructures, include the Fatigue Evaluation defined in Section 400.

201.2.5  COST ANALYSIS

The Structure Type Study shall include a cost analysis comparing alternative structures shall be performed, unless the site conditions discourage the use of all but one type of structure. The cost
analysis should include the initial construction cost and all major future rehabilitation/maintenance costs, converted to present dollars. Sufficient preliminary design must be performed for an accurate cost estimate. Cost data information may be obtained from “Summary of Contracts Awarded”. This publication is available from the Office of Contracts.

When a rehabilitation alternate involves salvaging existing concrete members, cost overruns should be anticipated and included in the cost analysis. See Section 400 of this Manual for additional rehabilitation information.

201.2.6 FOUNDATION RECOMMENDATION

The Structure Type Study shall include a Foundation Recommendation that consists of:

A. General foundation type (i.e. Drilled Shafts, Friction Piles, Bearing Piles or Spread Footings)
B. Typed boring logs
C. Laboratory test results as follows:
   1. Soil: Water content, particle size analysis, liquid limit, plastic limit
   2. Rock: RQD

For the scour evaluation, Section 203.3(D), provide $D_{50}$ values from the particle size analysis.

When the foundation recommendation for the preferred alternative includes MSE wall supported abutments, the Designer shall provide estimates for bearing pressure and allowable bearing capacity for the in-situ material below the MSE wall and an estimate for settlement of the MSE wall. Refer to Section 204.4 for additional considerations.

When unique subsurface conditions arise, include a brief narrative in the Foundation Recommendation for justification to obtain extra soils borings.

201.2.7 PRELIMINARY MAINTENANCE OF TRAFFIC PLAN

The various components of the bridge stage construction shall match those of the approach roadway, and the nomenclature used to identify the various stages (phases) of construction shall be the same for the roadway and the bridge (Stage 1 and Stage 2 or Phase 1 and Phase 2).

The Preliminary Maintenance of Traffic Plan shall include a transverse section(s) defining all stages of removal and construction. The following information should be provided:

A. The existing superstructure and substructure layout with overall dimensions (field verified) and color photographs.
B. Type of temporary railing or barrier.
C. Proposed temporary lane widths, measured as the clear distance between temporary barriers, shall be shown. A temporary single lane width of 11’-0” [3350 mm] or greater is preferred; 10’-0” [3000 mm] is the minimum allowable. Minimum preferred lateral clearance from edge of lane to barrier is 1’-6” [500 mm] (ODOT’s Traffic Engineering Manual Section 640-2) but Section 605-11.2 of the Traffic Engineering Manual, allows this lateral distance to be amended for specific sites and conditions. The designer should ensure that lane and lateral clearance requirements are evaluated versus effects of phased construction on a bridge structure.

D. Location of cut lines. The existing structure should be evaluated to determine where the cut-line can be made to provide structural adequacy. Cut lines through stone substructures should be carefully evaluated to maintain structural integrity through staged removals. Temporary shoring may be required and should be considered.

E. Temporary modifications to superelevated sections (existing and/or proposed) on the deck and/or shoulder in order to accommodate the traffic from the phase construction.

F. Width of closure pour. When determining the closure pour width (see Section 300 of this Manual), the designer should investigate the economics of using the lap splices versus using mechanical connectors. Any necessary structure modifications should be included in the cost estimate. Lap splices are preferred and recommended. A reduced closure width may cause transition problems in the finishing of the bridge deck surface when bringing the various phases of construction together.

G. Profile grade, alignment, approximate location and width of temporary structures

H. Location of temporary shoring

201.3 UTILITIES

All utilities should be accurately located and identified on the Preliminary Structure Site Plan. A note should state whether they are to remain in place, be relocated or be removed, and for the latter two, by whom.

Utilities should not be placed on bridges whenever possible.

The type of superstructure selected for a site may be dependent upon the number of utilities supported on the bridge. The request to allow utilities on the bridge shall be made through the ODOT District Utilities Coordinator. Refer to the ODOT Utilities Manual. Utilities shall be installed in substantial ducts or enclosures adequate to protect the lines from future bridge repair and maintenance operations. Utilities shall not be placed inside of prestressed concrete box beams. For some specific detail issues with utilities on bridges refer to Section 300 of this Manual.
202  BRIDGE PRELIMINARY DESIGN REPORT

202.1  GENERAL

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Bridge Preliminary Design Report shall be included in the Stage 1 Detailed Design Review Submission.

202.2  BRIDGE PRELIMINARY DESIGN REPORT SUBMISSION REQUIREMENTS

The Bridge Preliminary Design Report submission should contain the following:

A. Final Structure Site Plan .................................................................................... Section 202.2.1
B. Final Maintenance of Traffic Plan ................................................................. Section 202.2.2
C. Foundation Report ............................................................................................. Section 202.2.3
D. Supplemental Site Plan for Railway Crossings ................................................. Section 202.2.4

202.2.1  FINAL STRUCTURE SITE PLAN

In addition to the Preliminary Structure Site Plan requirements of Section 201.2.2, the Final Structure Site Plan should show the following information in plan view: bridge width and approach pavement widths, showing curb or parapet lines and outer limits of the superstructure and substructure units; skew with respect to the centerline of a substructure unit (not to centerline of stream or centerline of tracks); lateral clearances (both the minimum required and the actual) with respect to railroad tracks or highways under the proposed structure; location of minimum vertical clearance; treatment of slopes around the ends and under the bridge; channel changes; soil boring locations; centerline of temporary structure and temporary approach pavement; stationing of bridge limits (i.e. the bridge ends of approach slabs); limits of channel excavation shown by crosshatching with a description provided in a legend; the location and description of bench marks; the following earthwork note: “EARTHWORK limits shown are approximate. Actual slopes shall conform to plan cross sections.”; and guardrail stationing.

When providing guardrail stationing: for bridges with twin steel tube bridge railing, station the first top mounted post on the bridge, for bridges with deep beam railing and concrete barrier railing, station the first guardrail post off the bridge. Typically stationing should be given to the nearest 1/100th of a foot [mm]. Guardrail stationing may be changed during the detail design phase and then revised on the Site Plan.

In addition to the requirements of Section 201.2.2, the Final Structure Site Plan should show the following information in the profile view: profile gradient percent; embankment slopes and top of slope elevations; proposed footing elevations; type of foundations; top of bedrock elevations at each boring location; and shaded areas of the bridge that represent the new bridge components.
For geometrical clarification: spans on straight (tangent) alignments should be measured from center to center of bearings along the centerline of survey; spans on curved alignments should be measured along a reference line, a chord drawn from centerline to centerline of abutment bearings at the centerline of survey or extended tangent along the centerline of survey; the centerline of the abutment bearings for concrete slab bridges is assumed 7½” [190 mm] behind the face of abutment substructure or breastwall; skews should be given with respect to the centerline for straight alignments or to reference lines (chords or tangents) for curved alignments; for straight alignments, the bearing of the centerline of survey should be shown; for curved alignments, the bearing of the reference line (chord or tangent) should be shown; and a superelevation transition table or diagram similar to Figure 206, should be provided if the bridge crown (superelevation) changes across the structure, reference should be made to the table or diagram when detailing the typical bridge transverse section.

Descriptive data for the proposed structure should be shown in a “Proposed Structure” block. The “Proposed Structure” block should be placed in the lower right hand corner for the 22” x 34” [559 x 864 mm] sheet size. An “Existing Structure” block should be shown on the Site Plan if applicable and be placed above the “Proposed Structure” block. Structure blocks should be approximately 6½” [165 mm] wide for 22” x 34” [559 x 864 mm] sheet size.

For rehabilitation projects, the first item in the “Proposed Structure” data block should be “Proposed Work” followed by a brief description of the type of work to be done (for example: new composite reinforced concrete deck, new superstructure on existing substructure, or new superstructure on widened substructure). Provide a relatively thorough description (list of work) of the type of work to be done within a plan note entitled “Proposed Work” and include this note on the Final Site Plan.

### 202.2.2 FINAL MAINTENANCE OF TRAFFIC PLAN

In addition to the Preliminary Maintenance of Traffic Plan requirements of Section 201.2.7, the Final Maintenance of Traffic Plan should include the following information:

A. Plan views and preliminary working drawings to ensure constructability
B. Temporary barrier anchorage details and requirements
C. Location and type of temporary shoring (See Section 208)
D. Location of structural members that require strengthening
E. Temporary structure design information (See Section 500)
F. Additional notes and/or details necessary

For concrete slabs, early standard drawings called for the main reinforcement to be placed perpendicular to the abutments when the skew angle became larger than a certain value. This angle has been revised over the years as new standard drawings were introduced. When considering staged construction requirements and the orientation of the cutline, screen existing concrete single span slab bridges according to the following criteria:
Prior to 1931 the slab bridge standard drawing required the main reinforcement to be placed perpendicular to the abutments when the skew angle was equal to or greater than 20 degrees. This angle was revised to 25 degrees in 1931, 30 degrees in 1933 and finally 35 degrees in 1946. The standard drawing in 1973 required the main reinforcement to be parallel with the centerline of roadway regardless of skew angle. Existing exposed reinforcing steel may be used to confirm the direction of the reinforcing steel.

If the skew angle of the bridge is equal to or greater than the angles listed above for the year built, a temporary longitudinal bent will have to be designed to support the slab where it is cut or if possible locate the cutline parallel to the reinforcing if sufficient room exists. For example a bridge built in 1938 with a 25 degree skew does not require a bent, however a bridge built in 1928 with a 25 degree skew does require a bent to be designed.

When utilizing semi-integral construction, the stability of the new part-width superstructure is to be considered. There exists the potential of the superstructure to move laterally either from the effects of the traffic using the new deck or the lateral earth pressure against the approach slab. See Standard Bridge Drawing “SEMI-INTEGRAL CONSTRUCTION DETAILS” for more information.

202.2.3 FOUNDATION REPORT

The Bridge Preliminary Design Report shall include a Foundation Report in accordance with the ODOT Specifications for Subsurface Investigations. The Foundation Report shall include:

A. Investigational Findings
B. Analyses and Recommendations
C. Boring Logs and Undisturbed Test Data

Where the scour evaluation has been identified a potential problem, the probable scour depths, calculated in accordance with Section 203.3(D), should be considered in the design of the substructures; the location of the bottom of footings; the minimum tip elevations for piles and drilled shafts; and the frictional capacity of piles and drilled shafts.

The Foundation Report for MSE wall supported abutments shall include calculations for bearing pressure and bearing capacity for the in-situ material below the MSE wall and calculations for MSE wall settlement. The report shall also consider the effect of settlement and include all construction constraints, such as soil improvement methods, that may be required.

Specific design considerations for each foundation type are presented in the following sections.

202.2.3.1 SPREAD FOOTINGS

The use of spread footings shall be based on an assessment of design loads, depth of suitable bearing materials, ease of construction, effects of flooding and scour analysis, liquefaction and swelling potential of the soils and frost depth. Generally the amount of predicted settlement of the spread footing and the tolerable movement of the structure control the type of footing. To establish tolerable movements, engineering judgment should be used (also refer to FHWA’s Manual on Tolerable Movements, Report No. FHWA/RD-85/107).
The allowable bearing pressure for the foundation soil is a function of the footing dimensions, depth of overburden and the location of the water table. Procedures for computing allowable bearing pressure for both cohesive and cohesionless soils are given in the FHWA Manual “Soils and Foundations Workshop Manual”, Publication No. FHWA-HI-88-009, July 1993. A relationship between Standard Penetration Test (SPT) value, N, and the soil parameters, angle of internal friction, and cohesive strength, c, is given in tables presented in chapter 6 of the FHWA manual. The cohesive strength of soil is taken as one half of the ultimate strength, q_u.

All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term. See Section 600 for notes and additional guidance.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. Preliminary design loads, the estimated size of the footing and the allowable bearing pressure shall be provided for review with the Foundation Report. This information is to be furnished by the design agency preparing the plans.

During the detail design stage, the actual footing size shall be determined based on the actual design loads. Note that the allowable bearing pressure may need to be adjusted for the actual footing size. A safety factor of three (3) shall be used to determine the allowable bearing pressure.

### 202.2.3.2 PILE FOUNDATIONS

The type, size and estimated length of the piles for each substructure unit shall be shown on the Final Structure Site Plan. Preliminary pile design loads and approximate pile spacings shall be provided with the Foundation Report. This information will be furnished by the design agency preparing the plans. The estimated pay length(s) for the piling shall be measured from the pile tip to the cutoff elevation in the pile cap and shall be rounded up to the nearest five (5) feet [one meter]. Procedures for computing estimated pay length of the piles are given in the FHWA’s “Design and Construction of Driven Pile Foundations, Vols. 1 & 2”, FHWA-HI-97-013/014. Minimum pile tip elevations for friction designed piles may be required and should be shown on the Final Structure Site Plan.

When installing piles at a batter, the site conditions should be studied to determine if installation is practical. Piles under 15 feet [5 meters] in length should not be battered.

### 202.2.3.2.a STEEL ‘H’ PILES

When piles are driven to refusal on the bedrock, steel ‘H’ piles are generally used. The commonly used pile sizes are:
**H Pile Size** | **Design Load** | **Ultimate Bearing Value**
--- | --- | ---
HP10X42 | 75 tons | 150 tons
HP12X53 | 95 tons | 190 tons
HP14X73 | 130 tons | 260 tons

**H Pile Size** | **Design Load** | **Ultimate Bearing Value**
--- | --- | ---
HP250X62 | 650 kN | 1300 kN
HP310X79 | 850 kN | 1700 kN
HP360X108 | 1150 kN | 2300 kN

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2). Design load values for H piles are based on a maximum service load stress of 12.5 ksi [86 MPa] for Grade 50 steel.

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the H pile size selected but the calculated Ultimate Bearing Value load of the substructure unit or units.

For the piers, other than capped pile piers, HP10X42 [HP250X62] should be used if the calculated design load is less than 75 tons [650 kN] per pile.

In order to protect the tips of the steel “H” piling, steel pile points shall be used when the piles are driven to refusal onto strong bedrock. When the depth of overburden is more than 50 feet [15 meters] and the soils are cohesive in nature, piles driven to strong bedrock generally should not have steel points. Steel points should not be used when the piles are driven to bear on shale.

For projects where steel points are to be used, include the plan note entitled “Item 507, Steel Points, As Per Plan” with the Structure General Notes (Section 600 of this Manual).

For capped pile piers with steel H piles, pile encasement is required.

**202.2.3.2.b CAST-IN-PLACE REINFORCED CONCRETE PILES**

For piles not driven to bear on the bedrock, cast-in-place reinforced concrete piles should be used. This type of pile achieves its design load resistance through a combination of side friction and end bearing. The commonly used pile sizes are:
### Pipe Pile Design Load Table

<table>
<thead>
<tr>
<th>Pipe Pile Diameter</th>
<th>Design Load</th>
<th>Ultimate Bearing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 inch</td>
<td>50 tons</td>
<td>100 tons</td>
</tr>
<tr>
<td>14 inch</td>
<td>70 tons</td>
<td>140 tons</td>
</tr>
<tr>
<td>16 inch</td>
<td>90 tons</td>
<td>180 tons</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pipe Pile Diameter</th>
<th>Design Load</th>
<th>Ultimate Bearing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>300 mm</td>
<td>450 kN</td>
<td>900 kN</td>
</tr>
<tr>
<td>350 mm</td>
<td>650 kN</td>
<td>1300 kN</td>
</tr>
<tr>
<td>400 mm</td>
<td>800 kN</td>
<td>1600 kN</td>
</tr>
</tbody>
</table>

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2). The design values for pipe piles are based on a maximum allowable service load stress on the pile wall thickness of roughly 10 ksi [69 MPa] for ASTM A 252 Grade 2 steel, $F_y = 35$ ksi [$F_y = 240$ MPa].

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the Pipe pile size selected but the calculated Ultimate Bearing Value load of the substructure unit or units.

For capped-pile piers with cast-in-place piles, 16 inch [400 mm] diameter piles shall be used. 16 inch [400 mm] diameter piles with additional reinforcing steel are preferred because the need for pile encasement is eliminated. Additional reinforcing steel which consists of 8 - #6 [19M] epoxy coated reinforcing bars with #4 [13M] spiral at 12 inch [300 mm] pitch should be provided for 16 inch [400 mm] diameter piles. Reinforcing steel shall be detailed on the plans, included in the reinforcing steel list, and be paid for under Item 507, 16 Inch [400 mm] Cast-In-Place Piles Furnished, As Per Plan. The reinforcing steel cage should extend 15 feet [5 meters] below the flow line and into the pier cap. Pile encasement is not used when additional reinforcement is provided. Painting of the cast-in-place reinforced concrete pile is not required.

For capped-pile piers where the exposed length of the piles is more than 20 feet [6 meters], 18 inch [450 mm] diameter piles can be used. Consult the Office of Structural Engineering before recommending the use of 18 inch [450 mm] diameter piles.

**202.2.3.2.c DOWN DRAG FORCES ON PILES**

When a significant height of new embankment is constructed over a compressible layer of soil and long term settlement is anticipated, the possibility of down drag forces on the piles should be considered. The extra load that the pile receives due to the down drag force should be computed and accounted for by driving the piles to a higher design load capacity. For example, the total design load for the piles should be equal to Dead Load + Live Load + Down Drag Force. See Section 600 of this Manual for note.
202.2.3.2.d  PILE WALL THICKNESS

Minimum pipe pile wall thicknesses are specified by formula in CMS 507, which makes use of the Ultimate Bearing Value.

202.2.3.2.e  PILE HAMMER SIZE

According to Item 507, the contractor will select a pile hammer large enough to achieve the required Ultimate Bearing Load and perform a dynamic load test to verify that this load is achieved. Refer to Section 303.4.2 for specific pile testing requirements.

202.2.3.2.f  CONSTRUCTION CONSTRAINTS

For construction constraints regarding pile installation and embankment construction, see Section 600 of this Manual.

202.2.3.2.g  PREBORED HOLES

The Designer shall specify prebored holes, CMS 507.11, for each pile on a project to be driven through 15 ft [4.5 m] or more of new embankment. Clearly indicate the locations and lengths of all prebored holes in the plans. For design purposes, ignore the effect of skin friction along the length of the prebored holes. The length shall be the height of the new embankment at each pile location.

202.2.3.3  DRILLED SHAFTS

The diameter of the drilled shafts for the abutments and piers shall be shown on the Final Structure Site Plan. For drilled shafts with friction type design, the tip elevation shall also be shown. For drilled shafts supported on bedrock, the tip elevation should not be given. Instead, the approximate top of the bedrock elevation and the length of the bedrock socket shall be shown in the profile view on the Final Structure Site Plan. This information shall be furnished by the design agency preparing the plans.


Drilled shafts should be considered when their provision would:

A. Prevent the need of cofferdams
B. Become economically viable due to high design loads (eliminates the need of large quantities of pile)

C. Provide protection against scour

D. Provide resistance against lateral and uplift loads

E. Accommodate sites where the depth to bedrock is too short for adequate pile embedment but too deep for spread footings

F. Accommodate the site concerns associated with pile driving process (vibrations, interference due to battered piles, etc.).

When drilled shafts with friction type design are used, a minimum of three (3) shafts per pier are recommended. Consult the Office of Structural Engineering during preliminary design for additional information.

When lateral loads are controlling the design of drilled shafts, consult the Office of Structural Engineering to determine if lateral load testing should be specified.

The Design Agency should have the Department review any special proposed drilled shaft plan notes during the Stage 1 Review Submission. If casing is to be specified as to be left in place, a plan note will need to be added.

202.2.4 SUPPLEMENTAL SITE PLAN FOR RAILWAY CROSSINGS

For Railway-Highway grade separation structures, a Supplemental Site Plan is required. The Supplemental Site Plan should be completed and submitted with the Final Site Plan. The reproduced tracing of this plan should have the title block deleted so that the railroad can use the plan to show force account work necessary to complete the highway project.

This plan shall show information necessary for the determination of slope lines, probable property requirements, sight distance and other items involved in determining the type of separation. The following information should be shown:

A. A 1” = 100’ [1 to 1000] scale plan of the alignment of the railroad and the highway extended at least 1000 feet [300 meters] each way from the proposed point of intersection, taken from actual surveys.

B. Profile of top of rails of all railroads, extending at least 1000 feet [300 meters] each way from the proposed intersection.

C. Sufficient cross sections along the railroad and highway to determine approximate earthwork limits and encroachment on railroad property.
D. In case a highway underpass type of separation is at all possible, the submitted information should show the line and profile of the nearest or best outlet for drainage.

E. Intersection angle between highway centerline and railroad centerline.

F. Highway stationing and railroad mile post stationing at intersection.

G. Railroad right-of-way lines.

H. Railroad pole lines, signal control boxes, communications relay houses, signal standards and drainage structures.

I. Centerlines of all tracks and location of switch points.

J. Location of buildings or other structures within the railroad right-of-way.

K. Railroad traffic counts including type of movements and speed.

L. Location of all utilities occupying railroad right-of-way and the names of the owners of these utilities.

203 BRIDGE WATERWAY

203.1 HYDROLOGY


For urban drainage areas less than 4 square miles [10.4 km²] discharges shall be estimated by the method described in USGS Water-Resources Investigations Report 93-135, “Estimation of Peak Frequency Relations, Flood Hydrographs, and Volume-Duration-Frequency Relations of Ungaged Small Urban Streams in Ohio”.

B. Discharge estimates may be calculated by other methods for comparison with StreamStats against verified flood elevations and other known river data to ensure that the most realistic discharge for the area is used for the design of the waterway opening. Calculations and comparisons shall be submitted for review.

C. Federal Emergency Management Agency (FEMA), National Flood Insurance Program (NFIP) Flood Insurance Studies; U.S. Corps of Engineer Flood Studies; U.S. Soils Conservation Studies; U.S. Water Resources Data and other reliable sources may be used as reference information in estimating discharges and flood elevations. However, for waterway crossings located in a NFIP study area, the base discharge \( Q_{100} \) from the NFIP study takes precedence over all other calculated discharges.
D. Where a U.S. Geological Survey estimate is in conflict with that of another agency, the agencies should be contacted in order that the discrepancy can be resolved. In general, the U.S. Geological Survey estimate shall be given preference.

E. Proposed structures upstream or downstream from a flood control facility shall be designed for discharges as supplied by the U.S. Corps of Engineers, Ohio Department of Natural Resources or the agency responsible for the flood control facility.

203.2 HYDRAULIC ANALYSIS

A. The design flood frequency shall be based on the importance of the highway and the design average daily traffic (ADT) as follows:

1. Freeways or other multi-lane facilities with limited or controlled access .................. 50 years

2. Other Highways (2000 design ADT and over) and freeway ramps......................... 25 years

3. Other Highways (under 2000 design ADT) ........................................................... 10 years

B. The total backwater produced for the design flood should be calculated by WSPRO (HY-7), HEC-2, HEC-RAS or other comparable backwater calculation methods.

C. The allowable backwater depth shall generally be governed by the nature of the upstream area at the structure location and/or the induced mean velocity through the structure.

D. Local Flood Plain Coordinators will need to be contacted so they may be made aware of planned waterway crossings and proposed backwater effects. A listing of Local Flood Plain Coordinators is maintained by the Ohio Department of Natural Resources (ODNR) and may be obtained by calling (614) 265-6750 or visiting ODNR’s, Division of Water website: http://www.dnr.state.oh.us/water/.

The Local Flood Plain Coordinator may require a permit for any proposed waterway crossing regardless of the drainage area size. The District Production Administrator should be contacted, by the responsible governmental agency which initiated the project, as to how they wish to coordinate the permit process. The granted permit becomes a record which is kept by ODOT, at the appropriate District office. The governmental agency will be required to make application for the permit and to secure a granted permit prior to the initiation of any detail plan preparation.

E. In areas where the topography is flat, backwater should not be permitted to flood unreasonably large areas of usable land, if possible.

F. In urban areas the waterway opening for proposed structures shall be designed so that the
allowable backwater elevation corresponds with the backwater elevation which currently exists.

G. When a proposed structure is subject to the approval of a Conservancy District, the waterway shall be designed to comply with their regulations if more restrictive than ODOT’s.

H. The design of all highway encroachments on the 100 year flood plain shall comply with the regulations as stated in the Code of Federal Regulations (23 CFR 650 A). Engineers responsible for bridge hydraulics should read these regulations to become familiar with their contents. When a highway encroachment is located in a detailed NFIP study area, create a duplicate of the original NFIP water surface model. Actual field survey data may be used to supplement the original NFIP data.

When making an encroachment, the proposed structure size submitted for preliminary design review shall be supported by an analysis of design alternatives with consideration given to capital costs and risk. “Risk” is defined as the consequences attributable to an encroachment. Risk includes the potential for property loss and hazard to life (A Flood Hazard Evaluation).

When making an encroachment on a NFIP designated flood plain in the floodway fringe, the rise in the water surface is limited to one foot [0.3 meters] above the natural 100 year flood elevation as given by the NFIP study. No increase in the 100 year water surface is allowed when encroaching on a NFIP designated floodway (44 CFR 60.3(d)(3)). See Figure 201.

For bridges located outside NFIP jurisdiction regions, the responsible government agency which initiated the project will be responsible for contacting and coordinating with the Local Flood Plain Coordinator.

Longitudinal encroachments require alternative location studies to be summarized in the Conceptual Alternatives Study (L&D Section 1403.3). Evaluation of specific bridge hydraulics may not be necessary when alternative highway alignments are under consideration for the project. Refer to the Code of Federal Regulations (23 CFR 650 A) for more specific information.

I. It should not be assumed that an attempt should be made to lower existing high water elevations. However, for bridge replacement projects where the existing structure is severely hydraulically taxed, an effort should be made to improve the hydraulics for both the design and 100 year recurrence interval discharges, with consideration of the one foot [0.3 meters] rise criterion discussed in Section 203.2.h. No allowable backwater requirements are set by these criteria; rather the allowable backwater should be determined by good engineering judgment considering the area inundated and the mean velocities induced through the structure.

J. In general, the bridge should be designed to clear the design year frequency flood. This criterion
may be waived because of roadway design constraints such as existing at-grade intersections, perpetuating existing profile grades, existing backwater elevations, presence of existing road overflow or other reasons.
K. Spill-thru type structures are generally preferred for cost effectiveness and hydraulic efficiency.

203.3 SCOUR

For bridges over waterways, armor the entire spill-through slope in front of the abutments and wingwalls, including the corner cones with Rock Channel Protection of the type determined from the following table. Rock Channel Protection requires the use of a filter. A 6 inch [150 mm] bed of crushed aggregate is allowed as an alternate in CMS 601.09 and should be specified when the rock is to be placed below water. The Item Master pay item descriptions allow for the differentiation between all options: with filter, with fabric filter or with aggregate filter.

The following table, relating bridge channel mean velocity of the design discharge versus rock type and thickness, shall apply as minimums. Special circumstances such as protection on the outside of curves or in northern regions of the state where ice flow is a concern may require greater rock thickness.

<table>
<thead>
<tr>
<th>Velocity (ft/s)</th>
<th>Type</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-8</td>
<td>C</td>
<td>2'-0”</td>
</tr>
<tr>
<td>8-10</td>
<td>B</td>
<td>2'-6”</td>
</tr>
<tr>
<td>above 10</td>
<td>A</td>
<td>3'-0”</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Velocity (m/s)</th>
<th>Type</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.4</td>
<td>C</td>
<td>600 mm</td>
</tr>
<tr>
<td>2.4-3.0</td>
<td>B</td>
<td>750 mm</td>
</tr>
<tr>
<td>above 3.0</td>
<td>A</td>
<td>1000 mm</td>
</tr>
</tbody>
</table>

The locations, length, and the top of slope elevations for the rock channel protection should be shown on the Site Plan. The rock should be shown in greater detail in the roadway section in conjunction with the channel plans. It will generally be economical to provide bank protection during the initial construction in order to provide sufficient embankment protection to minimize future maintenance.

A. Excavation for stream channel work shall be limited to that portion of the channel one foot [300 mm] above normal water elevation in order to minimize intrusion and to preserve the natural low water channel. Where the spill-thru slope infringes upon the natural low water channel, excavation should be made for placement of the rock channel slope protection at the toe of the slope.

B. Substructures for bridges over waterways shall be supported by piling or drilled shaft foundations unless the footings can be founded on bedrock. Substructures for precast reinforced concrete three-sided flat-topped and arch culverts are addressed in the Location and Design Manual, Volume 2.

C. For bridges over waterways where bedrock is determined to be at or close to the flow line, spread footings or drilled shafts shall be used. Spread footings shall be embedded into the bedrock in
accordance with the requirements of Section 204.1, except in laminated bedrock such as interbedded shale and limestone, in which case drilled shaft foundations with sufficient embedment into the bedrock are preferred.

D. A scour evaluation shall be performed for all bridges not founded on scour resistant shale or bedrock. All major rehabilitation work requires a scour evaluation. The scour evaluation may simply consist of determining what the bridge is founded on. For example, on a bridge rehabilitation, noting that the bridge is founded on scour resistant bedrock or deep foundations to bedrock, would constitute the scour evaluation. As a minimum, piles shall be embedded 15 ft [4.5 m] below the streambed elevation.

When evaluating a structure for scour, review all inspection reports for evidence of stream degradation (lowering of stream bed), scour or previous scour countermeasures. For existing footings founded on shale, test probe the shale to determine its resistance to weathering and note the relationship of the bottom of the footing to the stream bed elevation.

When it is necessary to calculate scour depths, they are to be calculated by the equations in HEC-18 (Hydraulic Engineering Circular No. 18, Pub. No. FHWA NHI 01-001), “Evaluating Scour at Bridges”. The text of HEC-18 should be read in order to understand scour and river mechanics. The references cited in Chapter 3 of HEC-18 are also helpful in understanding the concepts of scour and river mechanics. Scour depths should be considered in the design of the substructures and the location of the bottom of footings and minimum tip elevations for piles and drilled shafts.

A value of Q500 should be used as the super flood is to be estimated by 1.3 x Q100.

203.4 BRIDGE AND WATERWAY PERMITS

Impacts to bridges or waterways may require legal authorization in the form of permits or certifications issued by various regulatory agencies, including:

A. U.S. Army Corps of Engineers .................................................404 Permit and/or Section 10 Permit
B. U.S. Coast Guard ...........................................................................Section 9 Bridge Permit
C. Ohio EPA ...................................................................................401 Certification and/or Isolated Wetland Permit

The jurisdictional limit of the U.S. Army Corps of Engineers (USACE) is termed the “Waters of the United States” and, as noted in ODOT CMS 101.03, includes: rivers, streams, lakes and wetlands. For rivers and streams, the jurisdiction begins below the Ordinary High Water Mark (OHWM). The OHWM is defined as the elevation on the shore established by the fluctuations of water and indicated by physical characteristics such as a clear, natural line impressed on the bank; shelving; changes in the character of soil; destruction of terrestrial vegetation; the presence of litter and debris; or other appropriate means that consider the characteristics of the surrounding areas.

The USACE recently issued a Regional General Permit (RGP) for various activities conducted by ODOT within the “Waters of the United States”. This RGP authorizes the Department the
responsibility of ensuring compliance with Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act for transportation projects meeting prescribed conditions. Permitted activities within “Waters of the United States” allowed by the RGP include: construction of permanent fills or structures; rehabilitation of authorized fills or structures; and the temporary placement of fills or structures. A copy of the RGP may be downloaded at:

http://www.lrh.usace.army.mil/permits/

The ODOT Office of Environmental Services – Waterway Permits Unit (OES-WPU) assumes the responsibility for determining project eligibility for the RGP as well as all other bridge and waterway permits. The RGP will not be applicable to all ODOT projects such as those that impact navigable waterways and scenic rivers. When projects exceed the applicable limits of the RGP, the designer, project manager and ODOT District Environmental Coordinator (DEC) should meet with OES-WPU to determine the best course of action. The designer and project manager shall coordinate with the DEC and the OES-WPU throughout the permit determination process to ensure that the final waterway permit applications are indicative of the final project design. For more information refer to the Waterway Permits Manual available from the ODOT Office of Environmental Services.

Special Provisions are the method ODOT uses to attach the waterway permits and certifications to the project construction plans. The waterway permits Special Provisions Package (SPP) is prepared by the Office of Environmental Services – Waterway Permits Unit and may contain the following:

A. All pertinent waterway permits, certifications and related conditions
B. Drawings and/or mapping submitted with a permit application
C. Specialized plan notes associated with the waterway permits

The designer and the project manager shall confirm that the bridge design plans (e.g. the navigational clearances shown on the site plan, BDM Section 201.2.2.J; etc.) meet the requirements in the project waterway SPP (e.g. U.S. Coast Guard Section 9 Bridge Permit; U.S.A.C.E. 404 Permit; RGP; etc.) and shall ensure the project waterway SPP are submitted with the Final Plan Package.

203.5 TEMPORARY ACCESS FILLS

A Temporary Access Fill (TAF) is a fill or structure that allows a contractor access to work on roads or bridges located within bodies of water. TAF’s are work-type specific and are not required on every project. Examples of TAF’s include: cofferdams; temporary structures for maintaining traffic; causeways and workpads; and demolition debris. The placement of all TAF’s in “Waters of the United States” must be performed according to permits issued by the USACE.

A contractor’s means and methods of construction will dictate the TAF required for a project. However, the Department must estimate the potential impacts to “Waters of the United States” during project development to ensure all permits are in-place during contract letting. Furthermore, it is incumbent upon the Department that these permits provide all bidding contractors the ability to construct the project without resulting in expensive delays, change orders or fines. To that end, the
Department partnered with the Ohio Contractor’s Association to develop the following guidance to estimate the size of TAF’s:

A. The TAF shall provide access to all piers located within the Ordinary High Water Mark (OHWM) of the waterway from at least one bank of the waterway.

    Access may be provided by construction staging of the TAF. When considering the constructability of staged TAF’s, typical superstructure erection plans for lifting lengths of 50-ft or more require two cranes. Unless the access for member delivery is from an adjacent structure, the TAF must provide access to each end of the lift from one bank. In the case of staging, the permit application shall reflect the construction stage that impacts the largest area of the waterway.

B. The TAF shall be located directly beneath the superstructure. The surface width of the TAF shall be equal to the out-to-out width of the superstructure plus 50’-0” outboard on one side of the structure and 20’-0” outboard on the other side of the structure.

C. The TAF shall extend at least 40’-0” beyond the furthest pier accessed by the TAF.

D. Side slopes of the TAF shall be no steeper than 1.5:1 (H:V).

E. The top surface of the TAF shall be located 1’-0” above the OHWM.

F. The TAF shall be designed to maintain a flow equal to two times the highest average monthly flow (i.e. the largest of Q1, Q2, Q3, …Q12), as reported by the USGS web based application StreamStats (see BDM Section 203.1.A), such that no rise in the backwater above OHWM is permitted.

This information is intended for permit application purposes only and should not be included in the project plan set. However, to assist the OES-WPU in the determination process, Designers should use the guidance above to develop a plan view and cross-section of a TAF. An example plan view and cross-section are shown in Figure 208. These details should be provided to the DEC along with a completed copy of the checklist shown in Figure 209. The minimum flow to be maintained during construction should be calculated according to item F above. Designers will need to estimate whether this flow can be maintained through conduits or if open channels will be required.

204     SUBSTRUCTURE INFORMATION

204.1     FOOTING ELEVATIONS

Substructure footing elevations should be shown on the Final Structure Site Plan. The top of footing should be a minimum of one foot [0.3 meters] below the finished ground line. The top of footing should be at least one foot [0.3 meters] below the bottom of any adjacent drainage ditch. The bottom of footing shall not be less than four feet [1.2 meters] below and measured normal to the finished groundline.
Due to possible stream meander, pier footings for waterway crossings in the overflow section should not be higher than the footings within the stream unless the channel slopes are well protected against scour. Founding pier footings at or above the flow line elevation is strongly discouraged.

Where footings are founded on bedrock (note that undisturbed shale is bedrock) the minimum depth of the bottom of the footing below the stream bed, D, in feet [meters], shall be as computed by the following:
(THIS PAGE INTENTIONALLY LEFT BLANK)
D = T + 0.50Y

Where:
T  = Thickness of footing in feet [meters]
Y  = distance from bottom of stream bed to surface of bedrock in feet [meters]

The footing depth from the above formula shall place the footing not less than 3 inches [75 mm] into the bedrock.

204.2 EARTH BENCHES AND SLOPES

A bench at the face of abutment shall not be used. Rehabilitation projects may require special slope considerations.

Spill thru slopes should be 2:1, except where soil analysis or existing slopes dictates flatter slopes. The slope is measured normal to the face of the abutment.

For superelevated bridges over waterways, the intersection of the top of slope with the face of abutment shall be on a level line. For other superelevated structures the top of slope shall generally be made approximately parallel to the bridge seat. For structures over streets and roads having steep grades, the intersection of earth slope and face of abutment may be either level or sloping dependent upon which method fits local conditions and gives the most economical and aesthetically pleasing structure.

The spill-thru slope should intersect the face of abutment a minimum of one foot [300 mm], or as specified in a standard bridge drawing, below the bridge seat for stringer type bridges. For concrete slab and prestressed box beam bridges this distance should be 1'–6" [450 mm].

204.3 ABUTMENT TYPES

Preference should be given to the use of spill-thru type abutments. Generally for stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4’-0” [1200 mm]. For stub abutments on spread footing on soil, the minimum dimension shall be 5’-0” [1525 mm]. For any type of abutment, integral design shall be used where possible, see Section 205.8 for additional information.

Wall type abutments should be used only where site conditions dictate their use.

204.4 ABUTMENTS SUPPORTED ON MSE WALLS

When conditions are appropriate, the designer may consider stub type abutments with piling or spread footings supported on MSE walls. Use spread footings to support the abutment if the MSE wall is on bedrock or shale. If the MSE wall is on soil, then the selection of spread footings or piles
to support the abutment should consider possible settlement of the MSE wall. Use piles to support the abutment if the bridge is a continuous multi-span structure, or if the bridge is constructed part width in phases. If the bridge is a single-span structure and is not constructed part width in phases, then either spread footings or piles may be used to support the abutment. Piles require a minimum 15-foot embedment below the MSE wall.

Refer to Sections 201.2.6, 202.2.3 and 204.6.2 for the staged review requirements for MSE walls. Consult the Office of Structural Engineering for additional design recommendations.

204.5 PIER TYPES

For highway grade separations, the pier type should generally be cap-and-column piers supported on a minimum of 3 columns. (This requirement may be waived for temporary conditions that require caps supported on less than 3 columns.) Typically the pier cap ends should be cantilevered and have squared ends.

For bridges over railroads generally the pier type should be T-type, wall type or cap and column piers. Preference should be given to T-type piers. Where a cap and column pier is located within 25 feet [7.6 meters] from the centerline of tracks, crash walls will be required.

For waterway bridges the following pier type should be used:

A. Capped pile type piers; generally limited to a maximum height of 20 feet [6 meters]. For heights greater than 15 feet [4.5 meters], the designer should analyze the piles as columns above ground. Scour depths shall be considered.

B. Cap-and-column type piers.

C. Solid wall or T-type piers.

Note that the use of T-type piers, or other pier types with large overhangs, makes the removal of debris at the pier face difficult to perform from the bridge deck. For low stream crossings with debris flow problems and where access to the piers from the stream is limited, T-type piers, or other similar pier types, should not be used.

For unusual conditions, other types may be acceptable. In the design of piers which are readily visible to the public, appearance should be given consideration if it does not add appreciably to the cost of the pier.

204.6 RETAINING WALLS

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Retaining Wall Justification shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. A description of the Retaining Wall Justification is provided in Section 1404 of the ODOT Location and Design Manual, Volume Three. Generally, the justification compares the
practicality, constructability and economics of the various types of retailing walls listed below:

A. Cast-in-place reinforced concrete  
B. Precast concrete  
C. Tied-back  
D. Adjacent drilled shafts  
E. Sheet piling  
F. H-piling with lagging  
G. Cellular (Block, Bin or Crib)  
H. Soil nail  
I. Mechanically Stabilized Earth (MSE)  

Refer to SS840 for accredited MSE wall systems. Contact the Office of Structural Engineering for modular block wall systems. For wall systems that utilize geogrid reinforcements, the wall height shall be limited to 30 ft.

### 204.6.1 DESIGN CONSTRAINTS

Below are some design constraints to consider in the wall justification study to establish acceptable wall types:

A. Future use of the site (future excavations cannot be made in Mechanically Stabilized Embankments)  
B. Deflection and/or differential settlements  
C. Accessibility to the construction site  
D. Aesthetics, including wall textures  
E. Right-of-way (or other physical constraints)  
F. Cost (approximate cost analysis)  
G. Stage construction  
H. Stability (long-term and during construction)  
I. Railroad policies

### 204.6.2 STAGE 1 DETAIL DESIGN SUBMISSION FOR RETAINING WALLS

When a justification study has determined that a retaining wall is required, generally the wall will be a cast-in-place reinforced concrete wall or some type of proprietary wall system. The use of proprietary wall systems should be considered when the wall quantity for the project exceeds 5000 ft² [450 m²].
204.6.2.1 PROPRIETARY WALLS

If a proprietary wall is justified, the Design Agency shall include the following information in the Stage 1 Detailed Design Submission: wall alignment; footing elevations; allowable bearing pressure at the leveling pad elevation; a global stability analysis; the effect of settlement and settlement calculations; and any construction constraints, such as soil improvement methods, that may be required. Refer to Section 303.5 for plan requirements for Detail Design.
The alignment of proprietary retaining walls should be straight and with as few corners or curves as is practical. When changes in wall alignment are required, use gradual curves or corners with an interior angle of at least 135 degrees whenever possible. Do not use corners with interior angles of less than 90 degrees (acute corners).

The design of the wall shall be in conformance with the 17th Edition of the “AASHTO Standard Specifications for Highway Bridges” and the following:

A. Determine the height of the wall (H) for minimum soil reinforcement lengths as follows:

1. If the wall is not located at an abutment, measure (H) as the elevation difference between the top of the coping and the top of the leveling pad.

2. If the wall is located at an abutment, measure (H) as the elevation difference between the profile grade at the face of the wall and the top of the leveling pad.

B. The soil reinforcement length shall not be less than 70% of the wall height (H) or 8'-0" [2.5 m], whichever is greater. Only increase this minimum soil reinforcement length as necessary to meet external stability requirements (sliding, bearing resistance, overturning, overall global stability). Generally, the soil reinforcement length should not be greater than 150% of the wall height (H). Provide calculations with the Foundation Report, BDM Section 202.2.3, that justify soil reinforcement lengths exceeding 0.70H.

C. The thickness of the unreinforced concrete leveling pad shall not be less than 6 inches [150 mm]. The minimum distance from the top of the leveling pad the ground surface at a point located 4’-0” [1.2 m] from the face of the wall shall be the larger of 3’-0” [900 mm] or the frost depth. Refer to Figure 202 for more information.

D. The minimum thickness of the precast reinforced concrete face panels may be assumed to be 5½ inches [140 mm].

E. The maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent. Provide slip joints if the estimated differential settlement is greater than one (1) percent.
F. The following factors of safety shall apply:

<table>
<thead>
<tr>
<th>Factor of Safety</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding</td>
<td>&gt; 1.5</td>
</tr>
<tr>
<td>Overturning</td>
<td>&gt; 2.0</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>&gt; 2.5</td>
</tr>
<tr>
<td>Overall Stability</td>
<td>&gt; 1.5 (for walls supporting spread footing abutments)</td>
</tr>
<tr>
<td></td>
<td>&gt; 1.30 (for all other walls)</td>
</tr>
</tbody>
</table>

G. Use the following soil parameters for design:

<table>
<thead>
<tr>
<th>Fill Zone</th>
<th>Type of Soil</th>
<th>Soil Unit Weight</th>
<th>Friction Angle</th>
<th>Cohesion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Zone</td>
<td>Select Granular Embankment Material</td>
<td>120 lb/ft³ [18.9 kN/m³]</td>
<td>34°</td>
<td>0</td>
</tr>
<tr>
<td>Retained Soil</td>
<td>On-site soil varying from sandy lean clay to silty sand</td>
<td>120 lb/ft³ [18.9 kN/m³]</td>
<td>30°</td>
<td>0</td>
</tr>
</tbody>
</table>

Determine soil parameters for the foundation soils based on the soils encountered by the soil borings.

H. Compute the coefficient of lateral earth pressure, $k_a$, using the Coulomb equation.

I. MSE walls located within 25'-0" [7.6 m] of the centerline of tracks, or other distance as specified by an individual railroad, shall be protected by a crash wall as specified in Section 209.8 and the AREMA Manual for Railway Engineering. The MSE wall system does not meet the definition of a crash wall as defined by the AREMA Manual for Railway Engineering.

J. For MSE walls supporting abutments on spread footings, the minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment footing shall be 3'-0" [915 mm] and the minimum distance between the back face of the MSE wall panels and the centerline of the abutment bearings shall be 5'-0" [1525 mm].

K. For MSE walls supporting abutments on piles, the minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment footing shall be 1'-0" [305 mm] and the minimum distance between the back face of the MSE wall panels and the centerline of the closest row of piles shall be 3'-6" [1065 mm]. The distance between the centerlines of adjacent rows of piles shall be 3'-6" [1065 mm] to allow compaction of the fill between the pile sleeves.

L. Integral abutment designs placed on MSE wall embankments are prohibited. Semi-integral abutment designs are allowed.
M. The maximum allowable bearing pressure for a spread footing abutment placed on an MSE wall embankment shall be 4 ksf [190 kPa].

N. When detailing the pile layout and the design of the abutments and/or wingwalls, consider that 100% of the ground reinforcement shall be connect to the facing elements and the Department will not allow field cutting of reinforcement systems to avoid piles or other obstacles.

204.6.2.2 CAST-IN-PLACE WALLS

If a cast-in-place wall is justified, the design agency will be responsible for providing the complete wall design in the detail plans. The Stage 1 Detailed Design Submission shall include: footing elevations; allowable bearing pressures; a global stability analysis; settlement calculations, if necessary; and any construction constraints that may be required.

204.6.2.3 OTHER WALLS

The other wall types listed in Section 204.6 are for use with special project conditions such as top-down construction and other excavation methods. Contact the Office of Structural Engineering for recommendations when considering these other wall types. Typically only one wall type design shall be prepared for these methods.

205 SUPERSTRUCTURE INFORMATION

205.1 TYPE OF STRUCTURES

The types of superstructure generally used in Ohio consist of cast-in-place concrete slabs, prestressed concrete box or I-beams, and steel beams or welded plate girders. Normally shallow abutments and spill-thru slopes will be used. The type of superstructure used should be selected on the basis of economy as well as appearance. For special conditions where other types of superstructures may be considered, consult the Office of Structural Engineering for recommendations prior to initiating the design.
205.2 SPAN ARRANGEMENTS

The length of a bridge will be determined by the requirements for horizontal clearance at grade (highway or railway) separations or by the requirements for waterway opening at stream crossings. Typically for any given bridge, there are a number of combinations of spans and lengths of spans that can be utilized. Generally a preferred span arrangement that minimizes the number of substructure units should be used (i.e. fewer piers with longer spans).

For grade separation structures spanning any divided highway a two-span bridge with spill-thru slopes is preferred.

For waterway crossings, one or three span bridges are typically used. This span arrangement is preferred so that a pier is not located in the middle of the waterway. If a series of precast, three-sided structures are used to produce a multiple span structure over a waterway, spread footings on soil shall not be used to support any of the precast structures.

When a multiple span arrangement (4 spans or more) is required, the Cost Analysis should examine the most economical number of spans required based on total bridge costs, including a substructure and superstructure cost optimization study. Site conditions will govern the location of substructure units with respect to required horizontal clearances, foundation conditions and appearance.

On structures with steep grades, the designer should account for the load effects of the grade on the substructure units.

205.3 CONCRETE SLABS

Cast-in-place concrete slabs are normally used where site geometry dictates a curved alignment or variable superelevation and the use of prestressed concrete box beams is impractical. Since concrete slabs will generally yield the least superstructure depth they should be considered when vertical clearance is limited. For stream crossings where flood waters often inundate the structure, a concrete slab should be considered. When using cast-in-place concrete slabs the construction clearance requirements of the falsework should be considered.

Standard bridge drawings are available for the design of single span and three span continuous concrete slabs. The Standard Bridge Drawing for single span concrete slab bridges is SB-1-03. The spans range from 11 to 38 feet [3350 to 11 580 mm] with a maximum skew angle of 30 degrees. The Standard Bridge Drawing for three span continuous concrete slabs is CS-1-03. The spans range from 14’ - 17.5’ - 14’ [4260 mm - 5334 mm - 4260 mm] to 46’ - 57.5’ - 46’ [14 020 mm - 17 530 mm - 14 020 mm] with a maximum skew angle of 30 degrees. The drawings are based on a 60 lb/ft² future wearing surface and a live load of an HS25 truck and the alternate military vehicle. The edge beam is designed to support live load according to AASHTO 3.24.8 and the weight of the 42” BR-1 deflector parapet.
**205.4 PRESTRESSED CONCRETE BOX BEAMS**

The span limits for prestressed, side by side, concrete box beams generally range from 15 to 100 feet [5 to 30 meters]. These span limits are based on the current design data sheets with 0.167 in² [108 mm²] low relaxation strands, a concrete 28-day compressive strength of 7000 psi [48.3 MPa], a release strength of 5000 psi [34.5 MPa] and an HS25 truck. Prestressed box beams of up to 120 foot spans [36 meters] have been designed using 10,000 psi [68.9 MPa] concrete and larger diameter strands. Concrete compressive strengths should be limited to 5000 psi [34.5 Mpa] at release and 7000 psi [48.3 Mpa] at 28-days. Consult the Office of Structural Engineering for recommendations prior to designing a structure with higher compressive strengths.

The skew angle should be limited to a maximum of 30 degrees. Consult the Office of Structural Engineering for recommendations prior to designing a box beam structure with a higher degree of skew. For all four lane divided highways or where the design ADTT (one way) is greater than 2500, prestressed box beam superstructures shall not be used. Box beams may be used on curved alignment where the mid-ordinate is 6 inches [150 mm] or less, as long as the required bridge width is provided. The maximum asphalt wearing surface thickness for a non-composite designed box beam bridge shall be 8 inches [200 mm]. For multiple span bridges, individual span lengths may vary but the proposed box beam depth should be constant.

The Designer shall consider the site limitations for practical hauling. While weight of a precast bridge member is not typically a limiting factor, its length and ability to reach the jobsite may be a restriction. Maximum lengths are normally dictated by the smallest turning radius enroute to the project site. For beams 100 ft [30 m] long or more, the Designer should contact at least two approved fabricators of precast bridge members to obtain a written agreement stating that the member can be shipped to the project site. The agreements should be included in the Structure Type Study, Narrative of Bridge Alternatives.

Non-composite boxbeam designs should be used where over the side drainage is provided and where the combined deck grade is less than 4 percent. The combined deck grade, Cg, should be computed by the following equation:

\[
Cg = \left[\text{transverse deck grade}^2 + \text{roadway grade}^2\right]^{1/2}
\]

For a normal transverse deck grade horizontal to vertical of 3/16 inch per foot [1 to 64 (1.56 percent)], the maximum roadway grade would be 3.68 percent or less for non-composite design. Where the combined deck grade is greater than 4 percent or the deck drainage is confined to the bridge deck by a parapet, curb, etc., a composite design should be used.

**205.5 PRESTRESSED CONCRETE I-BEAMS**

The span limits for prestressed concrete I-beams (AASHTO Type II, III, IV, and Modified Type IV) generally range from 60 to 125 feet [18 to 38 meters]. The shapes are to conform to Standard Bridge
Drawing PSID-1-99. Consult the Office of Structural Engineering for recommendations prior to designing a structure with a non-standard shape. Cast-in-place concrete decks compositely designed shall be used. The transportation and weight requirements listed for box beams will also apply for I-beams.

Standard Bridge Drawing PSID-1-99 allows 28-day concrete strengths up to 7000 psi [48.3 MPa] and release strengths up to 5000 psi [34.5 MPa]. Consult the Office of Structural Engineering for recommendations prior to designing a structure with higher compressive strengths. Straight strand and draped strand designs are allowed. Refer to Section 300 of this Manual for the preferred methods to relieve excessive tensile stresses.

Prestressed I-beam highway bridges should have a minimum of 4 stringer lines.

Prestressed I-beam bridges that meet the vertical clearance specified in Section 207 are acceptable over highway crossings.

### 205.6 STEEL BEAMS AND GIRDERS

For spans greater than 60 feet [18 meters], rolled beams, up to and including the 40 inch [1000 mm] depth, or welded plate girders should be considered. Continuous spans shall be used for multiple span bridges. The ratio of the length of the end spans to the intermediate spans usually should be 0.7 to 0.8. The latter ratio is preferred because it nearly equalizes the maximum positive moment of all spans. Integrally designed structures may have end span ratios of as low as 0.6 if prevention of uplift is considered. For multi-span, composite designed, rolled beams, the maximum intermediate span is generally around 115 feet [35 meters]. For single span, composite designed, rolled beams, the maximum span is generally around 100 feet [30 meters].

While constant depth plate girders can be used in the same range as rolled beams, they are generally not as cost effective as rolled beams for the same span lengths. Haunched girders over the intermediate substructure units should be considered for spans greater than 350’-0” [105 meters] or where economics warrant their use. Selections of any steel members should be based on an overall cost analysis of the structure.

<table>
<thead>
<tr>
<th>Stringer type</th>
<th>Span length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled beam</td>
<td>up to 115’ [35 m]</td>
</tr>
<tr>
<td>Constant Depth Girder</td>
<td>100’ - 350’ [30 - 105 m]</td>
</tr>
<tr>
<td>Haunched Girder</td>
<td>&gt; 350’ [105 m]</td>
</tr>
</tbody>
</table>

Generally the minimum economical beam spacing for rolled beams is 8’-0” [2450 mm]. For plate girders a minimum spacing of 9’-0” [2750 mm] is generally recommended.
In order to facilitate forming, deck slab overhangs should not exceed 4’-0” [1200 mm]. On over the side drainage structures, the minimum overhang shall be 2’-3” [700 mm]. Where scuppers are required for bridge deck drainage, the overhang shall be 1’-6” [450 mm].

Steel rolled beam or girder highway bridges should have a minimum of 4 stringer lines.

ASTM A588[M]/A709[M] 50W should be selected wherever possible as it eliminates the need for a coating system and the maintenance associated with a coating system. See Section 300 of this Manual.

If a steel structure requires a coating system, the steel should be ASTM A572[M]/A709[M] 50. A coating shall be specified. See Section 300 of this Manual.

For more information on steel materials, see Section 300 of this Manual.

For bridges with significant substructure costs, the difference in dead loads between the steel superstructure versus a concrete superstructure should be considered in the Structure Type Study, Cost Analysis for choosing the most economical structure type.

For rehabilitation of steel beam or girder superstructures, a fatigue evaluation will need to be completed, as defined in Section 400 of this Manual, and included in the Structure Type Study, Narrative of Bridge Alternatives.

For rehabilitation projects involving deck replacements, existing structural steel stringer elevations should be obtained (field verified) in order to establish proposed deck slab depths and the proposed bridge profile.

205.7 COMPOSITE DESIGN

Composite design of concrete slab on steel beams or girders shall be used when the resulting design is more economical than a non-composite design. The preliminary designs shall be in sufficient detail to permit an adequate cost comparison for the Structure Type Study.

Composite design should be used as a method of increasing load carrying capacity of existing beams or girders when replacing the concrete deck of a bridge which was originally designed and constructed non-composite.

The Strength Design Method (i.e. Load Factor Design) is preferred over the Service Load Design Method (i.e. Allowable or Working Stress Design). If a designer determines that an existing superstructure is structurally deficient based on the Service Load Design Method, the designer shall re-analyze the structure based on the Strength Design Method before opting for a total superstructure replacement.
205.8 INTEGRAL DESIGN

Integral construction involves attaching the superstructure and substructure (abutment) together. The longitudinal movements are accommodated by the flexibility of the abutments (capped pile abutment on single row of piles regardless of pile type). These abutment designs are appropriate for bridge expansion lengths up to 250 feet [75 meters] (400 feet [125 meters] total length, assuming 2/3 movement could occur in one direction) and a maximum skew of 30 degrees. See Figure 203 for further criteria. The superstructure may be structural steel, cast-in-place concrete, prestressed concrete boxbeam or prestressed-I beams. Integral design shall be used where practical. This design should be used for uncurved (straight beams) structures and at sites where there are no concerns about settlement or differential settlement. See Figure 203 for additional limitations. An example of an integral design can be found in the figures portion of Section 300 of this Manual. There is a standard bridge drawing available that establishes details for integral abutment designs.

The limitations previously discussed are basically for steel superstructures. If a concrete superstructure is being proposed, longer structure lengths may be investigated. During preliminary design, consult the Office of Structural Engineering for recommendations on a specific site that exceeds the prescribed limits.

The expansion length, at the abutment, is considered to be two-thirds (2/3) of the total length of the structure. On new structures, all pier bearings should be expansion bearings. The pier expansion bearings are designed proportionally (by distance) to the assumption that the 2/3 movement could occur at one of the abutments.

If unsymmetrical spans (from a thermal neutral point viewpoint) are used, either all pier bearings are to be expansion or piers with fixed bearings are to be designed for the forces induced by unbalanced thermal movements.

The use of a fixed pier (i.e. fixed bearings), regardless of structural rigidity, does not allow an increase in bridge length nor does it reduce the 2/3 movement assumption. Depending on its distance from the abutments, the pier will need to be designed for a portion of the movement from the superstructure.

On rehabilitation projects, preference should be given to using expansion bearings at all piers. However, this is not meant to be used as a blanket statement to automatically and blindly replace the existing bearings. If an existing pier has a fixed bearing, the pier will need to be analyzed for the new, additional loading that results from the 2/3 movement assumption. The load will be proportional to the distance from the pier to an abutment. The fixed bearing will not be the thermal neutral point as was assumed in the original design.

205.9 SEMI-INTEGRAL DESIGN

Semi-integral design should be considered and is preferred to abutments with a deck joint. These abutment designs are appropriate for bridge expansion lengths up to 250 feet [75 meters] (400 feet
[125 meters] total length, assuming 2/3 movement could occur in one direction). Generally there are no skew limitations. The foundation for these designs must be stable and fixed in position. These designs are not applicable when a single row of piles is used. The expansion and contraction movement of the bridge superstructure is accommodated between the end of the approach slab and the roadway. This design should be used for uncurved (straight beams) structures and at sites where there are no concerns about settlement or differential settlement. An example of a semi-integral design can be found in the figures portion of Section 300 of this Manual.

Spread footings may be appropriate for semi-integral abutments but settlement should be evaluated. Consult the Office of Structural Engineering for recommendations during preliminary design.

To utilize a semi-integral design, the geometry of the approach slab, the design of the wingwalls, and the transition parapets if any must be compatible with the freedom required for the integral (beams, deck, backwall and approach slab) connection to translate longitudinally. The expansion and contraction movements of the bridge superstructure will be transferred to the end of the approach slabs, see Section 209.6, Pressure Relief Joints.

There is a standard bridge drawing available that establishes details for semi-integral abutment designs.

The limitations previously discussed are basically for steel superstructures. If a concrete superstructure is being proposed, longer structure lengths may be investigated. During preliminary design, consult the Office of Structural Engineering for recommendations on a specific site that exceeds the prescribed limits.

The expansion length, at the abutment, is considered to be two-thirds (2/3) of the total length of the structure. On new structures, all pier bearings should be expansion bearings. The abutment bearings shall always be expansion bearings and be designed for the assumption that the 2/3 movement could occur at one of the abutments. The pier expansion bearings are designed proportionally (by distance) to the abutment design length.

If unsymmetrical spans (from a thermal neutral point viewpoint) are used, either all pier bearings are to be expansion or piers with fixed bearings are to be designed for the forces induced by unbalanced thermal movements.

The use of a fixed pier (i.e. fixed bearings), regardless of structural rigidity, does not allow an increase in bridge length nor does it reduce the 2/3 movement assumption. Depending on its distance from the abutments, the pier will need to be designed for a portion of the movement from the superstructure.

On rehabilitation projects, preference should be given to using expansion bearings at all substructure units. However, this is not meant to be used as a blanket statement to automatically and blindly replace the existing bearings. If an existing pier has a fixed bearing, the pier will need to be analyzed for the new, additional loading that results from the 2/3 movement assumption. The load will be proportional to the distance from the pier to an abutment. The fixed bearing will not be the thermal
neutral point as was assumed in the original design.

206 MINIMAL BRIDGE PROJECTS

Minimal projects are defined in Section 1400 of the ODOT Location and Design Manual, Volume Three, as projects that do not alter the basic highway cross section or geometry, require no additional right-of-way, are exempt from Categorical Exclusion documentation, and require little or no public involvement. Minimal project types include: bridge painting, deck overlays, scupper installations, barrier facings, concrete sealing, partial depth concrete repairs, etc. Minimal projects do not require a preliminary design submission.

Minimal bridge projects shall have a General Plan. A Site Plan is not required. The General Plan should define all necessary information.

For all rehabilitation projects an “Existing Structure” data block and a “Proposed Structure” data block shall be provided. These standard data blocks provide a quick reference and documentation of proposed design changes. The “Existing Structure” data block shall include the Structural File Number (SFN). The first item in the “Proposed Structure” data block should be “Proposed Work” followed by a brief description of the type of work to be done (for example: Bridge deck repair using Microsilica Concrete Overlay, Concrete parapet refacing, etc.). Provide a relatively thorough description (list of work) of the type of work to be done within a plan note entitled “Proposed Work” and include this note on the sheet containing the General Plan.

207 BRIDGE GEOMETRICS

207.1 VERTICAL CLEARANCE

The “Required Minimum” and “Actual Minimum” Vertical Clearances and their locations shall be shown on the Preliminary Structure Site Plan, Section 201.2.2. The “Actual Minimum” Vertical Clearance is the minimum overhead clearance provided by the design plans.

A. For new and reconstructed grade separation structures, the “Required Minimum” Vertical Clearance shall not be less than the preferred clearance specified in ODOT’s Location and Design Manual, Figure 302-1 unless otherwise specified in the scope of services. A “Required Minimum” Vertical Clearance less than the L&D Manual minimum clearance will require a Design Exception in accordance with Section 105 of the L&D Manual.

B. For grade separation structures to remain, the “Required Minimum” Vertical Clearance shall not be less than the minimum clearance specified in ODOT’s Location and Design Manual, Figure 302-2 or 302-3 unless otherwise specified in the scope of services. A “Required Minimum” Vertical Clearance less than the L&D Manual minimum clearance will require a Design Exception in accordance with Section 105 of the L&D Manual.
For the purposes of determining vertical clearances, “Reconstructed” shall refer to an improvement of an existing structure involving the replacement of the entire superstructure.

207.2 BRIDGE SUPERSTRUCTURE

Bridge superstructure widths shall be established in accordance with ODOT’s Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.3 LATERAL CLEARANCE

Divided highways having four or more lanes crossing under an intersecting highway shall be provided with a minimum lateral clearance of 30 feet [9000 mm] from the edge of traveled lane to the point where the 2:1 back slope intersects the radius at the toe of the 2:1 slope. Refer to ODOT’s Location and Design Manual, Figure 307-2. To satisfy cost considerations or in order to maintain the typical roadway section (including roadway ditch) of the underpass through the structure, for four or more lane highways, wall abutments or the 2:1 slope of typical two-span grade separation structures may be located farther than 30 feet [9000 mm] from the near edge of traveled lane.

Lateral clearances for other roadway classifications shall be established in accordance with ODOT’s Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.4 INTERFERENCE DUE TO EXISTING SUBSTRUCTURE

Where a new pier or abutment is placed at the location of an existing pier or abutment the usual “Removal” note (and also the text of CMS 202.03) calls for sufficient removal of the old pier or abutment to permit construction of the new. However, a new pier or abutment preferably should not be located at an existing pier or abutment where the existing masonry may extend appreciably below the bottom of the proposed footing, or appreciably below the ground in case of capped-pile construction. This applies particularly where piles are to be driven. It is desirable to avoid the difficulty and expense of removing deep underground portions of the existing substructure and to avoid the resultant disturbance of the ground.

Where existing substructure units are shown on the Site Plan, the accuracy of the locations and extent should be carefully drawn. The existing substructure configuration should be shown based on existing plans or field verified dimensions, otherwise just a vertical line showing the approximate face of the abutment or pier widths should be shown. Misrepresentation of the location of the existing substructure units has resulted in expensive change orders during construction. Existing dimensions should be labeled as (+/-) plus or minus.
207.5  **BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION**

During the Assessment of Feasible Alternatives, the location of the proposed structure should be studied to attempt to eliminate the presence of excessive skew, curves or extreme superelevation transitions within the actual bridge limits.

208  **TEMPORARY SHORING**

208.1  **SUPPORT OF EXCAVATIONS**

Provide a pay item for cofferdams and excavation bracing for the following conditions:

A. Excavation that extends below the water table or water surface.

B. Excavation of soil that supports adjacent structures, railroads or active roadways. Show the approximate locations of shoring in the plans.

For shoring that supports adjacent structures, railroads or active roadways with at least 8-ft of retained earth, the Design Agency shall provide a temporary shoring design in the plans. The designer shall consider the feasibility of this temporary shoring during the Structure Type Study.

For projects involving Railroads, the requirements will be different as each railroad company has their own specific requirements. The Design Agency is responsible for contacting the responsible railroad and obtaining the specific requirements for design and construction.
Following are some conceptual ideas for the design of temporary shoring:

A. A cantilever sheet pile wall should generally be used for excavation up to approximately 12 feet [3.5 meters] in height. Design computations are necessary.

B. For cuts greater than 12 feet [3.5 meters] in height, anchored or braced walls will generally be required.

C. For anchored walls, the use of deadmen is preferred. Braced walls using waler and struts can sometimes be braced against another rigid element on the excavated side.

The use of soil or rock anchors (tiebacks) is generally the last option considered in the design of anchored walls.

D. The use of steel “H” piles with lagging is also a practical solution for some sites. Please note that some railroad companies allow only interlocking steel sheet piling adjacent to their tracks.

E. Where sufficient embedment can not be attained by driving sheet piling because of the location of shallow bedrock, predrilled holes into the bedrock with soldier “H” piles and lagging should be considered.

For cuts greater than 12-15 feet [3.5-4.5 meters], the “H” piles may need to be anchored.

F. The highway design live loading should be equal to two feet [600 mm] of equivalent soil height as a surcharge.

G. The following items at a minimum should be shown on the detail plans:

1. Minimum section modulus
2. Top and minimum bottom elevation of shoring
3. Limits of shoring
4. Sequence of installation and/or operations.
5. Method of payment
6. If bracing or tiebacks are required, all details, connections and member sizes shall be detailed.
7. A general note in plans allowing a Contractor designed alternate for temporary shoring.
208.2  SUPPORT OF EXISTING STRUCTURE

Whenever temporary support is required for a portion of an existing structure used to maintain traffic, the Design Agency shall provide sufficient information in the plans to allow contractors to prepare bids and construct the project. The feasibility of temporary support of an existing structure should be considered and discussed during the Structure Type Study.

The design shown in the plans should include: permissible locations of temporary support; temporary support loads; construction sequences; construction limitations not otherwise provided in C&MS 501.05; and any remaining plan notes. As a minimum, the plan notes should address method of measurement and basis of payment for temporary support.
209  MISCELLANEOUS

209.1  TRANSVERSE DECK SECTION WITH SUPERELEVATION

If the change in cross slope at the superelevation break point is less than or equal to 7 percent, then no rounding is required. For changes greater than 7 percent the bridge deck surface profile shall be as follows:

A. When the roadway break point is located between roadway lanes (not at the edge of pavement) the bridge cross slope is to extend to the toe of parapet. See “CASE a” in Figure 204.

B. When the roadway break point is located at the edge of pavement (adjacent shoulder width is less than four feet [1.2 meters]), the bridge cross slope is to be continued past the break point to the toe of deflector parapet. See “CASE b” in Figure 204.

C. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than four feet [1.2 meters] and less than eight feet [2.4 meters]), a four foot [1.2 meter] rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the ½ inch per foot [0.04] shoulder cross slope. See “CASE c” in Figure 205.

D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is equal to or greater than eight feet [2.4 meters]), a five foot [1.5 meter] rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the ½ inch per foot [0.04] shoulder cross slope. See “CASE d” in Figure 205.

The transition from the roadway approach transverse section to the bridge deck transverse section is to take place within the limits of the approach slab, whenever possible. On bridges with high skews, it may not be possible to do the transition within these limits and other alternatives should be considered during the Assessment of Feasible Alternatives.

For decks with over the side drainage, the treatment of the deck and the shoulder slopes shall be as described in subsections a through d above except that the slope shall continue to the edge of the deck.

209.1.1  SUPERELEVATION TRANSITIONS

Because of the complexities associated with superelevation transitions on bridge superstructures (i.e. beam and girder cambering, crossframe fabrication, deck form construction, slip forming of parapets, etc.) all reasonable attempts should be made to keep such transitions off of bridge decks. Where transitions must be located on bridge decks, preferably, the transitions should be straight. An example of a transition diagram is shown in Figure 206. A table with the information shown in Figure 206 is also acceptable. Where this is not practicable, then transition's discontinuities should
be smoothed by inserting 50 foot [15 meter] roundings at each discontinuity.

209.2  BRIDGE RAILINGS

All bridge structures on the National Highway System (NHS) or the State System require the use of crash tested railing meeting the loading requirements of TL-3 as defined by NCHRP report 350. The requirement for the NHS became effective October 1, 1998. For detailed information, refer to Section 304.

For structures with over the side drainage on the National Highway System, Twin Steel Tube Bridge Guardrail, Standard Bridge Drawing TST-1-99 should be used.

Over the side drainage shall not be used for bridges over highways and railroads. For four lane divided highways concrete deflector parapets shall be used. For bridges with heights of 25 feet [7.6 meters] or more above the lowest groundline or normal water, concrete deflector parapets should be used.

Refer to Section 305 of this Manual for vandal protection fencing requirements.

209.3  BRIDGE DECK DRAINAGE

The preferred minimum longitudinal grade of the bridge deck surface, when using concrete parapets, is 0.3 %, whenever possible.

The number of scuppers used for collecting the deck surface drainage should be minimized or eliminated if possible. The allowable spread of flow, which is used to help determine the need for scuppers, can be computed by the procedures as described in Section 1103 of the ODOT Location and Design Manual. Scuppers when provided, should preferably be located inside the fascia beam.

Drainage collection systems should be sloped as steeply as practical, generally not less than 15 degrees. The system should have a minimum bend radius of 18 inches [450 mm], no 90 degree bends, adequate pipe supports and cleanouts at the low ends of runs. The cleanout plugs should be easily and safely accessible. The necessary deck drainage outlet locations should be included in the Structure Type Study, Hydraulics Report.

Scuppers with drainage collection systems should be placed as closely as possible to the substructure unit which drains them. Uncollected scupper downspouts should be as far away from any part of the structure as possible.

When the deck drainage is to flow off the ends of the bridge, provisions must be made to collect and carry away this run-off. On bridges without MSE walls at the abutments and where the pavement flow from the deck is no more than 0.75 ft³/s [0.021 m³/s], a flume, as shown on Standard Construction Drawing DM-4.1, should be provided. On grade separation structures with 2:1
approach embankment slopes and where the pavement flow from the deck exceeds \( 0.75 \text{ ft}^3/\text{s} \) [0.021 \( \text{m}^3/\text{s} \)], an integral curb shall be provided on the approach slab with a standard catch basin located off the approach slab in lieu of the flume. At the trailing end of bridge barriers, a bridge terminal assembly is required to protect this curb. The catch basin should be a Catch Basin No. 3A, as shown on Standard Construction Drawing CB-2.2. A properly sized conduit (Type F, 707.05 Type C) shall be used to provide an outlet down the embankment slope and the outlet shall be armored to prevent erosion.

Control of drainage is especially critical at abutments with MSE walls. On structures with MSE walls at the abutments, a barrier shall be provided on the approach slab with a standard catch basin to collect the drainage. Where possible, the catch basin shall be located at least 25 ft [7.6 m] beyond the limits of the MSE wall soil reinforcement. Continue the barrier 10 ft [3.0 m] past the catch basin. Use the same type of catch basin and conduit as described above.

For bridges that have deck joints consisting of finger joints or sliding plates with a trough collector system scuppers should be considered near the joint to minimize the amount of deck drainage flow across the joint.

For bridges that have over the side drainage a stainless steel drip strip should be provided to protect the deck edge and beam fascia from the deck surface run-off.

### 209.4 SLOPE PROTECTION

For structures of the spill-thru type where pedestrian traffic adjacent to the toe of the slope is anticipated or the structure is located in an urban area within an incorporated city limit, the slope under the structure shall be paved with Concrete slope protection, CMS 601.07. Consideration of slope protection should be given to all areas under freeway bridges over city streets not covered by pavement or sidewalk. Drainage discharge from the bridge should be checked to ensure that discharge is not crossing sidewalks, etc. so that ice, dirt and debris build-ups are prevented.

On spill-thru slopes under grade separation structures, areas that are not protected by concrete slope protection, shall be protected by crushed aggregate material as provided in CMS 601.06.

The slope protection, either concrete or rock, shall extend from the face of the abutment down to the toe of the slope and shall extend in width to 3 feet [1 meter] beyond the outer edges of the superstructure, except that at the acute corners of a skewed bridge the outside edge of the slope protection shall intersect the actual or projected face of the abutment 3 feet [1 meter] beyond the outer edge of the superstructure and shall extend down the slope, normal to the face of the abutment, to the toe of the slope. The base of the slope protection shall be toed in. Note that the natural vegetation on the slopes when shaded by a new structure will die out. For this case additional slope protection should be considered.
209.5  APPROACH SLABS

Approach slabs should be used for all ODOT bridges. Determine the length of the approach slab using the following formula:

\[
L = \frac{1.5(H + h + 1.5)}{\cos \theta} \leq 30 \text{ ft} \\
L = \frac{1.5(H + h + 0.45)}{\cos \theta} \leq 9.15 \text{ m}
\]

Where:
- \( L \) = Length of the approach slab measured along the centerline of the roadway rounded up to the nearest 5 ft [1.5 m]
- \( H \) = Height of the embankment measured from the bottom of the footing to the bottom of the approach slab (ft) [m]
- \( h \) = Width of the footing heel (ft) [m]
- \( \theta \) = Skew angle

For four lane divided highways on new embankment, the minimum approach slab length shall be 25 ft [7.6 m] (measured along the roadway centerline). For structures with MSE walls at the abutments, the minimum approach slab length shall be 30 ft [9.1 m]. For all other structures the minimum length shall be 15 ft [4.6 m]. Refer to the approach slab standard bridge drawing for details.

Provide detail drawings for approach slabs which differ from the standard approach slabs. Examples include approach slabs that are a non-standard length, tapered, have a non-uniform width, or other such variation. When an approach slab falls within the limits of a vertical curve or superelevated section, the elevations at the edges of the approach slab shall be provided. Include these detail drawings in the structure plans for review during the detail design review stage. Approach slabs are paid for under Item 526.

On structure rehabilitation plans, when the existing approach slab is to be removed, the Designer shall include Item 202 - Approach Slab Removed in the structures estimated quantities.

209.6  PRESSURE RELIEF JOINTS

Type A pressure relief joints shall be specified when the approach roadway pavement is rigid concrete and shall be placed at the end of the approach slab.

The pressure relief joints are detailed on Standard Construction Drawing BP-2.3 (Revised 7/28/00), “Pressure Relief Joint Type A”.

209.7  AESTHETICS

Each structure should be evaluated for aesthetics. Normally it is not practical to provide cost premium aesthetic treatments without a specific demand, however careful attention to the details of the structure lines and forms will generally result in a pleasing structure appearance.
Some basic guidelines that should be considered are as follows:

A. Avoid mixing structural elements, for example concrete slab and steel beam superstructures or cap and column piers with wall type piers.

B. In general, continuous superstructures shall be provided for multiple span bridges. Where intermediate joints cannot be avoided, the depth of spans adjacent to the joints preferably should be the same. Avoid the use of very slender superstructures over massive piers.
C. Abrupt changes in beam depth should be avoided when possible. Whenever sudden changes in the depth of the beams in adjacent spans are required, care should be taken in the development of details at the pier.

D. The lines of the structure should be simple and without excessive curves and abrupt changes.

E. All structures should blend in with their surroundings.

One of the most significant design factors contributing to the aesthetic quality of the structure is unity, consistency, or continuity. These qualities will give the structure an appearance of a design process that was carefully thought out.

The aesthetics of the structure can generally be accomplished within the guidelines of design requiring only minimum special designs and minor project cost increase. As special situations arise preliminary concepts and details should be developed and coordinated with the Office of Structural Engineering.

If formliners are being considered, the depth of the projections should be as deep as possible in order to have the desired visual effect. Using shallow depths, such as \( \frac{3}{8}'' \) to \( \frac{1}{2}'' \) [6 to 13 mm], provides very little, if any, visual effect (relief) when viewed from a distance. The depth of the formliner shall not be included in the measurement of the concrete clear cover.

The use of colored concrete, where the color is integral with the concrete mix, should generally not be used since the final visual appearance of the concrete is not uniform. The color varies greatly due to the aggregate, cement type, cement content and the curing of the concrete. None of these items are reasonably controlled in the field to a sufficient enough degree to insure a uniform final appearance. If color is required, a concrete coating should be used which will not only produce the required color but will also provide the necessary sealing of the concrete as required in Section 300 of this Manual.

The use of formliners and/or coloring of the concrete should be evaluated on a cost basis and submitted as part of the Structure Type Study, Cost Analysis.

For additional guidance, refer to the Department's document entitled “Aesthetic Design Guidelines” available at the Design Reference Resource Center on the Department's website.

### 209.8 RAILWAY BRIDGES

For railway overpasses the specific requirements of the railway company involved need to be addressed. The design and operational requirements of the railway companies will vary from railway line to railway line and between companies. Some of the common railway concerns are as follows:

A. Horizontal and vertical clearances for both the proposed design and during construction,
B. The constructability of the substructure units adjacent to their tracks,

C. Allowing adequate clearances for drainage ditches and access roads that are parallel to their tracks,

D. Location of railway utilities, and

E. Provisions for crash walls on piers.

Consideration for providing future tracks and the possibility of track abandonment should be investigated. All submissions are to be made in accordance with the Department's review process. Railway submissions shall be made as directed by the District planning administrator. The guidelines of the individual railway company may be requested thru the District's designated rail transportation coordinator.

Generally if a steel superstructure is proposed over the railway the type of steel should be ASTM A588[M]/A709[M] 50W steel. Bridges located in urban areas or which have sidewalks located on the bridge should include protective fencing. Preferably drainage from the bridge should be collected in drain pipes and drained away from the railway right of way. No drains shall be allowed to drain on the railroad tracks or roadbed.

Where piers are located within 25'-0" [7.6 meters] of the centerline of tracks or if required by an individual railroad, a crash wall shall be provided unless a T-type or wall type pier is used. Crash walls should have a minimum height of 10 feet [3.1 meters] above the top of rail, except where a pier is located within 12 feet [3.6 meters] of the centerline of tracks and in that instance the minimum height should be 12 feet [3.6 meters] above the top of rail. The crash wall shall be at least 2'-6" [760 mm] thick. For a cap and column pier the face of the wall shall extend 12 inches [300 mm] beyond the face of the columns on the track side. The designer should note that this requirement does not automatically require a crash wall thickness greater than the minimum. The crash wall should be anchored to the footings and columns.

When temporary shoring details are required for construction of substructure units adjacent to railway tracks, details shall be included in the plans. When considering excavation for substructure units, address whether sheet piling can be driven (avoid existing footing, clear any battered piles, elevation of bedrock, etc.) and whether the proper lengths can be provided to retain the railway tracks. The design should be such that no settlement of the tracks is allowed. Interlocking sheet piling of cantilever design is preferred. It may be appropriate to leave the temporary shoring in place after construction.

The minimum vertical clearance from the top of rail should be 23'-0" [7.0 meters]. The point of minimum vertical clearance should be measured (calculated) from a point six feet [1.8 meters], measured horizontally, from the centerline of tracks measured level with the top of the high rail. The horizontal clearances vary between railway companies and need to be addressed for each specific site. Minimum construction clearances shall at least be 14'-0" [4.25 meters] horizontal,
measured from centerline of tracks, and 22’-0” [6.7 meters] vertical, measured six feet [1.8 meters] from centerline of tracks, wherever possible.

209.9 **BICYCLE BRIDGES**

Reference should be made to ODOT’s most current design guidelines and Section 300 of this Manual. The current design guidelines can be obtained from ODOT’s Office of Multi-Modal Planning. For new structures generally the minimum bridge width should be the same as the width of the paved bicycle path and approach shoulders. A minimum transverse slope of 1/4 inch per foot [0.021] sloped in one direction should generally be used. Bicycle railings should be a minimum of 3’-6” [1065 mm] high. Except as noted herein, refer to AASHTO Section 2.7.2 for additional bicycle railing design requirements. If an occasional maintenance vehicle is going to use the bridge, the railing should only be designed as a bicycle railing. The type of bridge deck joints used should be bicycle safe.

If a timber deck is used, a 1½ inch [38 mm] minimum thickness of Item 448, Asphalt Concrete Surface Course, Type 1, PG64-22, shall be applied in order to provide an abrasive skid resistant surface. Consult the Office of Structural Engineering for recommendations before specifying other alternative surfaces.

209.10 **PEDESTRIAN BRIDGES**

Pedestrian facilities shall meet the grade and cross slope requirements specified in Volume One, Section 306.2.5 of the ODOT Location & Design Manual. For pedestrian bridges over highways an additional one foot [300 mm] of vertical clearance shall be provided. The current AASHTO design guide for pedestrian bridges should be followed.

If a timber deck is used, a 1½ inch [38 mm] minimum thickness of Item 448, Asphalt Concrete Surface Course, Type 1, PG64-22, shall be applied in order to provide an abrasive skid resistant surface. Other alternative surfaces may be used if approved by the Department.

209.11 **SIDEWALKS ON BRIDGES**

Sidewalks should be provided where significant pedestrian traffic is anticipated and/or the approach roadway has sidewalks or requires provisions for future sidewalks. Refer to Volume One, Section 306.4 of the ODOT Location & Design Manual for specific pedestrian traffic requirements. The width of the bridge sidewalk is generally the width of the approach sidewalk plus 12 inches [300 mm], with the widths typically between 5 and 6 feet [1500 and 1800 mm] wide.

An 1/4 inch per foot [0.02] cross slope should be provided to drain the sidewalk towards the curbline. The sidewalk height shall be 8 inches [203 mm] on the bridge, tapering down to the approach curb height within the length of the approach slab.
A detail of the standard curb (height, face slope, and corner rounding) should be given. Refer to Section 300 of this Manual for vandal protection fencing requirements.

209.12 MAINTENANCE AND INSPECTION ACCESS

Maintenance and inspection access requirements should be included in the Structure Type Study, Narrative of Bridge Alternatives. For multiple span bridges with 8 feet [2400 mm] or deeper girders, an inspection handrail located on the girders should be provided. Also catwalks should be considered. Safety cables and other fall arrest systems should be considered in addition to handrails and catwalks. Provisions for maintenance and inspection access should be provided for fracture critical girders, cross girders and bents that cannot be inspected from a snooper. The use of fracture critical members is strongly discouraged. For these types of structures, consult the Office of Structural Engineering for details and recommendations. Additional information is provided in “FHWA Guidelines for Providing Access to Bridges for Inspections”, dated November 1985.

209.13 SIGN SUPPORTS

Research has shown that overhead sign supports located on bridges are highly susceptible to fatigue damage. Every effort shall be made to locate overhead sign supports off of bridge structures. When this is not possible, only two locations on the structure are acceptable and are listed below in order of preference:

A. Mounted directly to the substructure unit.
B. Mounted to the superstructure directly over a substructure unit.

Sign supports attached to the fascia of overpass bridges, as shown on Standard Construction Drawings TC-18.24 and TC-18.26, should also be avoided. Consult with the District Bridge Engineer before specifying their use.
Figure 201

100 YEAR FLOOD PLAIN

FLOODWAY
FRINGE

FLOODWAY

FLOODWAY
FRINGE

STREAM
CHANNEL

FLOOD ELEVATION WHEN CONFINED WITHIN FLOODWAY

ENCROACHMENT

ENCROACHMENT

AREA OF FLOOD PLAIN THAT COULD BE USED FOR DEVELOPMENT BY RAISING GROUND

THALWEG ELEVATION

FLOOD ELEVATION BEFORE ENCROACHMENT ON FLOOD PLAIN

SURCHARGE

LINE A - B IS THE FLOOD ELEVATION BEFORE ENCROACHMENT
LINE C - D IS THE FLOOD ELEVATION AFTER ENCROACHMENT
* SURCHARGE IS NOT TO EXCEED 1.0 FOOT OR THAT ALLOWED BY OTHER REGULATORY AGENCIES IF MORE RESTRICTIVE.

FLOODWAY SCHEMATIC
This figure has been retired
(Effective 01-21-05)
CASE a.

CASE b.
HEC-RAS File Structure

Figure 207
The temporary access fill shall accommodate a flow rate (Q) equal to twice the highest mean monthly flow such that the backwater elevation does not exceed the OHWM. Q for this location is XXXX cfs.
Temporary Construction, Access and Dewatering Activities
ODOT Regional General Permit (RGP) (Part C) Determination Checklist

The purpose of this form is to aid the Office of Environmental Services – Waterway Permits Unit (OES-WPU) in the permit determination process and in determining eligibility for the ODOT RGP-Part C. A completed copy of this form and a temporary construction access plan shall be forwarded to the DEC to be included in the Permit Determination Package submitted to OES-WPU.

Co-Rte-Sec: ___________________________ PID: _________________________________

Description: __________________________________________________________________

During the construction of this project, the following activities in the waters of the United States are anticipated: (check all that apply)

☐ Temporary structure for maintaining traffic
☐ Cofferdams
☐ Temporary access fill (e.g. causeways and work pads)
☐ Demolition and debris removal

The RGP requires an authorized temporary activity to accommodate a minimum flow equal to twice the highest mean monthly flow without creating a rise in backwater above the OHWM. **The minimum flow to be maintained throughout construction for this location is _____ cfs.**

The means that will most likely be implemented by the Contractor to maintain this flow will be:

☐ Conduit(s)
☐ Open channel(s)\Temporary Bridge

The RGP has limitations. Please read the limitations and provide the required measurement as it applies for this project.

☐ The maximum length of temporary impact, as measured upstream to downstream along one bank, cannot exceed 250-ft. **Proposed impact length for this project is ____ ft.**

☐ The proposed activity cannot be located within 2000-ft of a flood control facility or within 1000-ft of a stream gage. **Distance to flood control facility is ____ ft. Distance to stream gage is ____ ft.**

☐ The duration of the impact to waters of the United States cannot exceed 2 years. **Proposed temporary impact duration is ____ years.**

A complete copy of the RGP with the OEPA conditions may be downloaded at the following website:
http://www.dot.state.oh.us/Divisions/TransSysDev/Environment/Ecological_Resources_Permits/WATERWAY_PERMITS/Pages/default.aspx

cc. District Environmental Coordinator (DEC)

**Figure 209**
Figure 2014

100 YEAR FLOOD PLAIN

FLOODWAY FRINGE

FLOODWAY FRINGE

FLOODWAY

FLOODWAY

STREAM CHANNEL

FLOOD ELEVATION WHEN CONFINED WITHIN FLOODWAY

ENCROACHMENT

ENCROACHMENT

A

C

D

B

AREA OF FLOOD PLAIN THAT COULD BE USED FOR DEVELOPMENT BY RAISING GROUND

THALWEG ELEVATION

FLOOD ELEVATION BEFORE ENCROACHMENT ON FLOOD PLAIN

LINE A - B IS THE FLOOD ELEVATION BEFORE ENCROACHMENT
LINE C - D IS THE FLOOD ELEVATION AFTER ENCROACHMENT

* SURCHARGE IS NOT TO EXCEED 300 mm OR THAT ALLOWED BY OTHER REGULATORY AGENCIES IF MORE RESTRICTIVE.

FLOODWAY SCHEMATIC
THIS FIGURE HAS BEEN RETIRED
(EFFECTIVE 01-21-05)
CASE a.

CASE b.

Figure 204M
CASE c.

1.2 m < SHOULDER < 2.4 m

LANE  LANE  SHOULDER

ROADWAY

1.2 m ROUNDING

EDGE OF LANE

1.5m ROUNDING

EDGE OF LANE

CASE d.

SHOULDER > 2.4 m

LANE  LANE  SHOULDER

ROADWAY

1.5 m ROUNDING

BRIDGE

Figure 205M
HEC-RAS File Structure
SECTION 300 – DETAIL DESIGN

301 GENERAL

301.1 DESIGN PHILOSOPHY

Section 300 of this Manual establishes general design guidelines, details, special requirements and reasonable alternatives, which, when incorporated by the engineer in a set of bridge plans, will provide a bridge structure that meets load requirements, provides structural integrity, provides structural efficiency and reduces long term maintenance to a minimum level.

301.2 DETAIL DESIGN REVIEW SUBMISSIONS

The detail design review for structures is conducted as part of the Stage 2 and Stage 3 review submission.

The Stage 2 Detail Design submission should include an updated cost estimate and the items listed below. Not every item listed will apply to every project.
A. Bridge Plans generally consisting of the following:
   1. Site Plan in compliance with all Stage 1 review comments
   2. General Plan (if required)
   3. General Notes
   4. Phase Construction Details
   5. Foundation Plan
   6. Abutment Details with all dimensioning, bar marks and bar spacings properly shown
   7. Pier Details with all dimensioning, bar marks and bar spacings properly shown
   8. Superstructure Details with all dimensioning, bar marks and bar spacings properly shown
   9. Other Details as necessary

B. Retaining Wall Plans generally consisting of the following:
   1. General Notes
   2. Retaining wall details
   3. Other Details as necessary

C. Noise Barrier Plans generally consisting of the following:
   1. General Notes
   2. Plan and Profile Views
   3. Noise Barrier Details
   4. Foundations Table
   5. Subsurface Investigation Plan Sheets
   6. Other Details as necessary

D. Special Provisions

E. Load Rating Reports for bridges (Major and Minor PDP)

The Stage 3 Detail Design plan submission should include an updated cost estimate and the
following:
A. Stage 2 Detail Design plans in compliance with all Stage 2 review comments.
B. Completed Estimated Quantities Table
C. Completed Reinforcing Steel Schedule
D. Estimated Quantities calculations
E. Load Rating Reports for bridges (Minimal PDP only)

Refer to Section 1400 of the ODOT Location and Design Manual, Volume Three, for additional staged review submission requirements.

For structures with non-redundant and/or fracture critical design details, a complete Stage 2 Detail Design Review Submission shall be made to the Office of Structural Engineering for concurrent review and comment. The Office of Structural Engineering will forward all comments to the responsible District Office or LPA.

301.3 DESIGN METHODS

Ohio Department of Transportation bridge designs are to be developed in general conformance with the latest edition of the American Association of State Highway and Transportation Officials’ Standard Specifications for Highway Bridges (AASHTO), including all interims. Exceptions to AASHTO standards are documented in this Manual. Bridges designed within the limitations placed on the various superstructure types by AASHTO and this Manual can be considered as “typical” or “normal” in that these designs make use of empirical formulae and methods rather than more refined analysis methods.

The Strength Design Method (i.e. Load Factor Design) is preferred over the Service Load Design Method (i.e. Allowable or Working Stress Design). If a designer determines that an existing superstructure is structurally deficient based on the Service Load Design Method, the designer shall re-analyze the structure based on the Strength Design Method before opting for a total superstructure replacement.

When site conditions require the use of a superstructure type that exceeds the recommended limits set forth by AASHTO and/or this Manual, a special design method may be required using either a two-dimensional or three-dimensional model and some type of numerical analysis to solve the model. When this occurs, the designer should place a note in the General Notes section of the detail construction plans listing the type of model used, method of analysis and assumptions made during the design. Examples of special design methods include grillage, finite element, finite strip and classical plate solutions. A sample note can be found in Section 600 of this Manual.
For design of Temporary Structures see Section 500 of this Manual.

301.4 LOADING REQUIREMENTS

All bridge structures shall be designed for an HS25 [MS22.5] loading or the alternate military loading, whichever produces the greatest stresses and live load deflections, unless otherwise stated in this manual. Figure 301 illustrates the HS25 [MS22.5] truck and lane loadings.

All bridges shall be designed for a future wearing surface (FWS) of 60 psf [2.87 kPa].

All steel structures shall be designated as Case I or Case II as defined by AASHTO for fatigue design.

Bridge structures on LPA projects shall be designed to the same loading requirements as traditionally funded projects except an HS20-44 [MS18] loading may be used in lieu of the HS25 [MS22.5] loading.

301.4.1 PEDESTRIAN AND BIKEWAY BRIDGES

Pedestrian and bikeway bridges shall be designed in accordance with the latest edition of AASHTO, ODOT design guidelines and this Manual. The most current design guidelines can be obtained from ODOT’S Office of Local Projects (614)644-7095.

Bridges that cannot accommodate vehicles because of narrow roadway or walkway widths or other access limitations shall be designed in accordance with the AASHTO Guide Specifications for Design of Pedestrian Bridges.

Bridges whose width can accommodate service vehicles shall be designed in accordance with the AASHTO Guide Specifications for Design of Pedestrian Bridges and an H15-44 [M13.5] vehicle.

301.4.2 RAILROAD BRIDGES

Bridges are to be designed in accordance with current AREMA specifications and the individual railway company's loading requirements. All other aspects of the structure design shall conform to AASHTO.
301.4.3 SEISMIC DESIGN

Outlined in this section, are general Seismic design requirements for Ohio. Ohio is considered to be in Zone A based on acceleration coefficients below 0.09. The following information is only meant to highlight AASHTO requirements. The designer should refer to AASHTO for complete requirements.

Zone A structure designs are to comply with two requirements:

A. Connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times the dead load reaction force in the restrained direction. The restrained direction for an expansion bearing is transverse to the structure.

B. Bearing seats shall be designed to provide minimum support length N, measured normal to the face of an abutment or pier. N shall not be less than computed by the following formula:

\[ N(\text{in}) = (8+0.02L+0.08H)(1+0.000125S^2) \]

\[ N(\text{mm}) = (203+1.67L+6.67H)(1+0.000125S^2) \]

Where:

\[ L = \text{length, in feet [meters], of the bridge deck to the adjacent expansion joint or to the end of the bridge deck.} \]

\[ S = \text{Angle of skew of support in degrees, measured from a line normal to the span.} \]

For Abutments:

\[ H = \text{average height, in feet [meters], of columns supporting the bridge deck to the next expansion joint. For single span bridges, } H = 0. \]

For columns and/or piers:

\[ H = \text{column or pier height, in feet [meters].} \]

For hinges within a span:

\[ H = \text{Average height of the adjacent two columns or piers, in feet [meters].} \]

Abutment and pier designs with level caps and raised pedestal bearing seats shall not be used.
301.5 REINFORCING STEEL

Reinforcing steel - ASTM A615 or A996, Grade 60, F_y = 60,000 psi.

Reinforcing steel - ASTM A615M or A996M, Grade 420, F_y = 420 MPa

All reinforcing steel shall be epoxy coated.

301.5.1 MAXIMUM LENGTH

Generally maximum length of reinforcing steel should be 40 feet [12.2 meters]. This limit is for both transit purposes and construction convenience. The maximum length before a lap splice is required is 60 feet [18.4 meters]. To facilitate an economical design using 60 foot bar stock, where multiple sets of lapped bars are required (i.e. longitudinal slab reinforcement) consideration should be given to using multiple sets of 30 foot long bars.

The length of the short dimension of L-shaped bars should be limited in order not to extend beyond the sides of a highway vehicle of maximum legal width. The short dimension should preferably be not greater than 7'-6" [2300 mm], and in no case greater than 8'-0" [2450 mm].

301.5.2 BAR MARKS

Bar marks shall be used on detail plans to identify the bar's size and general location and to reference the bar to the reinforcing bar list.

Letters should be incorporated into the bar marks to help identify their location in the detail plans: "A" for abutments, "P" for piers, "S" for superstructure, “SP” for spirals, “DS” for drilled shafts, etc.

The following bar mark represents a #5 [16M] abutment bar ......................... A501 [A16M01]
The following bar mark represents a #4 [13M] spiral bar ................................. SP401 [SP13M01]
The following bar mark represents a #9 [29M] drilled shaft bar ...................... DS901 [DS29M01]

A note or legend within the bar list sheet in the plans shall describe each bar mark's meaning. See Figure 302.

301.5.3 LAP SPLICES

Bar splice lengths shall be shown on the plans.

Development and splice lengths shall conform to AASHTO requirements.

Reinforcing steel at construction joints should extend into the next pour only by the required
splice length.

Reinforcing steel shall not project through expansion and contraction joints.

In lieu of lap splices, mechanical splices in accordance with the requirements of CMS 509 may be used.

CMS 509 Mechanical splices should develop a minimum ultimate strength of 125 percent of the required yield strength of the reinforcing steel they connect. Standard reinforcing steel develops a minimum ultimate strength of 150 percent of the minimum required yield strength. The designer should be aware of this 17% reduction in the ultimate tensile strength of the reinforcing at the location of the mechanical splice.

Due to lap splice lengths required, the designer should use mechanical type splices for #14 [43M] and #18 [57M] bars.

Splicing of reinforcing by welding is not permitted.

Where a horizontal construction joint is used in a column or pier, the reinforcement should be continuous and splices avoided if at all possible. An exception to this is the construction joint between a column and a footing, where the reinforcement should be discontinuous and adequate splice length should be furnished.

For tension splice lengths, see Figure 303.

For compression splice lengths, see Figure 304.

For development length requirements for reinforcing steel, see Figures 304, 305 & 306.

**301.5.4 CALCULATING LENGTHS AND WEIGHTS OF REINFORCING**

Reinforcing steel lengths shall be calculated to the nearest 1 inch [25 mm]. Standard bend lengths shall be based on criteria in CMS 509.

The length or height of a spiral is defined as the distance out-to-out of coils, including the finishing turns at top and bottom.

The weight of the additional 1-1/2 coils of spiral required at the end by AASHTO 8.18.2.2.4 shall be calculated and included in the estimated quantities. For one, #4 [13M] spiral with a 4½" pitch, the weight, including the 1-1/2 coils at each end, is given by the following formula:
Spiral Weight (lb) = 0.148πH\left\sqrt{\frac{4.5}{2}} + (D - 0.5)^2 + 0.167π(D - 0.5)\right\]

Where:  
D = Outside Diameter of the Spiral (in)  
H = Height or Length of the Spiral (ft)

Spiral Weight (kg) = 8.643πH\left\sqrt{\frac{0.115}{2}} + (D - 0.013)^2 + 0.248π(D - 0.013)\right\]

Where:  
D = Outside Diameter of the Spiral (m)  
H = Height or Length of the Spiral (m)

See Figure 307 for area, weight and diameter of standard reinforcing. See Figure 308 for bar bending data. See Figure 309 for standard bar length deductions of common bends.

301.5.5 BAR LIST

Bar lists should include the following:

A. Bar Mark  
B. Number of bars required  
C. Overall length required of the bar  
D. Total Weight for each bar mark  
E. Column for type of bar:  
   1. "ST" for straight  
   2. "Number" assigned to  
   3. "Numbered Bent Bar Detail"  
   4. "Number" and "Series" for series bars

Dimensions are defined by letters A through Z associated with the “Numbered Bent Bar Detail” showing position of letters.

Spiral reinforcing shall also be included in the detail plan's bar list. The following information shall be shown on the bar list:

A. Core diameter  
B. Pitch  
C. Mark  
D. Number  
E. Height  
F. Weight  
G. Plan note for spiral bars

A sample bar list is provided in Figure 302.
301.5.6 USE OF EPOXY COATED REINFORCING STEEL

All reinforcing steel shall be epoxy coated except as noted for prestressed box beams in Section 302.5.1.8.

All approach slabs shall have epoxy coated reinforcing steel.

301.5.7 MINIMUM CONCRETE COVER FOR REINFORCING

The clearances of reinforcing steel from the face of the cast-in-place concrete shall be as follows:

A. Top reinforcing steel in bridge decks and sidewalks (including a one inch [25 mm] monolithic wearing surface) ................................................................. 2½ inches [65 mm]
B. Bottom reinforcing steel in bridge decks ............................................... 1½ inches [40 mm]
C. Bottom steel in footings ........................................................................... 3 inches [75 mm]
D. Column steel or spirals ............................................................................. 3 inches [75 mm]
E. All other concrete .................................................................................. 2 inches [50 mm]

Clearances not given in CMS 509.04 shall be shown in the detail plans.

301.5.8 MINIMUM REINFORCING STEEL

Minimum reinforcing steel requirements shall conform to AASHTO requirements for shrinkage and temperature reinforcement. Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced.

The total area of reinforcing steel to be provided shall be 1/8 in² per foot [265 mm² per meter] in each direction.

The spacing of temperature and shrinkage reinforcement shall not exceed 3 times the wall or slab thickness, or 18 inches [450 mm].

301.6 REFERENCE LINE

For structures on a horizontal curve a reference line, usually a chord of the curve shall be provided. This reference line should be shown on the General Plan/Site Plan view with a brief description, including, for example, “Reference Line (centerline bearing to bearing),” and the stations of the points where the reference line intersects the curve. Skews, dimensions of substructure elements and superstructure elements should be given from this Reference Line,
both on the General Plan/Site Plan and on the individual detail sheets. Dimensions from the curve generally should be avoided. The distance between the curve and reference line should be dimensioned at the substructure units. In this manner a check is available to the contractor.

The reference tangent can be used if appropriate.

301.7 UTILITIES

Utilities should not be supported on the fascia of bridge decks.

Utilities, other than gas and water, may be run through sidewalk sections or parapets of bridges but shall be encased in a protective conduit.

Placing utilities through or underneath MSE walls should be avoided when possible. When it is necessary to place a utility through or beneath an MSE wall, it shall be encased in a protective conduit or casing pipe that extends ten feet \([3.0 \text{ m}]\) beyond the limits of the select granular backfill for the MSE wall. Placing pipe culverts through MSE walls should be avoided. Water and sewer lines within ten feet \([3.0 \text{ m}]\) of an MSE wall shall also be encased in a protective conduit or casing pipe.

Utility conduits embedded in concrete should be shown and dimensioned so as to clear construction joints by a minimum of one inch \([25 \text{ mm}]\) and other conduits by a minimum of 2 inches \([50 \text{ mm}]\).

No utilities shall be embedded in the actual vehicular traffic carrying section of a concrete deck.

Utilities should not be suspended below the bottom of the bridge superstructure.

For approval procedures for installation of utilities on bridges, please refer to ODOT’s “Utilities Manual.”

301.7.1 UTILITIES ATTACHED TO BEAMS AND GIRDERS

All utility lines placed between the stringers of grade separation structures should not be located in the floor panel behind the fascia stringer. This is to protect the lines from collisions.

Critical utility lines (gas, etc.) that could contribute to the severity of a collision should be located well above the bottom of the superstructure or be otherwise protected.

If the bridge design is a composite deck on prestressed box beams, the design may either eliminate an interior box beam or provide a space between two interior box beams to provide utility access in this space. This alternative will require a special design for both the boxbeams and the deck.
No utilities shall be placed inside of box beams.

301.8 CONSTRUCTION JOINTS, NEW CONSTRUCTION

Construction joints should be anticipated and provided for in the detail plans. Joint locations should be selected such that they are aesthetically least objectionable, allow construction to be
properly performed and are at locations of minimum stress. Construction joints shall be
designed to transfer all loads.

**302 SUPERSTRUCTURE**

**302.1 GENERAL CONCRETE REQUIREMENTS**

**302.1.1 CONCRETE DESIGN ALLOWABLES**

A. Superstructure Concrete - Class S or HP:

1. Load Factor Design.................................................................4500 psi [31.0 MPa]
2. Service Load Design...... Unit stress = 0.33 x 4500 psi [31.0 MPa] = 1500 psi [10.3 MPa]

B. Substructure Concrete - Class C or HP:

1. Load Factor Design .................................................................4000 psi [27.5 MPa]
2. Service Load Design........ Unit stress = 0.33 x 4000 psi [27.5 MPa] = 1300 psi [9.2 MPa]

C. Drilled Shaft Concrete - Class S Modified:

1. Load Factor Design.................................................................4000 psi [27.5 Mpa]
2. Service Load Design........ Unit stress = 0.33 x 4000 psi [25.7 Mpa] = 1300 psi [9.2 Mpa]

**302.1.2 SUPERSTRUCTURE CONCRETE TYPES**

**302.1.2.1 CLASS S & HP CONCRETE, QC/QA CONCRETE FOR STRUCTURES & CONCRETE WITH WARRANTY**

Class S Concrete is the Department’s traditional concrete mix design for superstructures.

Class HP (High Performance) Concrete mix designs are intended to give a highly dense, very
impermeable concrete resulting in a longer structure life. When Class HP Concrete is specified,
the Designer shall include the bid item for Class HP Concrete Test Slab. However, the bid item
for Class HP Concrete Testing is no longer required because the Department has acquired
sufficient test data since the inception of High Performance Concrete.

QC/QA Concrete for Structures, SS898, is a contractor designed mix that meets minimum
requirements for strength, permeability and air content. QC/QA Concrete is divided into three
classes: substructure (QSC1), superstructure (QSC2) and project specific (QSC3). The
contractor assumes responsibility for quality control sampling and testing. Final payment for in-
place concrete includes incentives for concrete meeting or exceeding minimum requirements and disincentives for concrete not meeting minimums. QC/QA concrete should not be considered for pay items with less than 100 yd$^3$ [75 m$^3$] of concrete.

Class S Concrete for New Bridge Decks with Warranty, SS893, and Class HP Concrete for New Bridge Decks with Warranty, SS894, are standard Class S and HP mix designs that warrant the concrete for a period of seven years against scaling, spalling and cracking. Remedial measures required during the warranty period are to be performed by the original Contractor.

The mix design, curing and placing requirements for both Class S and HP concretes are defined in the CMS.

### 302.1.2.2 SELECTION OF CONCRETE FOR BRIDGE STRUCTURES

The following concrete types may be specified for superstructure concrete:

A. Class S Concrete
B. Class HP Concrete
C. Class S Concrete for New Bridge Decks with Warranty
D. Class HP Concrete for New Bridge Decks with Warranty
E. QC/QA Concrete Class QSC2
F. QC/QA Concrete Class QSC3

The following concrete types may be specified for substructure concrete:

A. Class C Concrete
B. Class HP Concrete
C. QC/QA Concrete Class QSC1

Contact the District to confirm the selection of concrete type to be used for a specific structure.

High performance concrete shall not be used as a replacement for the drilled shaft concrete specified in 524.

### 302.1.3 WEARING SURFACE

#### 302.1.3.1 TYPES

A. 1 inch [25 mm] monolithic concrete - defined as the top one inch [25 mm] of a concrete deck slab. This one inch [25 mm] thickness shall not be considered in the structural design of the deck slab or as part of the composite section.

B. 3 inches [75 mm] asphalt concrete - defined as the minimum asphaltic concrete wearing surface to be used on only non-composite prestressed box beams. The asphalt concrete
wearing surface shall be composed as follows:

1. 1½ inches [38 mm] of Item 448 Asphalt Concrete Surface Course, Type 1H.

2. 1½ inches [38 mm] minimum thickness of Item 448 Asphalt Concrete Intermediate Course, Type 2, PG64-28.

3. Two applications of Item 407 Tack Coat - one prior to placement of the intermediate course and one prior to placement of the surface course. Refer to the ODOT Pavement Design & Rehabilitation Manual, Section 404.11 for application rates.

C. 6 inches [155 mm] cast-in-place composite deck - defined as the minimum thickness of concrete slab for composite prestressed box beams. The top 1 inch [25 mm] shall be considered monolithic as defined above. Also see Section 302.5.1.3.

302.1.3.2 FUTURE WEARING SURFACE

All bridges shall be designed for a future wearing surface (FWS) of 60 psf [2.87 kPa].

The future wearing surface is considered non-structural and shall not be used in design to increase the strength of the superstructure. The presence of a future wearing surface does not exclude the use of the 1 inch [25 mm] monolithic wearing surface as defined above.

302.1.4 CONCRETE DECK PROTECTION

302.1.4.1 TYPES

A. Epoxy Coated Reinforcing Steel - CMS 709.00

B. Minimum concrete cover of 2½ inches [65 mm]

C. Class S Concrete

D. Class HP Concrete

E. Drip Strips

F. CMS 512, Type D, Waterproofing or CMS 512 Type 3 Waterproofing

G. Asphaltic concrete wearing surface
302.1.4.2 WHEN TO USE

All reinforcing steel shall be epoxy coated.
All cast-in-place concrete decks shall have minimum concrete top cover of 2½ inches [65 mm].

A drip strip may be used on decks with over the side drainage.

Non-composite box beam bridges, with over the side drainage, shall have an asphalt concrete overlay. The overlay shall be placed over either Type D Waterproofing, CMS 512 or Type 3 Waterproofing, CMS 512. Minimum thickness of overlay is 3 inches [75 mm] - See Section 302.1.3.1.

302.1.4.3 SEALING OF CONCRETE SURFACES SUPERSTRUCTURE

Specifications for sealing material are defined in CMS 512. Concrete surfaces shall be sealed with an approved concrete sealer as follows: (See Figures 310 & 311)

A. Concrete slabs or concrete decks on steel superstructures with over-the-side drainage:

    The exterior 9 inch [230 mm] width on the top of the deck, the deck fascia and a 6 inch [150 mm] (minimum) width under the deck shall be sealed with either an epoxy-urethane or non-epoxy sealer.

B. Concrete slabs, composite prestressed box beam superstructures or concrete decks on steel superstructures with sidewalks:

    A 9 inch [230 mm] width of the roadway along the curbline; the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the parapet; the deck fascia; and a 6 inch [150 mm] (minimum) width under the deck shall be sealed with either an epoxy-urethane or non-epoxy sealer.

C. Concrete slabs, composite prestressed box beam superstructures or concrete decks on steel superstructures with deflector parapets:

    A 9 inch [230 mm] width of the roadway along the face of parapet; the inside face, top and outside face of parapet; the deck fascia; and a 6 inch [150 mm] (minimum) width under the deck shall be sealed with either an epoxy-urethane, or non-epoxy sealer.

D. Non-composite prestressed concrete box beam decks with over-the-side drainage:

    The fascia of the outside beams and a minimum 6 inch [150 mm] width under the beam shall be sealed with an epoxy-urethane or a non-epoxy sealer.

E. Concrete decks on prestressed I-beam superstructures with over-the-side drainage:

    The exterior 9 inch [230 mm] width on the top of the deck; the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the
bottom flange; and the inside face of the bottom flange shall be sealed with an epoxy-urethane sealer.

F. Concrete decks on prestressed I-beam superstructures with sidewalks:

A 9 inch [230 mm] width of the roadway along the curbline; the vertical face of curb; the top of the curb/sidewalk; the inside face, top and outside face of the parapet; the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the bottom flange; and the inside face of the bottom flange shall be sealed with an epoxy-urethane sealer.

G. Concrete decks on prestressed I-beam superstructures with deflector parapets:

A 9 inch [230 mm] width of the roadway along the face of parapet; the inside face, top and outside face of parapet; the deck fascia; the underside of the deck to the edge of the top flange; the exterior fascia of the beam; the underside of the bottom flange; and the inside face of the bottom flange shall be sealed with either an epoxy-urethane sealer.

Concrete surfaces that include patches should be sealed with an epoxy-urethane sealer so the concrete color will remain uniform.

The designer should include in the plans actual details showing the position, location and area required to be sealed. A plan note should not be used to describe the location as there can be both description and interpretation problems.

The designer has the option to select a specific type of sealer, epoxy-urethane or non-epoxy. The designer may also use a bid item for sealer, with no preference, and allow the contractor to choose based on cost.

Due to poor performance, epoxy-only sealers shall not be used.

In areas where concrete surfaces have a history of graffiti vandalism, the designer may add a sacrificial or permanent graffiti coating meeting the requirements of Supplement 1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 600. The designer should limit the concrete surfaces that are treated with sacrificial or permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

302.2 REINFORCED CONCRETE DECK ON STRINGERS

302.2.1 DECK THICKNESS

Bridge deck concrete thickness shall meet the requirements of AASHTO, this Manual and Standards.
For reinforced concrete decks on steel or concrete stringers the deck thickness shall be computed by the following formula:

\[
T_{min} \text{ (inches)} = \frac{(S + 17)(12)}{36} \approx 8\frac{1}{2} \text{ in}
\]

\[
T_{min} \text{ (mm)} = \frac{(S + 5200)}{36} \approx 215 \text{ mm}
\]
Where $S$ is the effective span length in feet [millimeters]. $T_{\text{min}}$ shall be rounded up to the nearest one-quarter inch [5 mm].

The one inch [25 mm] wearing thickness, Section 302.1.3.1, is included in the calculations for minimum concrete deck thickness but not in the calculations during actual structural design of the deck slab.

For transversely reinforced concrete deck slabs supported on steel stringers the effective span length "$S$" shall be considered equal to the distance center-to-center of stringers minus 6 inches [150 mm].

For concrete I-beam stringers the effective span length shall meet the requirements of AASHTO 3.24.1.2.

302.2.2 CONCRETE DECK DESIGN

The concrete deck design shall be in conformance with AASHTO, latest edition, and additional requirements in this Manual. The design live load shall be HS25 for decks on new superstructures and HS20 for decks on existing superstructures.

For continuous slabs on three or more supports a continuity factor of 0.80 shall be applied to the simple span bending moments for both live load and dead load.

See Figures 312 & 313 for an illustration of a method of design for a reinforced concrete deck slab. Design data tables for HS25 (Fig. 314A) and HS20-44 (Fig. 314B) live loads are also provided.

Upon completing the concrete deck design from the example shown in Figure 312 & 313, or similar method, the designer should assure any cantilevered deck overhang will not over stress the initial deck design due to the dead load and the greater live load of either the vehicle wheel loads or the railing live loads. See relevant AASHTO sections for live load application requirements. See example Figures 315 & 316.

Transverse spacing of the top and bottom reinforcing in a deck design shall meet section 302.2.4.2.

302.2.3 DECK ELEVATION REQUIREMENTS

302.2.3.1 SCREED ELEVATIONS

Screed elevations are control elevations for concrete deck finishing machines that account for dead load deflections to ensure that the bridge deck is completed to the correct elevation. To establish screed elevations, the final surface elevations are adjusted for non-composite
deflections resulting from deck placement and composite deflections resulting from utility and railing loads. Screed elevations shall not include adjustment for deflections due to the future wearing surface loading. Calculated deflections caused by the weight of the deck concrete should assume a completed placement sequence. Use deflection data from girder lines closest to each screed line to determine elevations. Refer to Figure 335.

If the deflections are determined through a line girder analysis method, the deck load should be distributed evenly to all beams/girders loaded in each construction phase to establish screed elevations. If a refined analysis method is used, establish screed elevations using the individual beam/girder deflections.

The bridge plans shall include a screed elevations table. The locations of all screed elevations in the table should be identified on a transverse section and plan view. Elevations should be provided for all: curblines or deck edges; profile grade points; transverse grade-break lines; and phased construction lines for the full length of the bridge. Screed elevations are not required above beam/girder lines. Bearing points, quarter-span points, mid-span points and splice points shall be detailed as well as any additional points required to meet a maximum spacing between points of 25'-0" [7.5 m].

For bridges with a separate wearing course, the elevations given should be those at the top of the portland cement concrete deck. Provide a plan note stating at what surface the elevations are given in order to eliminate any confusion.

Screed elevations are not required for non-composite box beam bridges or slab bridges. Screed elevations for composite box beam bridges shall meet the same requirements as steel beam, girder and prestressed I-beam bridges.

302.2.3.2 TOP OF HAUNCH ELEVATIONS

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed. Top of haunch elevations should be provided at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25'-0" [7.5 m]. The top of haunch elevation locations should be identified in a plan view and on the transverse section. Top of haunch elevations are not required for composite box beam bridges. Provide a plan note for a definition and description of the purpose for the top of haunch elevations (see BDM Section 700). Refer to Figure 335.

302.2.3.3 FINAL DECK SURFACE ELEVATIONS

Final deck surface elevations represent the position of the deck after all dead loads except future wearing surface have been applied. Final deck surface elevations shall be provided at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum
spacing between points of 25'-0” [7.5 m] for each: girder centerline; curbline or deck edge; transverse grade-break line; and phased construction line. The final deck surface elevation locations should be identified in a plan view. Refer to Figure 335.

302.2.4 REINFORCEMENT

302.2.4.1 LONGITUDINAL

Distribution reinforcement in the top-reinforcing layer of a reinforced concrete deck on steel or concrete stringers shall be approximately 1/3 of the main reinforcement, uniformly spaced.

Research has shown that secondary bars in the top mat of reinforced concrete bridge decks on stringers should be small bars at close spacing. Therefore the required secondary bar size shall be a #4 [#13M]. The only exception to this requirement is if the bar spacing becomes less than 3 inches [75 mm].

For stringer type bridges with reinforced concrete decks, the secondary bars shall be placed above the top of deck primary bars. This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

For reinforced concrete deck slabs on stringer type bridges, where the main reinforcement is transverse to the stringers, additional top longitudinal reinforcement shall be provided in the negative moment region over the piers. This additional secondary reinforcement shall be equal to the distributional reinforcement (1/3 of the main reinforcement). This additional reinforcement shall be uniformly spaced and furnished in length equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of AASHTO 8.24.3.3.
This reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of AASHTO.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

For steel beam or girder bridges with a skew of less than 15 degrees the transverse reinforcing may be shown placed parallel to the abutments. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. For steel beam and girder structures, the haunch shall be tapered back to the original concrete deck thickness in a 9 inch [225 mm] length and the concrete haunch shall encase the edges of the top flange. See Figures 317 & 318.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

Mechanized finishing machines are preferred to hand finishing methods for both consistency of
surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

### 302.2.7.1 FINISHING MACHINES

Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called “bogies,” which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a “carriage,” which levels, compacts, vibrates and finishes the concrete.

Finishing machines can be placed such that the truss sections are skewed with respect to the screed rails. This orientation allows for concrete placement parallel to the substructure skew as required by the C&MS 511. For skew angles of 15° and greater, the finishing machine can be skewed to within 5 degrees of the plan specified skew angle.

The carriage can also be skewed with respect to the truss sections. This feature allows the carriage to finish the concrete transverse to the bridge when the truss sections are placed at some other orientation (e.g. parallel to the substructure skew). In order to ensure a proper finish at transverse grade breaks (e.g. crown points), the carriage should always be oriented to finish the concrete transverse to the bridge. A special length truss section insert is required above the grade break locations such that the grade break line lies directly below opposite corners of the section.
For skewed bridges without transverse grade breaks, skewing the carriage with respect to the truss sections is not required.

Most finishing machines do not easily accommodate non-parallel rails. The distance between the screed rails should be a fixed width. Designs that require tapered paving widths should be avoided.

The finishing machines can be hinged at the pin connections between truss sections in order to provide transverse grade breaks (e.g. crown points). In theory, multiple transverse grade breaks can be accommodated, but the grade breaks must remain at a fixed spacing in order to line up with a pin connection. The figure below illustrates the complexity of the machine set-up to accommodate multiple grade breaks in a transverse section placed on a skew. Note that the length of truss sections required between grade breaks must fit the standard truss section lengths.
Grade break locations that move laterally along the length of the bridge cannot be paved in a single operation using a mechanized finishing machine and should therefore be avoided. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See the figure below.

302.2.7.2 SOURCES OF GIRDER TWIST

The interconnectivity between girders, intermediate crossframes/diaphragms and end crossframes/diaphragms is essential to a structure’s stability throughout the construction process. Therefore, it is of utmost importance to ensure that all crossframes/diaphragms are fully installed.
prior to deck placement. Failure to do so may lead to construction disputes, expensive repairs and lengthy construction delays or even impact project safety. One major drawback to this interconnectivity is that the deflection caused by the placement of the concrete deck will result in girder twisting.

There are primarily three independent sources of girder twist resulting from deck placement. This manual will refer to these sources as: global superstructure distortion, oil-canning and girder warping.

302.2.7.2.a **GLOBAL SUPERSTRUCTURE DISTORTION**

Global superstructure distortion is distortion of the bridge transverse section primarily caused by differential deflections between adjacent girders. As a girder deflects downward with respect to an adjacent girder, the rigidity of the cross framing between the two girders causes the deflecting girder to rotate as it deflects. This distortion may occur with both steel and prestressed concrete superstructures. The most common differential deflections occur between the exterior girders and adjacent interior girders for a given construction phase when the loaded tributary areas over the girders differ.

Transverse sections with more heavily loaded exterior girders distort in a convex shape.
Transverse sections with more heavily loaded interior girders distort in a concave shape.

Twisting of the exterior girders can result in deck thickness and cover loss if the screed rails are supported on cantilevered falsework. The magnitude of girder twist (measured as $N_g$) will vary over the length of the bridge and will be different for the left and right sides if loading or geometry is not symmetrical.

For bridges with tangent alignments and adjacent substructure skews that vary by no more than 15°, the magnitude of the girder twist can be reduced by utilizing transverse sections with balanced tributary deck loadings. For a new superstructure, the amount of girder twist due to global superstructure deformation can be neglected when the tributary deck load carried by the fascia girder does not exceed 110% of the average of the tributary deck load carried by the interior members for a given construction phase. For an existing superstructure, the amount of global deformation may be neglected when the tributary deck load carried by the fascia girder does not exceed 115% of the average of the tributary deck load carried by the interior members for a given construction phase.
When the aforementioned tributary deck loading requirements of the fascia members cannot be met or, because of geometry, do not apply, the Designer shall perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist ($N_g$) due to deck concrete placement. To properly calculate the effect of the twist angle on deck thickness, the analysis should be based on the deflection occurring due to the concrete present at the time that the finishing machine passes over the point under consideration. This degree of precision requires a separate refined analysis for each point of consideration. It is generally sufficient to calculate $N_g$ based on the full wet concrete load placed over the entire structure. However, on complex structures with variable skews and/or curved girders, a higher degree of precision may be warranted to ensure proper deck thickness.

Additional measures to reduce global deformation include: adding or stiffening the crossframes/diaphragms; and increasing the stiffness of the girders. An increase in the crossframe stiffness results in better load distribution across the width of the structure and less distortion. An increase in the stiffness of the girders reduces the magnitude of vertical deflection resulting in less distortion of the transverse section.
302.2.7.2.b  OIL-CANNING

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web.

Locating the falsework bracket near the bottom flange will reduce the amount of web deformation. C&MS Item 508 requires the lower point of contact to be within 8” of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist ($N_o$) resulting from oil-canning may be neglected for girder webs 84” deep or less.

For web depths greater than 84”, designers shall provide the location of the falsework bracket in the plans. Provide a General Note that removes the lower point of contact requirement of C&MS Item 508 (see BDM Section 600 for an example). The pay item for deck concrete shall be “as per plan”. Using the plan bracket location, designers shall determine $N_o$. Designers may assume the lowest location of the falsework bracket to be 76” measured below the bottom of the top flange. The magnitude of twist can be predicted using finite element analysis of the web or by various approximate methods. If the magnitude results in excessive deck thickness loss, reducing the transverse stiffener spacing or adding temporary bracing on the inside of the web may be necessary. Any temporary bracing should be detailed in the plans.

The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.
302.7.2.7.c GIRDER WARPING

Distortion due to girder warping occurs as a result of deck slab overhang falsework loading on the fascia girder between points of lateral bracing (e.g. crossframes). The bracket loads produce twist between the crossframes due to a combination of girder warping and pure torsional distortion. The girder is restrained from warping at the crossframe locations. Due to the inherent torsional stiffness of prestressed I-beams, the distortion due to girder warping may be neglected. Other design considerations for I-beams due to the overhang bracket loadings are presented at the end of this section.

For steel superstructures, Designers should calculate the magnitude of twist \( N_w \) due girder warping using the TAEG software developed by the Kansas Department of Transportation. TAEG (“Torsional Analysis of Exterior Girders”) is available at no cost and can be downloaded at: [http://www.ksdot.org/kart/](http://www.ksdot.org/kart/).

Since most of the data input in TAEG is dependent upon the contractor’s equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources. For design-build projects and value engineering change proposals (VECP’s), data input for TAEG shall represent the actual falsework and equipment to be used by the contractor. Designers may use the following assumptions in lieu of actual contractor supplied information:

A. Girder Data:

   For bridges with constant web depths, designers may select the cross section with the least torsional resistance to represent the entire structure. For bridges with variable depth webs, designers may disregard the effect of girder warping in the web depth transition sections.

B. Bridge Lateral Data:

   Designers may select the largest crossframe spacing to represent the entire structure. For structures with variable beam spacings (i.e. flared girders) designers may select the largest
spacing dimension to represent the entire structure. Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

C. Permanent Lateral Support Data:

The default crossframe type assumed by the TAEG software consists of a stiffener and diagonal x-bracing with top and bottom horizontal chords. In order to analyze the structure with a standard ODOT crossframe, designers should input stiffener dimensions and select the “Diaphragms (Inputted Ix)” option. For ODOT Type 1 crossframes, designers should assume a fictitious stiffener of dimensions: 5” x 3/8”. Determine the diaphragm moment of inertia for all standard ODOT crossframes as follows:

\[
I_x = \frac{h^2 s}{4L_d^2 \left( \frac{1}{A_d L_h^2} + \frac{L_h}{A_h L_d^2 + A_d L_h^3} \right)}
\]

Where:

- \( A_d \) = Area of the diagonal member (in\(^2\))
- \( A_h \) = Area of the horizontal member (in\(^2\))
- \( L_d = \sqrt{L_h^2 + h^2} \)

D. Temporary Lateral Support Data:

Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

E. Load Data:

1. Live Load on Walkway..........................................................50 lb/ft\(^2\)
2. Live Load on Slab..............................................................50 lb/ft\(^2\)
3. Dead Load of Formwork.........................................................10 lb/ft\(^2\)
4. Dead Load of Concrete ..................................................150(t\(_{avg}\)) lb/ft\(^2\)
   (t\(_{avg}\) = Average thickness [ft.] of deck slab overhang)
5. Wheel Spacing [1-2-3]........................................................36” – 31” – 36”
6. Maximum Wheel Load:

   To estimate the total finishing machine length required for placement along the skew, add the rail-to-rail length and the extra end length from the following table using the plan specified skew rounded to the nearest 5 degrees. W is the rail-to-rail length as measured perpendicular to the centerline of the bridge.
<table>
<thead>
<tr>
<th>Skew Angle</th>
<th>Rail-to-Rail Length, ft.</th>
<th>Extra End Length, ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00 W</td>
<td>0.0</td>
</tr>
<tr>
<td>15</td>
<td>1.04 W</td>
<td>5.0</td>
</tr>
<tr>
<td>20</td>
<td>1.06 W</td>
<td>5.5</td>
</tr>
<tr>
<td>25</td>
<td>1.10 W</td>
<td>6.5</td>
</tr>
<tr>
<td>30</td>
<td>1.15 W</td>
<td>7.0</td>
</tr>
<tr>
<td>35</td>
<td>1.22 W</td>
<td>8.0</td>
</tr>
<tr>
<td>40</td>
<td>1.31 W</td>
<td>9.0</td>
</tr>
<tr>
<td>45</td>
<td>1.41 W</td>
<td>10.5</td>
</tr>
<tr>
<td>50</td>
<td>1.56 W</td>
<td>11.5</td>
</tr>
<tr>
<td>55</td>
<td>1.74 W</td>
<td>13.5</td>
</tr>
</tbody>
</table>

For total machine lengths of 36 ft. and less, assume a total machine weight of 7.6 kip. Add 0.09 kip for each additional foot of machine length required above 36 ft. The maximum total machine length shall not exceed 120 ft. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

To determine the maximum wheel load, divide the total machine weight by 8.0.

F. Bracket Data:

1. Refer to the following figure to determine TAEG dimensions A, B, C, D, E, F and G.
2. Designers may assume a center-to-center bracket spacing of 48.0 in.
3. Designers may assume a bracket weight of 50 lbs.
Assumptions for TAEG Bracket Data Input
For prestressed I-beam superstructures, Designers should verify that the intermediate crossframes/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework.

302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST

Once all sources of girder twist are quantified, Designers should determine the total effect that girder twist has on the finished deck surface. The primary effect of greatest concern is the loss of concrete cover over the top mat of deck reinforcing steel and the subsequent loss of deck thickness. The maximum loss due to twisting shall not exceed 0.5 in.

The total amount of girder twist at both the left and right screed rail should be determined as follows:

\[ \phi_{\text{left}} = (\phi_g + \phi_o + \phi_w)_{\text{left}} \quad \text{and} \quad \phi_{\text{right}} = (\phi_g + \phi_o + \phi_w)_{\text{right}} \]

where:

- \( \phi_g \) = Girder twist due to global superstructure distortion (See BDM Section 302.2.7.2.a)
- \( \phi_o \) = Girder twist due to “oil-canning” (See BDM Section 302.2.7.2.b)
- \( \phi_w \) = Girder twist due to girder warping (See BDM Section 302.2.7.2.c)
The total amount of screed rail deflection at both the left and right screed rail should be determined as follows:

\[
\delta_{\text{left}} = \tan(\phi_{\text{left}}) \times L_b \quad \text{and} \quad \delta_{\text{right}} = \tan(\phi_{\text{right}}) \times L_b
\]

where:

- \( \phi_{\text{left}} \) and \( \phi_{\text{right}} \) = Deflection of the screed rail due to total girder twist (in.). Upward deflection is positive and downward deflection is negative.
- \( L_b \) = Lateral distance from center of screed rail to centerline of fascia girder (in.)

The total loss of deck thickness should be determined as follows:

\[
\delta_{\text{Total}} = (\delta_{\text{left}} + \delta_{\text{right}})/2
\]

### 302.2.8 SLAB DEPTH OF CURVED BRIDGES

For a curved deck on straight steel beams, steel girders or prestressed I-beams, the distance from the top of the slab to the top of the beams or girders will vary from end to end. The slab depth dimension shall show this variation by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

### 302.2.9 STAGED CONSTRUCTION

For all bridge types, except non-composite concrete box beams, where the differential dead load deflection between adjacent beams, girders or structural slabs is greater than ¼ inch [6 mm], a deck closure is required if the bridge is constructed in stages.

For requirements regarding closure pours on bridge widenings or on existing structures with new concrete decks see Section 400 of this Manual.

The closure pour between the stages shall be a minimum width of 30 inches [800 mm] but should be wide enough to accommodate the required reinforcing steel lap splices. In special cases, this distance may be reduced when mechanical reinforcing steel connectors are used (see...
Section 200). The mechanical connector system used shall be able to develop 125 percent of the full yield strength of the reinforcing steel as a minimum.

Intermediate cross frames and diaphragms shall not be permanently attached in the closure pour location until the concrete pours on both sides of the closure pour location have been completed.

The two construction joints created by the concrete closure pour should be sealed with High Molecular Weight Methacrylate (HMWM), 705.15. The sealing width shown in the plans should be 2'-0" [600 mm], centered on the construction joints.

Placement of the staged construction joints above beam flanges is not recommended. The preferred location is the positive moment regions of the cast-in-place concrete deck slab.

The designer shall provide plan notes on the stage construction details sheet that detail the
sequence of construction.

302.3 CONTINUOUS OR SINGLE SPAN CONCRETE SLAB BRIDGES

302.3.1 DESIGN REQUIREMENTS

Continuous reinforced concrete slab bridge design shall be in conformance with AASHTO, latest edition, and additional requirements in this Manual.

For simple span reinforced concrete slab bridges cast in place directly on concrete substructures, the effective span length shall be considered equal to the clear span plus 15" [380 mm].

The Designer shall include a final deck surface elevation table. Elevations shall be shown for all profile grade lines, curblines, crownlines, and phased construction lines for the full length of the bridge. Bearing points, quarter-span points and mid-span points shall be detailed as well as any additional points required to meet a maximum spacing between points of 25'-0".

Details for simple span reinforced concrete slab bridge superstructures are provided in Standard Bridge Drawing SB-1-03.

Details for multi-span reinforced concrete slab bridge superstructures are provided in Standard Bridge Drawing CS-1-03.

302.4 STRUCTURAL STEEL

302.4.1 GENERAL

Structural steel shall be designed utilizing a composite section. A non-composite design may be used only if the design is the most economical.

Designs incorporating shear connectors in the negative moment region may be used.

All curved beams or girders shall be designed in accordance with AASHTO, this Manual and the latest AASHTO Guide Specifications for Horizontally Curved Highway Bridges including all interims.

The laterally unsupported length of top flanges of beam and girder members with a concrete deck encasing the top flange or compositely designed with studs shall be considered to be zero. In the absence of such fastening or direct contact of an individual beam or girder member, the unsupported length shall be considered as the distance between the diaphragms, struts, bridging, or other bracing.

For designs that assume the unbraced length of the top flange to be zero as mentioned above, the designer shall investigate the strength of the non-composite section during steel erection, deck
slab construction, etc. using laterally unsupported lengths that reflect actual bracing conditions.

302.4.1.1 MATERIAL REQUIREMENTS

Types of steel to be selected for use in the design and construction of bridges is as follows:

A. ASTM A709[M] grade 50W shall be specified for an un-coated weathering steel bridge.

B. ASTM A709[M] grade 50 shall be specified for a coated steel bridge.

C. ASTM A709[M] grade 36 is not recommended and is being discontinued by the steel mills.

D. High Performance Steel (HPS), A709[M] grade 70W, un-coated weathering steel is most economical when used in the flanges of hybrid girders. Consult the Office of Structural Engineering for recommendations prior to specifying its use. A plan note is provided in the appendix.

There are several systems available for coating steel bridges. These coatings are specified in CMS 514 or by the plan notes provided in the appendix. See Section 302.4.1.5 for guidelines on selecting a coating system.

302.4.1.2 ATTACHMENTS

Detail plans of steel beam and girder bridges shall show where welded attachments are allowed for construction purposes.

Welding of attachments, either permanent or temporary, is not acceptable in tension areas. Welding is allowed in compression areas. Detail plans shall show the extent of compression and tension areas.

Welding of scuppers, down spouts or drainage supports should not be allowed in tension areas of main members.

302.4.1.3 STEEL FABRICATION QUALIFICATION

The Department’s requirements for steel fabricators are defined in CMS 513 and Supplement 1078. Steel fabricators are classified according to their capabilities into eight levels (1 thru 6, SF & UF). Levels 1 thru 6 require certification according to the American Institute of Steel Construction (AISC). No AISC certification is required for Levels SF and UF.

The AISC categories of certification are listed here for information:

A. AISC Category Sbr - Fabricators qualified for single span rolled beam bridges
B. AISC Category Mbr - Fabricators qualified for all other bridge structures

C. AISC has also established a P and F endorsement for fabricators:
   1. P - Painting of steel structures endorsement
   2. F - Fracture Critical endorsement

A plan note is available in Section 600 for Level UF work items on existing structures requiring field fabrication.

302.4.1.4 MAXIMUM AVAILABLE LENGTH OF STEEL MEMBER

Mills can supply lengths up to maximum shipping limits, but extra charges may be added for lengths over 80'-0" [24 meters]. The designer should consider cost by providing for field splices and allowing for optional field splices. The National Steel Bridge Alliance (NSBA) and the American Iron and Steel Institute (AISI) are available to provide assistance with material sizes.

Length of a girder is generally limited by the ability to transport the member from the fabricator's shop to the job site. A length of 120'-0" [36 meters] is generally the maximum trucking length between splices, but girder lengths of 160'-0" [49 meters] and greater have been transported to project sites.

302.4.1.5 STRUCTURAL STEEL COATINGS

This section shall serve as a guide in selecting corrosion control systems.

302.4.1.5.a PRIMARY COATING SYSTEMS

The Department’s primary system is un-coated weathering steel. Weathering steel reduces the initial cost of the structure by approximately 3 dollars per square foot [30 dollars per square meter] of steel surface area. It may also eliminate future maintenance coatings. However, weathering steel structures should be monitored on a five year cycle to determine if section loss justifies a partial or total structure maintenance coating. Contact the Office of Structural Engineering for further guidance on field monitoring.

If a site-specific study finds that un-coated weathering steel should not be used, specify a coated non-weathering grade of steel.

Site-specific studies should consider the following:

A. Salt usage tons per lane mile
B. Site condition potential for tunnel like conditions and salt spray from under passing traffic. These are structures or zones that receive deposits of salt and high humidity or long term wet conditions.

Additional resources available to aid in the site studies include:

C. NCHRP 314, 1989
D. AISC Uncoated Weathering Steel Bridges, Vol. 1, Chap. 9
E. Ohio Department of Transportation, unpublished internal study, September 2000.
F. Texas Department of Transportation, Research report 1818-1, May 2000.
G. Missouri Department of Transportation, Task Force Report on Weathering Steel

Un-coated weathering steel shall have a protective coating applied to a 10'-0" [3 meter] length of beam or girder adjacent to abutments with expansion joints and on both sides of intermediate expansion joints. All cross frames, end frames or other steel in this 10'-0" [3 meter] section shall also be coated. The top coat shall be tinted to a color closely matching Federal Standard No. 595B-20045 or 20059, the color of weathering steel.

Un-coated bridges with integral or semi-integral abutments shall not have this protective coating.

Aesthetics may dictate coating the fascia lines of an otherwise un-coated weathering steel bridge. If this treatment is selected, the use of darker fascia colors (forest green, medium to dark browns, dark blues, rustic reds) may provide a more homogenous look than the light neutral colors.

New steel structures, including the aforementioned ends of weathering steel beams and girders, shall receive a three-coat paint system consisting of an inorganic zinc prime coat, an epoxy intermediate coat and a urethane finish coat (formerly called system IZEU). The top coat shall be tinted to a color closely matching Federal Standard No. 595B-20045 or 20059, the color of weathering steel. The inorganic prime coat is shop applied while the intermediate and top coats are field applied. This system has proven to have a life span of up to 30 years.

302.4.1.5.b ALTERNATIVE COATING SYSTEMS

Special conditions or member size may warrant the use of other coating systems.
An IZEU three coat shop applied system with field touch up may be the system of choice where:

A. There is limited access to the superstructure in the field. Examples may include stream crossings with shallow clearances.

B. The environment is especially sensitive to possible construction debris.

C. The bridge is located in a highly urbanized area that may have limited access for future coatings.

The IZEU three coat shop applied system may provide a better protective coating than the standard IZEU system due to the fabricator’s automated blasting processes, environmental controls and better coating application access. However, the total cost of the shop-applied system is higher than the standard IZEU system. Extra costs include special care needed during shipping and erection, field painting of field splices, field touch up, final cleaning and possible time delays to the project due to the additional shop work. Additionally, all field connections are to be bolted to minimize the damage to the coating by field welding.

Required plan or proposal notes for the IZEU three-coat shop applied systems are located in the appendix.

A galvanized coating system is an alternative for new steel when the requirements addressed in Section 302.4.2.1 are met.

Galvanized systems are proven durable up to 40 years. Additional information is also available from the American Galvanizers Association.

Required plan or proposal notes for the galvanized coating system are located in the appendix.

A shop applied metallizing system is another alternative coating system, but currently the costs are relatively high compared to the standard IZEU system.

Metallized systems have an expected life similar to galvanizing. Metallized coating systems should have field bolted connections rather than field welded connections, but oversized holes are not required. Metallizing, as compared to galvanizing, has no limit on the size of members being coated and causes no additional distortion from heat.

Required plan or proposal notes for the metallized coating system are located in the appendix.

302.4.1.5.c EXISTING STEEL COATING SYSTEMS

If the total existing structure is to be field painted, the coating should be a three-coat paint system consisting of an organic zinc prime coat, an epoxy intermediate coat and a urethane finish coat (formerly referred to as system OZEU) according to CMS 514. The OZEU system is better for
field application on existing steel since the organic zinc prime coat is more surface-tolerant.

For widened structures, the new steel shall be coated with the inorganic zinc prime coat in the shop and the existing steel shall receive the organic prime coat. The intermediate and finish coats are the same for each system.

Field metallizing is another option but its current costs are more than twice the cost of OZEU.

When estimating the quantity for Item 514, Grinding Fins, Tears, Slivers on Existing Structural Steel, provide 1 minute for each linear foot of beam/girder to be coated.

302.4.1.6 STEEL PIER CAP

Steel pier caps are non-redundant, fracture critical members. As specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering. In general, structure designs that require stringers to be continuous through, and in the same plane with a steel pier cap or cross beam, should be avoided if at all possible.

302.4.1.7 OUTSIDE MEMBER CONSIDERATIONS

The designer is to evaluate the actual loads for outside main members. Heavy sidewalks, large overhangs of the concrete deck slab and/or live loads may cause higher loads on an outside member than loads on an internal member. This analysis requirement does not alleviate the designer from conforming to AASHTO Section 3.23.2.3.1.4.

In order to facilitate forming, deck slab overhang should not exceed 4'-0" [1200 mm]. On over the side drainage structures the minimum overhang shall be 2'-3" [700 mm]. Where scuppers are required for bridge deck drainage the overhang shall be 1'-6" [450 mm].

302.4.1.8 CAMBER AND DEFLECTIONS

When establishing dead load deflection for determining the required shop camber of non-composite steel beam or girder bridges with concrete deck slabs and determining deck screed elevations, the weight of curbs, railings, parapets and separate wearing surface, may be equally distributed to all beams. Future wearing surfaces shall not be included in determining required camber. This weight may be assumed (for dead load deflection only) to be supported by the beams acting compositely, based on a moment of inertia approximately twice that of the beam. Therefore, deflection due to dead loads above the deck slab may be based on one-half of the weight distributed to each beam, using the beam moment of inertia.

When establishing dead load deflection for determining the required shop camber of composite beam or girder bridges with concrete deck slabs and determining deck screed elevations, the
weight of curbs, railings, parapets and separate wearing surface may be equally distributed to all beams. Future wearing surfaces shall not be included in determining required camber.
The deflection and camber table in the design plans shall detail all points for each beam or girder line for the full length of the bridge. Bearing points, quarter-span points, mid-span points and splice points shall be detailed and any additional points required to meet a maximum spacing between points of 25'-0".

In cases of special geometry, i.e. spirals, horizontal or vertical curves, superelevation transitions, etc., additional points are to be detailed in the deflection and camber table if the normally required points do not adequately define a beam or girder required curvature.

The required shop camber shall in all cases be the algebraic sum of the computed deflections, vertical curve adjustment, horizontal curve adjustment and adjustment due to heat curving. Camber shall be measured to a chord between adjacent bearing points.

A camber diagram shall be provided showing the location of the points developed above and giving vertical offset dimensions at the bearing points from a “Base” or “Work” line between abutment bearings.

302.4.1.9  FATIGUE

The following paragraphs are intended to clarify the application of the AASHTO Section 10.3 regarding fatigue stresses.

For allowable fatigue stresses, reference shall be made to the AASHTO specifications.

302.4.1.9.a  LOADING

In applying loads for fatigue stresses, a single lane of traffic shall be used and positioned to produce maximum stress ranges in the member under consideration. The design loading shall be HS20 [MS18] for all structures.

In computing live load stress ranges for fatigue stresses in structures with concrete decks supported on steel beams, a distribution fraction of S/7 shall be used.

To establish the Case of loading for a structure, according to AASHTO Section 10.3.2, an estimated Average Daily Truck Traffic shall be determined for the Design Year. Consideration shall be given to the potential traffic volumes of the proposed roadway as a result of future industrial or commercial development.

For steel beam bridges designed for Case I loading, the intermediate cross frames shall be connected to the stringers by the use of plate stiffeners shop welded to the stringer webs and flanges.
302.4.1.9.b  STRESS CATEGORY

In order to allow for future rehabilitation involving welded attachments, steel member designs in the negative moment regions should be limited to an allowable fatigue stress range of Category C, even though shop or field welded attachments are avoided in the original design.

302.4.1.10  TOUGHNESS TESTS

On steel structures, main load carrying members, such as beams, moment plates, bolted joint splice plates (excluding fill plates) require Charpy V Notch Testing. These components shall be identified on the detail plans by placing “(CVN)” after the component's description.

Example:  W36x150 (CVN)  [W920 x 223 (CVN)]

The web and all flanges of plate girders shall be CVN material.

Cross frame members, cross frame connection stiffeners and any steel connecting these elements on horizontally curved beam or girder structures are considered main members and shall require and be identified on the detail plans as CVN.

302.4.1.11  STANDARD END CROSS FRAMES

End cross frames for needed support and reduction of deflection of expansion devices should be designed to provide support at intervals not exceeding 4'-0" [1200 mm]. Standard expansion joints have designs already established as part of the standard drawings. For suggested details of special conditions review existing expansion joint Standard Bridge Drawings.

302.4.1.12  BASELINE REQUIREMENTS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES

CMS 513 requires the fabricator to include in the shop drawings an overall layout with dimensions showing the horizontal position of beam or girder segments with respect to a full-length base or workline. Offsets from this full-length base line are to be provided by the fabricator for each 10 feet [3000 mm] of length. The designer shall provide this baseline in the plans along with enough information for the fabricator to be able to readily calculate the required offsets. The requirement for this information is especially critical on structures located on a curve or spiral or having other complex geometry.

302.4.1.13  INTERMEDIATE EXPANSION DEVICES

Intermediate expansion devices for a structure, if required, shall be located over a pier and the structural members shall be designed to be discontinuous at that pier.
302.4.1.14  BOLTED SPLICES

Bolted splices for rolled beams are detailed in a Standard Bridge Drawing. The standard incorporates Load Factor designed beam splices for A709[M] grade 36, 50 and 50W steel materials. The designer is required to confirm that the capacity of the standard splice is greater than the actual loads for the designer's structure.

For galvanized structures the designer should not specify standard drawing splices. The bolt hole size requires a 1/16 inch [1.5 mm] increase over the standard drawing’s hole size to allow for the additional thickness of the zinc coating. This increase in hole size decreases the standard drawing’s splice capacity. The designer should either evaluate the standard drawing splice based on the decreased capacity or design a new splice.

Bolt allowable stresses for painted surfaces or unpainted weathering steel surfaces shall be based on AASHTO’s values for Class A, Contact Surface, Standard Hole Type.

Bolt allowable stressed for metallized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Standard Hole Type.

Bolt allowable stressed for galvanized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Oversized Hole Type.

Beams having bolted splices at bend points shall have additional details incorporated in the plans to completely detail the joint requirements. The minimum edge distances specified in AASHTO shall be provided at the edges of all main members and splice plates.

For splices at bend points the lines of holes in the beam or girder flanges should be parallel to the centerline of the web. If the bend angle is small enough use rectangular splice plates (splice plates should not overhang flange by more than ½ inch [13 mm] and inside splice plates should not have to be trimmed to clear web or web to flange radius). When the angle is too large to allow rectangular splice plates the plates should be trimmed to align with the flange edges. In either case minimum edge distances shall be met.

Bolted compression splices, such as in a column, while designed as a friction type connection, also require the ends of the spliced members to be in full bearing by milling of the ends. For compression splice members with milled ends the AASHTO requirements of Section 10.18.3.1 shall be met.

The designer should recognize that “FULL BEARING” of beams and girders is not defined by AASHTO. “FULL BEARING” has been generally defined by ODOT as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32 inch [0.8 mm]. The designer shall specify the required fit definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

Refer to Figure 302.4.1.14-1 for additional bolted splice details.
302.4.14.a  BOLTS

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A325[M].

The designer shall specify the diameter of the bolts and check that the type (Type I for Galvanized or Type III for Weathering) of A325[M] bolts is described in the coating notes or bolt material specifications.

Coating systems that are zinc based, such as OZEU, IZEU, Galvanizing or Metallizing require galvanized Type I bolts.

Un-coated weathering steel structures shall have A325[M], Type III bolts. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized A325[M] Type I bolts. Otherwise, specify A325[M], Type III bolts.

Generally, bolted splices should be designed using 1 inch [25 mm] or 1½ inch [29 mm] diameter bolts. No metric bolts or studs are available in the small quantities required for bridges.

The use of A490[M] bolts is not permitted.

302.4.14.b  EDGE DISTANCES

1" [25 mm] diameter bolts used in splice plates should be detailed to allow for 2" [50 mm] edge distances in lieu of the AASHTO requirements. 1½ inch [29 mm] diameter bolts used in splice plates should be detailed to allow for 2¼ inch [60 mm] edge distances in lieu of the AASHTO requirements.

This increase to AASHTO's edge distances is to help alleviate the problem fabricators have of drilling bolt holes in flange splice plates and maintaining required minimum edge distances, especially on the inside splice plates.

If larger diameter bolts are specified the designer shall add ¼ inch [6 mm] to the AASHTO minimum edge distance.

302.4.14.c  LOCATION OF FIELD SPLICES

Generally bolted splices should be located at points of dead load contraflecture on a continuous structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.

302.4.15  SHEAR CONNECTORS

AASHTO Sections 10.38.2.3 and 10.38.2.4 on studs shall be followed.
Shear studs shall be automatic welded studs. The use of channel sections is not allowed. 7/8 inch [22 mm] diameter studs are recommended as a standard diameter. The length of stud specified should be checked with manufacturers as to availability.

The Department’s policy of using a 2 inch [50 mm] deep haunch over the top flange will have an effect on the length of shear studs.

Shear studs shall be field installed. In the case of galvanized structures, the design plans shall allow shop installation of studs prior to galvanizing or field installation after removing the coating by grinding at each stud location. If the studs are shop installed, the Contractor will be responsible for meeting all applicable OSHA requirements. A Detail note is available in Section 700.

### 302.4.2 ROLLED BEAMS

Effective in January 2006, the producers of rolled beams implemented changes to the physical dimensions of the W36X16 group of shapes (i.e. beams with 16” and wider flanges). The traditional W36X16 series of shape sizes will no longer be available from the producers. Below is a complete list for the new W36X16 group of shapes.

<table>
<thead>
<tr>
<th>Designation</th>
<th>Area, $A$ (in$^2$)</th>
<th>Depth, $d$ (in)</th>
<th>Flange Width, $b_f$ (in)</th>
<th>Flange Thickness, $t_f$ (in)</th>
<th>Web Thickness, $t_w$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W36 X 800</td>
<td>236.4</td>
<td>42.55</td>
<td>17.990</td>
<td>4.290</td>
<td>2.380</td>
</tr>
<tr>
<td>W36 X 652</td>
<td>192.5</td>
<td>41.05</td>
<td>17.575</td>
<td>3.540</td>
<td>1.970</td>
</tr>
<tr>
<td>W36 X 529</td>
<td>156.1</td>
<td>39.79</td>
<td>17.220</td>
<td>2.910</td>
<td>1.610</td>
</tr>
<tr>
<td>W36 X 487</td>
<td>143.8</td>
<td>39.33</td>
<td>17.105</td>
<td>2.680</td>
<td>1.500</td>
</tr>
<tr>
<td>W36 X 441</td>
<td>130.2</td>
<td>38.85</td>
<td>16.965</td>
<td>2.440</td>
<td>1.360</td>
</tr>
<tr>
<td>W36 X 395</td>
<td>117.4</td>
<td>38.41</td>
<td>16.830</td>
<td>2.200</td>
<td>1.220</td>
</tr>
<tr>
<td>W36 X 361</td>
<td>106.5</td>
<td>37.99</td>
<td>16.730</td>
<td>2.010</td>
<td>1.120</td>
</tr>
<tr>
<td>W36 X 330</td>
<td>97.4</td>
<td>37.67</td>
<td>16.630</td>
<td>1.850</td>
<td>1.020</td>
</tr>
<tr>
<td>W36 X 302</td>
<td>89.3</td>
<td>37.33</td>
<td>16.655</td>
<td>1.680</td>
<td>0.945</td>
</tr>
<tr>
<td>W36 X 282</td>
<td>83.4</td>
<td>37.11</td>
<td>16.595</td>
<td>1.570</td>
<td>0.885</td>
</tr>
<tr>
<td>W36 X 262</td>
<td>77.4</td>
<td>36.85</td>
<td>16.550</td>
<td>1.440</td>
<td>0.840</td>
</tr>
<tr>
<td>W36 X 247</td>
<td>72.9</td>
<td>36.67</td>
<td>16.510</td>
<td>1.350</td>
<td>0.800</td>
</tr>
<tr>
<td>W36 X 231</td>
<td>68.5</td>
<td>36.49</td>
<td>16.470</td>
<td>1.260</td>
<td>0.760</td>
</tr>
</tbody>
</table>

### 302.4.2.1 GALVANIZED BEAM STRUCTURES

If a galvanized bridge structure is the selected structure type, the following problems should be recognized and dealt with by the designer.

Galvanizing tanks are shallow and normally not longer than 45 feet [13.7 meters] in length.
Therefore, beam lengths should not be longer than 60 feet [18.5 meters]. Before a design is completed, the designer should confirm with local galvanizers if a local plant can galvanize the structural members detailed.

Since standard holes may become partially filled with galvanizing, bolted splice designs will require a non-standard hole size equal to the nominal bolt diameter plus 1/8". Bolted crossframes will be required due to field installation issues. Bolted cross frames as detailed in the Standard Bridge Drawing may be specified.

Field welding of end crossframes, intermediate cross frames and bearings is not acceptable because welding onto galvanizing causes damage to the coating and no quality touch-up system is available to handle the number of repairs required.

302.4.2.2 STIFFENERS

Intermediate stiffeners shall only be used when required for cross frames. Stiffeners shall be a minimum 3/8 inch [10 mm] thickness and wide enough to make an adequate and easily accessible cross frame connection. Stiffeners generally should not extend beyond the edge of flange.
Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be one inch [25 mm] horizontally and 2½ inches [65 mm] vertically. Dimensions are shown on the Standard Bridge Drawing.

Both sides of the stiffener shall be fillet welded to the beam web and both flanges.

302.4.2.3 INTERMEDIATE CROSS FRAMES

For structures with the stringers placed on tangent alignments, detail cross frames as follows:

A. Cross frames for rolled beams shall be connected directly to the web or to intermediate web stiffeners.

B. Cross frames shall be perpendicular to stringers and be in line across the total width of the structure.

C. Cross frame spacings between points of dead load contraflexure in the positive moment regions shall not exceed 25 ft [7.6 m].

D. Cross frame spacings between points of dead load contraflexure in the negative moment regions shall not exceed 15 ft [4.6 m].

E. Horizontal legs of cross frame angles shall align on both sides of the stringer.

F. The AASHTO 10.20.1 requirement for cross frames at each support should be waived.

See the General Steel Details Standard Bridge Drawing for standard cross frame configurations.

For structures with flared stringers, the following exceptions apply:

A. If the differential angle between individual stringers is 5 degrees or less, the cross frames shall be perpendicular to one stringer and in line across the total width of the structure.

B. If the differential angle between individual stringers is greater than 5 degrees, the differential angle shall be divided evenly between connections to both stringers.

The design plans shall show:

A. The maximum cross frame spacing for each region along the length of the stringer. Actual spacing of the cross frames should be left to the steel fabricator’s detailer.

B. The typical cross frame details or reference to the General Steel Details Standard Bridge Drawing for standard cross frame configurations. If a design requires a specific location of cross frames, clearly show the cross frame locations that cannot be adjusted.
A detail showing a completely bolted connection for cross frame to the steel member is shown in the Standard Bridge Drawing.

Holes for erection bolts are normally provided in the connection of cross frames to stiffeners. Refer to the Standard Bridge Drawing for details.

In phased construction of new steel structures cross frames should not be permanently attached between phases until all deadload (deck, parapet, etc.) has been applied to the members. The crossframes can then be permanently attached and a deck closure pour can be completed to finish the superstructure. See Section 302.2.9.

For curved or flared bridges with “dog-legged” stringers, cross frames should be placed near the bend points. The cross frames should be located approximately 1 foot [300 mm] from the bend point but not interfere with the splice material. The cross frame should be placed normal to the stringer used to set the 1 foot [300 mm] clearance dimension and should be connected to the adjacent stringer only on the same side of the centerline of the splice. The cross frame units should be similar to standard cross frames but should have an additional horizontal angle near the top flange of the stringers.

See Figure 319 for plan view layout of cross frames for dog-legged stringers.

Cross frames for curved stringers may be one of the types shown on the Standard Bridge Drawing with an additional top strut. The designer shall confirm that the standard cross frames and their connections meet the additional loading developed in a curved member design. Since cross frame components in a curved structure share the live loading, Charpy V-notch (CVN) testing shall be specified. If specially designed cross frames are used, they should be bolted to stiffeners with oversized holes. The designer shall recognize the reduction in allowable capacity associated with oversized holes. If the capacity reduction is too much to allow for oversized holes and standard holes are required, the designer shall denote on the plans that shop assembly of the specially designed cross frames and adjacent curved member is required.

Both doglegged stringer cross frames at the dogleg or curved stringer cross frames shall be connected to the main member by use of welded stiffeners.
302.4.2.4 WELDS

CMS 513 permits welding by the following processes:

A. Shielded Metal Arc Welding (SMAW)

B. Flux Cored Arc Welding (FCAW)

C. Submerged Arc Welding (SAW)

Fabricators may choose to use one or more of these processes and each process has its advantages. Therefore, the designer should not specify the process.

The designer should specify fillet weld leg size required, in the case of fillet welds, or CP (complete joint penetration) in the case of full penetration groove welds. The designer should not select the joint configuration to be used for a full penetration weld. This should be left to the fabricator and the welding code.

302.4.2.4.a MINIMUM SIZE OF FILLET WELD

Fillet welds shall be designed for required stresses but should also meet the following size requirements:

A. Minimum size of fillet weld is based on the thickness of the thicker steel section in the weld joint. AWS D1.5 defines the minimum size of fillet weld.

B. 1/4 inch [6 mm] leg for up to 3/4 inch [19 mm] thick material.

C. 5/16 inch [8 mm] leg for greater than 3/4 inch [19 mm] material.

302.4.2.4.b NON-DESTRUCTIVE INSPECTION OF WELDS

Non-destructive testing (NDT) of welds is defined in CMS 513. The designer should be familiar with and understand these NDT requirements and their application.

For any special NDT inspection of unique or special welded joints, the designer should clarify the NDT requirements with the Structural Steel Section of the Office of Structural Engineering. A plan note defining any special requirements is required.

302.4.2.5 MOMENT PLATES

Fully welded moment plates shall not be used in areas of tensile stress due to the poor fatigue characteristics. End bolted cover plates, as defined in AASHTO, are acceptable for use in zones
of tensile stress if cost effective. Welded moment plates may be economical in the compression flange areas over the piers of continuous span structures and may be investigated by the designer. Welded moment plates shall not extend into a zone where the calculated total stresses are tensile. Designers should consult the Office of Structural Engineering for recommended moment plate details.

302.4.3 GIRDERS

302.4.3.1 GENERAL

Multiple designs should be investigated to determine the most economical. Often a design with an unstiffened web, eliminating transverse stiffeners, is the most economical. A design with a thicker web is also desirable from a maintenance standpoint because field and shop painting of stiffeners is a problem and is often a localized point of failure for the coating system. The NSBA and AISI are available to evaluate your options. Fabrication costs should be reviewed with the Department.

Longitudinal stiffeners shall not be used.

For haunched girders the corner between the flat bottom flange bearing seat area and the curved section of the bottom flange should be detailed as two plates with a full penetration weld. The fabricator shall be given the option of hot bending this flange per AASHTO Division II Section 11.4.3.3.3. A detail note is provided in Section 700.

In applying the above practices, consideration should also be given to the availability of plate lengths. Plates should not be extended beyond the lengths that can be furnished by the rolling mills.

302.4.3.2 FRACTURE CRITICAL

This section is not intended to recommend fracture critical designs. The designer should make all efforts to not develop a structure design that requires fracture critical members. As specified in Section 301.2, structures with fracture critical details require a concurrent detail design review to be performed by the Office of Structural Engineering.

Fracture critical members are defined in Section 2, Definitions, of the AASHTO/AWS D1.5, chapter 12 Fracture Control Plan.

If a bridge design includes any members or their components that are fracture critical, those members and components should be clearly identified as FRACTURE CRITICAL MEMBERS (FCM) in the plans. Fracture critical welds shall also be designated FCM in the plans. Include the detail note provided in Section 700 that references the appropriate sections of the AASHTO/AWS Bridge Welding Code.
If a girder is non-redundant, include the entire girder in the pay quantity for Item 513 - Structural Steel Members, Level 6. The designer shall designate the tension and compression zones in the fracture critical members.

302.4.3.3 WIDTH & THICKNESS REQUIREMENTS

302.4.3.3.a FLANGES

In addition to design limitations of width to thickness, flanges shall be wide enough that the girder will have the necessary lateral strength for handling and erection. An empirical rule is that the minimum width of top flange should be:

\[
W = \frac{d_w}{6} + 2.5 \times 12''
\]
\[
W = \frac{d_w}{6} + 65 \times 300\text{ mm}
\]

Where:

\(d_w = \text{web depth, inch [mm]}\)

\(W = \text{flange width, inch [mm] rounded up to the nearest inch [25 mm]}\)

Whenever possible, use constant flange widths throughout the length of the girder. The minimum thickness for any girder flange shall be 7/8 inch [22 mm]. Generally, selection of flange thicknesses should conform to the following:

A. For material 7/8" [22 mm] to 3" [76 mm] thick, specify thickness in 1/8" [2 mm] increments.

B. For material greater than 3" [76 mm] thick, specify thickness in 1/4" [5 mm] increments.

In the design of welded steel girders, the thickness of the flange plates is varied along the length of the girder in accordance with the bending moment. Each change in plate thickness requires a complete penetration butt-weld in the flange plate. These butt-welds are an expensive shop operation requiring considerable labor. In determining the points where changes in plate thickness occur, the designer should weigh the cost of butt-welded splices against extra plate thickness. In many cases it may be advantageous to continue the thicker plate beyond the theoretical stepdown point to avoid the cost of the butt-welded splice.

In order to help make this decision, guidelines proposed by United States Steel in their pamphlet “Fabrication - Its Relation to Design, Shop Practices, Delivery and Costs” may be used. The amount of steel that must be saved to justify providing a welded splice should be as follows:

A. For A709[M] grade 36 steel:

\[300 \text{ lb} + (25 \text{ lb} \times \text{cross sectional area, in }^2, \text{of the lighter flange plate})\]
B. For A709[M] grade 50 & 50W steel, the cutoff point shall be 85 percent of the value for grade 36 material.

### 302.4.3.3.b WEBS

The minimum web thickness shall be 3/8 inch [10 mm].

See Section 302.4.3.1 for recommendations on use of unstiffened web designs.

### 302.4.3.4 INTERMEDIATE STIFFENERS

Intermediate web stiffeners shall be a minimum 3/8 inch [10 mm] thickness. Stiffeners that extend beyond the edge of flange shall be clipped at a 45° angle. All intermediate stiffeners should be the same size.

Where intermediate stiffeners are to be used for the purpose of stiffening the web, it is preferable to use single stiffeners on alternate sides of the web of interior girders and only the inside of the web for fascia girders. These stiffeners shall be welded to the web and the compression flange. The tension flange shall be a tight fit.

Stiffeners shall be provided for the attachment of cross frames and shall be welded to the web and both flanges to help eliminate cracking of the web due to out of plane bending. The designer shall investigate that the fatigue criteria is met in these areas.

Stitch welding or single sided welding is not acceptable.

Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be 1 inch [25 mm] horizontally and 2½ inches [65 mm] vertically.

For details of stiffeners refer to the General Steel Details Standard Bridge Drawing.

### 302.4.3.5 INTERMEDIATE CROSS FRAMES

Cross frames for girders shall be connected to intermediate web stiffeners as shown in the Standard Bridge Drawing for General Steel Details.

For plate girder bridges, erection bolts shall be provided for the connections of cross frames to girder stiffeners. Erection bolts are normally 5/8 inch [16 mm] diameter. Bolt holes should generally be oversized. See the General Steel Details standard bridge drawing for typical details.
For additional intermediate cross frame information, refer to Section 302.4.2.3

302.4.3.6 WELDS

CMS 513 permits welding by the following processes:

A. Shielded Metal Arc Welding (SMAW)

B. Flux Cored Arc Welding (FCAW)

C. Submerged Arc Welding (SAW)

Fabricators may choose to use one or more of these processes and each process has advantages. Therefore, the designer should not specify the process.

The designer should specify fillet weld leg size required, in the case of fillet welds, or CP (complete joint penetration) in the case of full penetration groove welds. The designer should not select the joint configuration to be used for a full penetration weld. This should be left to the fabricator and the welding code.

For full penetration welds splicing flange materials or web materials a plan note should be added requiring removal of the weld reinforcement by grinding in the direction of the main stresses. The removal of reinforcement improves fatigue characteristics and makes NDT interpretation easier.

302.4.3.6.a TYPES

There are generally two (2) types of welds acceptable for bridge fabrication, fillet and complete penetration welds.

302.4.3.6.b MINIMUM SIZE OF FILLET AND COMPLETE PENETRATION WELDS, PLAN REQUIREMENTS

Fillets welds shall be designed for required stresses but should also meet the following size requirements:

A. Minimum size of fillet weld is based on the thickness of the thicker steel section in the weld joint. AWS D1.5 defines the minimum size of fillet weld.

B. 1/4 inch [6 mm] leg for up to 3/4 inch [19 mm] thick material.

C. 5/16 inch [8 mm] leg for greater than 3/4 inch [19 mm] material.
Complete or full Penetration welds are by definition welded through the full section of the plates to be joined. No partial penetration welds are acceptable for use except in secondary members not subject to tension or reversal stresses.

The designer should specify either fillet weld leg size, in the case of fillet welds, or CP (complete penetration) for complete joint penetration groove welds. The designer should not detail actual complete penetration welded joints symbols but only show the requirement that the welded joint be Complete Joint Penetration, CP.

Inspection and acceptance of a complete penetration weld is based on whether the weld will be loaded in tension or compression. In order to utilize this permissible quality difference between welds subjected to only compression or tension stresses, detail plans for steel girders should designate all flange butt welds that are subjected to compressive stresses only. This designation should be made by placing the letters “CS” next to full penetration welds shown on detail drawings. The following explanatory legend should be placed on the same detail sheet:

CS - indicates butt weld subject to compressive stresses only.

302.4.3.6.c INSPECTION OF WELDS, WHAT TO SHOW ON PLANS

Non-destructive testing (NDT) of welds is defined in CMS 513. The designer should be familiar with and understand these NDT requirements and their application.

For any special NDT inspection of unique or special welded joints, the designer should clarify the NDT requirements with the Structural Steel Section of the Office of Structural Engineering. A plan note for any special requirements shall be necessary in the design plans.

On railroad bridges, when full penetration web to flange welds are specified, the designer should add a note requiring 10 percent ultrasonic inspection. (The designer should check the AREMA specifications and with the actual railroad to confirm the individual railroad's requirements for NDT of welds.)

302.4.3.7 CURVED GIRDER DESIGN REQUIREMENTS

When designing curved girder structures, investigate all temporary and permanent loading conditions, including loading from wet concrete in the deck pour, for all stages of construction. Consider future re-decking as a separate loading condition. Design diaphragms as full load carrying members according to Section 302.4.2.3. The Designer shall perform a three-dimensional analysis representing the structure as a whole and as it will exist during all intermediate stages and under all construction loadings. Such analysis is essential to accurately predict stresses and deflections in all girders and diaphragms and to ensure that the structure is stable during all construction stages and loading conditions.
The Designer shall supply basic erection data on the contract plans. As a minimum, include the following information:

A. If temporary supports are required, provide the location of the assumed temporary support points, reactions and deflections for each construction stage and loading condition.

B. Instructions to the Contractor as to when and how to fasten connections for cross frames or diaphragms to assure stability during all temporary conditions.

Further design information for curved structures is contained in the “Guide Specifications for Horizontally Curved Highway Bridges”, published by the American Association of State Highway and Transportation Officials.

302.5 PRESTRESSED CONCRETE BEAMS

Model multi-span, non-composite members as simple-span for all loading conditions. The live load and future wearing surface shall be as defined in Section 301.4.

Model multi-span, composite members using the two loading conditions that follow. The loading condition that produces the largest load effects shall govern.

1. Simple-span for non-composite dead loads; continuous span for live load and composite dead loads. The live load and future wearing surface shall be as defined in Section 301.4.

2. Simple-span for all loading conditions. Do not include future wearing surface. The live load shall be as defined in Section 301.4.

302.5.1 BOX BEAMS

Physical dimensions and section properties of box beam cross sections shall be as shown on the Prestressed Concrete Box Beam Bridge Details, Standard Bridge Drawing.

Box beams should be limited to a maximum skew of 30 degrees.

Multiple span box beam bridges shall be joined over the piers with a T-joint as shown in the Standard Bridge Drawing. Structurally, non-composite beams shall be designed as simple spans. Composite beams shall be designed as simple span for non-composite dead loads and continuous for live loads and composite dead loads.

Expansion at the piers shall be accommodated by elastomeric expansion bearings or by flexibility of the piers for integral designs.

The length of abutment seats of prestressed concrete box beam bridges should be long enough to accommodate the total width out-to-out of all beams including a fit-up allowance of ½ inch [12 mm] per joint between beams.
In order to keep the beam seat from extending beyond the fascia of any pier of a box beam bridge, the length of the pier seat should only include a fit-up allowance for the joints between the beams of 1/4 inch [6 mm] per joint.

For box beam bridges that have skew combined with grade or which have variable superelevation, beam seats shall be designed and dimensioned to provide support for the full width of the box beams.

If a bridge structure's geometry causes a bridge deck in an individual span to have a different cross slope at one bearing than at the other bearing, the difference should be evenly divided so that the box beam seat cross slopes at both bearings are made to be the same. This adjustment gives the box beam full support at the seat without creating any twist or torsion on the box beam. Any elevation differences created by this beam seat adjustment should be adjusted for in the overlay, whether asphaltic or concrete.

Prestressed box beam members shall be supported by two bearings at each support.

Abutment wingwalls above the bridge seat and backwalls should not be cast until after box beams have been erected. The cast in place wingwall and box beam should normally be separated by one inch [25 mm] joint filler, CMS 705.03. The designer should show both requirements in the plans. Casting the backwall and wingwalls after the box beams are erected eliminates installation problems associated with the actual physical dimensions of the box beam and the joint filler. Cracking and spalling of backwall and wingwall concrete due to movements of the elastomeric bearings is also alleviated.

For box beam bridges with steel railing, the post spacing and position of post anchorage shall be detailed on the plans. The dimensioning for the post spacing shall be referenced to each prestressed beam end. The designer shall check that the post anchor spacing does not interfere with tierod locations or the "T" joint over the pier. The designer should confirm that post anchors at the ends of skewed box beams have both adequate concrete cover and do not interfere with the tierods. If the designer finds that no post spacing option can comply with the above requirements, the option of relocating the tie rods may be chosen. See standard drawings for maximum allowable spacing of tie rods.

When the box beam ends are not completely encased in concrete, the Standard Bridge Drawing requires Type B waterproofing on the ends. When required, Designers shall include a pay item for Item 512, Type B Waterproofing, in the estimated quantities.

**302.5.1.1 DESIGN REQUIREMENTS**

For box beam members, the live load distribution factors of AASHTO Section 3.23.4.3 shall be used.
Prestressed box beam design data sheets for non-composite designs are available on the Office of Structural Engineering website. The sheets are designed for TST-1-99 railing, a 60 lb/ft² future wearing surface and an HS25 or alternate military loading.

In order to prevent fabrication mistakes for beam length, the effect that the longitudinal grade has on dimensions measured along a beam’s length should be addressed in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8" [10 mm], all affected dimensions measured along the length of the beam should be clearly labeled so that the fabricator can make the necessary allowances in the shop drawings. A Typical Detail note is available in Section 700.
302.5.1.2 STRANDS

Debonding of strands, by an approved plastic sheath, shall be done to control stresses at the ends of the beams. Refer to Section 302.5.2.2.d for debonding limits.

Deflecting of strands in box beams to limit stresses shall not be allowed.

The designer shall show on the plans the number, spacing and length of debonding. The box beam fabricator may have the option to change the position of debonding as long as the change is still symmetrical.

All strands extended from a beam to develop positive moment resistance shall not be debonded strands.

302.5.1.2.a TYPE, SIZE OF STRANDS

A. Low-relaxation ½ inch diameter ($A_S = 0.153 \text{ in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

<table>
<thead>
<tr>
<th>Strands</th>
<th>Diameter (mm)</th>
<th>Area ($\text{mm}^2$)</th>
<th>Standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low-relaxation</td>
<td>12.7</td>
<td>$99$</td>
<td>ASTM A416M, Grade 270</td>
</tr>
</tbody>
</table>

B. Low-relaxation ½ inch diameter ($A_S = 0.167 \text{ in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

<table>
<thead>
<tr>
<th>Strands</th>
<th>Diameter (mm)</th>
<th>Area ($\text{mm}^2$)</th>
<th>Standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low-relaxation</td>
<td>12.7</td>
<td>$108$</td>
<td>ASTM A416M, Grade 270</td>
</tr>
</tbody>
</table>

C. Low-relaxation 0.6 inch diameter ($A_S = 0.217 \text{ in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

<table>
<thead>
<tr>
<th>Strands</th>
<th>Diameter (mm)</th>
<th>Area ($\text{mm}^2$)</th>
<th>Standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low-relaxation</td>
<td>15.24</td>
<td>$140$</td>
<td>ASTM A416M, Grade 270</td>
</tr>
</tbody>
</table>

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute prior to specifying 0.6 inch [15.244 mm] diameter or larger strand sizes.

302.5.1.2.b SPACING

Strands shall be spaced at increments or multiples of 2 inches [50 mm].

The location of the centerline of the first row of strands shall be 2 inches [50 mm] from the bottom of the beam. If possible, all strands shall be completely enclosed by the #4 [#13M]
stirrup bars. For designs that cannot meet this requirement, the minimum distance from the side of the beam to the centerline of the first strand shall be 2 inches [50 mm]. Strands near the top flange shall be placed below all transverse and longitudinal reinforcing steel and to the left and right of the void.

### 302.5.1.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be according to AASHTO requirements and shall be detailed on the plans.

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stress</td>
<td>$0.75 \frac{f_N}{N} = 202,500 \text{ psi [1400 MPa]}$</td>
</tr>
<tr>
<td>Initial tension load</td>
<td>$30,982 \text{ lb/strand (}A_S = 0.153 \text{ in}^2)$</td>
</tr>
<tr>
<td></td>
<td>$33,818 \text{ lb/strand (}A_S = 0.167 \text{ in}^2)$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial stress</td>
<td>$0.75 \frac{f_N}{N} = 1400 \text{ MPa}$</td>
</tr>
<tr>
<td>Initial tension load</td>
<td>$138,600 \text{ N/strand (}A_S = 99 \text{ mm}^2)$</td>
</tr>
<tr>
<td></td>
<td>$151,200 \text{ N/strand (}A_S = 108 \text{ mm}^2)$</td>
</tr>
</tbody>
</table>

The estimated stress in the prestressing tendon immediately after transfer shall be $0.69 \frac{f_N}{N} = 186,300 \text{ psi [1285 MPa]}$ for loss calculation purposes.

Total losses shall be calculated with 70% relative humidity (RH) and a modulus of elasticity of prestressing reinforcement ($E_S$) equal to 28,500 ksi [196 500 Mpa]. Total losses may be expressed by:

$$F_S = SH + ES + CR_C + CR_S$$

$$F_S = 11.175 + \left[ \frac{25,650}{E_{ci}} + 11.4 \right] f_{c_{ir}} - 6.65 f_{cds}$$

$$F_S = 77.0 + \left[ \frac{176,850}{E_{ci}} + 11.4 \right] f_{c_{ir}} - 6.65 f_{cds}$$

### 302.5.1.3 COMPOSITE

Composite reinforced deck slabs on prestressed box beams shall be a minimum of 6 inches [155 mm] thick and shall be reinforced with #6 [#19M] bars. The longitudinal bars shall be spaced at 18" [450 mm] and the transverse bars spaced at 9" [225 mm]. For ease of placement on skewed structures, the transverse bars may be placed parallel to the substructure units with spacing measured parallel to the longitudinal axis of the structure.

On multiple span composite box beam bridges additional longitudinal reinforcing steel over the piers is required. The additional bars shall be alternately spaced with the standard longitudinal reinforcement and the pier bar's length shall be equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of AASHTO 8.24.3.3. The pier bars should be placed longitudinally and approximately centered on the pier but with a 3 foot [1000 mm] stagger.
In the negative moment regions of structures made continuous over the piers, the tensile stresses in the precast section shall not exceed the AASHTO allowable stresses for members with bonded reinforcement. Unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled as a continuous beam on a single support centered on the pier.

Composite box beam structures with concrete parapets or sidewalks should not incorporate fit-up tolerances in the finished roadway width. To compensate for fit-up tolerances the composite deck and barrier and/or sidewalk should be designed to cantilever or overhang the boxbeam units by 2" [50 mm] to 8" [200 mm] each side with the fit-up being absorbed in the overhang. A mixture of 48" [1220 mm] and 36" [915 mm] boxbeam units may be necessary to meet this requirement.

See Figure 320 for a sketch of the cross-section of the composite deck superstructure.

302.5.1.4 NON-COMPOSITE WEARING SURFACE

Non-composite box beam bridges with asphalt overlays shall have either Type D Waterproofing or Type 3 Waterproofing as specified in CMS 512 placed on the boxes before the 1½ inch [38 mm] minimum layers of CMS type 448 asphaltic concrete is applied. See section 302.1.3.1. The Type 3 Waterproofing is preferred.

Non-composite box beam bridges with asphalt overlays shall be limited to a 4 percent combined grade. Combined grades greater than 4 percent require a composite deck design. Combined grade includes both the longitudinal and transverse structure grades calculated as follows:

\[ \text{Combined Grade (Cg)} = \left( \text{[deck slope]}^2 + \text{[transverse grade]}^2 \right)^{1/2} \]

302.5.1.5 CAMBER

In establishing bridge seat elevations and assuring a minimum design slab or overlay thickness, allowance shall be made for camber due to prestressing according to the following:

\[ \begin{align*}
A &= \text{Minimum topping thickness} \\
B &= \text{Anticipated total mid-span camber due to the design prestressing force at time of release} \\
C &= \text{Mid-span deflection due to the self weight of the beam (including diaphragms)} \\
D &= \text{Mid-span deflection due to dead load of the topping and other non-composite loads} \\
E &= \text{Mid-span deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface}
\end{align*} \]
\( F = \) Adjustment for vertical curve (Positive for crest vertical curves)

\( G = \) Total topping thickness at beam bearings = \( A + 1.8B - 1.85C - D - E - F \). If \( F > 1.8B - 1.85C - (D + E) \) then \( G = A \).

\( H = \) Total topping thickness at mid-span = \( A \). If \( F > 1.8B - 1.85C - (D + E) \) then \( H = A - (1.8B - 1.85C) + D + E + F \).

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section when designing a composite box beam otherwise use the non-composite section. Note that with the exception of when \( F > 1.8B - 1.85C - (D + E) \), the dead load deflection adjustment \( (D + E) \) is made by adjusting the beam seat elevations upward.

The designer shall provide the camber at the time of release (B–C), the camber at the time of erection (1.8B – 1.85C), long term camber (2.45B – 2.40C), and a longitudinal superstructure cross section in the plans. For non-composite beams, show the thickness of the Item 448 Intermediate course and the Item 448 surface course at each centerline of bearing and at mid-span points. For composite beams, show the total topping thickness at each centerline of bearing and at mid-span points and provide a screed elevation table.

#### 302.5.1.6 ANCHORAGE

In a box beam design, all beams shall be anchored at abutments and piers. The anchor shall be in the center of the cross section of the box beam and shall conform to details presented in the Standard Bridge Drawing.

Fixed end anchor dowels shall be installed with a non-shrinking grout (mortar). Expansion end anchor dowel holes shall be filled with joint sealer, CMS 705.04.

Preformed expansion joint filler, 705.03, the same thickness as the elastomeric bearing, shall be installed under the box beam, around the anchor dowel, to halt the grout or sealer from leaking through to the beam seat.

#### 302.5.1.7 CONCRETE MATERIALS FOR BOX BEAMS

The designer has a choice of 28-day compressive strengths ranging form 5500 psi [38 Mpa] to 7000 psi [48 Mpa]. The 28-day compressive strength chosen for design shall be listed in the contract plan General Notes.

The designer has a choice of compressive strength at the time of release ranging from 4000 psi [27.5 Mpa] to 5000 psi [34.5 Mpa]. The release strength chosen for design shall be listed in the
contract plan General Notes.

Cast-in-place concrete for composite decks, pier “T” sections, etc., shall be Class S or HP superstructure concrete - 4500 psi [31.0 MPa] at 28 days.

For concrete in composite decks see Section 302.1.2.

302.5.1.8 REINFORCING

Epoxy coated reinforcing steel shall be used in composite deck slabs and shall be Grade 60 [420], \( F_y = 60 \text{ ksi} \) [420 MPa].

Reinforcing steel used in the standard design box beams is Grade 60 [420], \( F_y = 60 \text{ ksi} \) [420 MPa].

The fabricator, by specification, is required to use a corrosion-inhibiting admixture in the concrete. Reinforcing bars projecting from the prestressed members shall be epoxy coated.

302.5.1.9 TIE RODS

Tie rods shall be provided and installed according to the Prestressed Concrete Box Beam Bridge standard bridge drawing.

Diaphragms and transverse tie rods for prestressed concrete box beam spans shall be provided at mid-span for spans up to 50 feet [15 000 mm], at third points for spans from 50 feet [15 000 mm] to 75 feet [23 000 mm] and at quarter points for spans greater than 75 feet [23 000 mm].

302.5.2 I-BEAMS

AASHTO standard prestressed I-beam shapes, type II through type IV and modified type IV, as shown in standard bridge drawing PSID-1-99, shall be used.

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute to review details and assess the constructability of I-beam cross-sections other than those shown on Standard Bridge Drawing PSID-1-99.

In designing prestressed I-beams, the non-composite section shall be used for computing stresses due to the beam and deck slab. The composite section shall be used for computing stresses due to the superimposed dead, railing and live loads. For multiple span continuous structures, the non-composite loadings shall be applied to the I-beam modeled as a simple span, and the composite loadings applied to the beam modeled as a continuous structure. Unless a more precise method of analysis is performed, the composite structure shall be conservatively modeled.
as a continuous beam on a single support centered on the pier.

302.5.2.1 DESIGN REQUIREMENTS

Prestressed I-beam designs should reflect the change in the loading requirements of Section 301.4. Prestressed I-beam load distribution factors shall conform to AASHTO.

In order to prevent fabrication mistakes for beam length, the effect that the longitudinal grade has on dimensions measured along a beam’s length should be addressed in the plans. When the beam length measured along the grade differs from the beam length measured horizontally by more than 3/8" [10 mm], all affected dimensions measured along the length of the beam should be clearly labeled so that the fabricator can make the necessary allowances in the shop drawings. A Typical Detail note is available in Section 700.

When detailing beam elevations, dimension the locations of all inserts, hold-downs, etc. to the ends of the beam rather than the centerlines of bearing.

302.5.2.2 STRANDS

Prestressed I-beam designs shall follow the criteria established in standard bridge drawing PSID-1-99. The preferred strand pattern is straight, parallel strands with no debonding. However, excessive tensile stresses may develop in the beam ends during the release of the prestressing force. To relieve these excessive stresses, the following strand patterns are allowed: (listed in order of preference)

A. Straight strands fully bonded with additional strands placed above the neutral axis, debonded at midspan if necessary. The additional strands shall be placed inside the web and top flange reinforcing steel and shall be symmetrical about the centerline of the beam.

B. Draped strands.

C. Combination of draped strands and bottom flange strands debonded at the beam ends.

Transforming strand area in order to increase section properties is not allowed.

302.5.2.2.a TYPE, SIZE

A. Low-relaxation ½ inch diameter \( (A_s = 0.153 \text{ in}^2) \) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 12.7 mm diameter \( (A_s = 99 \text{ mm}^2) \) seven wire uncoated strands, ASTM A416M, Grade 270.
B. Low-relaxation $\frac{1}{2}$ inch diameter ($A_S = 0.167 \text{ in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 12.7 mm diameter ($A_S = 108 \text{ mm}^2$) seven wire uncoated strands, ASTM A416M, Grade 270.

C. Low-relaxation 0.6 inch diameter ($A_S = 0.217 \text{ in}^2$) seven wire uncoated strands, ASTM A416, Grade 270.

Low-relaxation 15.24 mm diameter ($A_S = 140 \text{ mm}^2$) seven wire uncoated strands, ASTM A416M, Grade 270.

Consult the Office of Structural Engineering and the Ohio/Indiana/Kentucky Prestressed Concrete Institute prior to specifying 0.6 inch [15.244 mm] diameter or larger strand sizes.

The prestressing strand type and size selected for the I-beam design shall be listed in the contract plan General Notes.

302.5.2.2.b **SPACING**

Strands shall be spaced at increments of 2 inches [50 mm].

A minimum 2 inch [50 mm] dimension from bottom of beam to centerline of the first row of strands and any exterior beam surface shall also be maintained.

302.5.2.2.c **STRESSES**

Initial prestressing loads for low-relaxation strand shall be as per AASHTO requirements and shall be detailed on the plans.

Initial stress .............................................................. $0.75 f_N = 202,500 \text{ psi}$
Initial tension load .......................................................... $30,982 \text{ lb/strand} (A_S = 0.153 \text{ in}^2)$

$33,818 \text{ lb/strand} (A_S = 0.167 \text{ in}^2)$

Initial stress .............................................................. $0.75 f_N = 1400 \text{ MPa}$
Initial tension load .......................................................... $138 \text{ 600 N/strand} (A_S = 99 \text{ mm}^2)$

$151 \text{ 200 N/strand} (A_S = 108 \text{ mm}^2)$

The estimated stress in the prestressing tendon immediately after transfer shall be $0.69 f_N = 186,300 \text{ psi} [1285 \text{ Mpa}]$ for loss calculation purposes.

Total losses shall be calculated with 70% relative humidity (RH) and a modulus of elasticity of prestressing reinforcement ($E_S$) equal to 28,500 ksi [196 500 Mpa]. Total losses may be
expressed by:

\[
F_S = SH + ES + CR_c + CR_S
\]

\[
F_S = 11,175 + \left\{ \frac{25,650}{E_{ci}} + 11.4 \right\} f_{c_{ir}} - 6.65 f_{c_{ds}}
\]

\[
F_S = 77.0 + \left\{ \frac{176,850}{E_{ci}} + 11.4 \right\} f_{c_{ir}} - 6.65 f_{c_{ds}}
\]

302.5.2.2.d DEBONDING

Debonding or shielding of the strands, with an approved plastic sheath, may be done at the beam ends to relieve excessive stresses. The following guidelines shall be followed for debonded strand designs:

A. The maximum debonded length at each end shall not be greater than \(0.16L - 40'' \) \([0.16L - 1000]\). Where \(L\) equals the span length in inches [millimeters].

B. A minimum of one-half the number of debonded strands shall have a debonded length equal to one-half times the maximum debonded length.

C. No more than 25\% of the total number of strands in the I-beam shall be debonded.

D. No more than 40\% of the strands in any row shall be debonded.

E. Debonded strands shall be symmetrical about the centerline of the beam.

F. Strands extended from a beam to develop positive moment resistance at pier locations shall not be debonded strands.

G. Locate debonded strands as high as possible in the bottom flange to aid in the placement of the sheath during fabrication.

The designer shall show on the detail plans the number, spacing and the length of required debonding per strand.

302.5.2.2.e DRAPING

Draping or harping of the strands may be done to relieve excessive stresses at the beam ends. The hold down point shall be located at least 5'-0'' [1.5 m] on each side of the midspan of the beam using increments of 6'' [150 mm]. The Designer shall calculate the vertical uplift force, \(P_U\), at each hold-down location to ensure the following limits are not exceeded:
<table>
<thead>
<tr>
<th>No. of Draped Strands per Row</th>
<th>P&lt;sub&gt;U&lt;/sub&gt;/Strand (lb) [kN]</th>
<th>P&lt;sub&gt;U&lt;/sub&gt;/Unit (lb) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6000 [26.6]</td>
<td>48,000 [213.5]</td>
</tr>
<tr>
<td>2</td>
<td>4000 [17.7]</td>
<td>48,000 [213.5]</td>
</tr>
<tr>
<td>3</td>
<td>4000 [17.7]</td>
<td>48,000 [213.5]</td>
</tr>
</tbody>
</table>

Where:

\[
P_{U/Strand} = (1.05)^{0.75} f_N A_{PS} \tan \psi
\]

\[
P_{U/Unit} = \sum_{i=1}^{n} (P_{U/Strand})_i
\]

and,

\[f_N = 270,000 \text{ psi} [1860 \text{ MPa}]
\]

\[A_{PS} = \text{Area of single strand (in}^2) [\text{mm}^2]\]

\[n = \text{no. of strands}\]

\[\psi = \text{Angle of strand inclination measured from horizontal}\]

To minimize the uplift force, locate the hold down point as close as allowed to the midspan and limit the height of the draped strands at the beam ends to only the height required to control stresses. It is not necessary for the angle of inclination for each row of draped strands to be the same. The height of draped strands at beam ends and at midspan shall be multiples of 2" [50 mm]. Do not place straight strands above draped strands in the same vertical column.

**302.5.2.3 CAMBER**

A variable depth haunch shall be required to account for the effects of camber due to the design prestressing. This haunch depth shall include an additional 2 inches [50 mm] that may be sacrificed to account for differences between actual and design camber. This sacrificial depth shall not be included when determining the composite section properties; however, its weight shall be included in the dead load of the slab. Haunch concrete shall be included in the total volume of superstructure concrete for payment.

In establishing bridge seat elevations and assuring a minimum design slab thickness, allowance shall be made for camber due to prestressing as per the following:

\[A = \text{Design slab thickness}\]

\[B = \text{Anticipated total mid-span camber due to the design prestressing force at time of release}\]
C = Mid-span deflection due to the self-weight of the beam.

D = Mid-span deflection due to dead load of the slab, diaphragms and other non-composite loads.

E = Mid-span deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface.

F = Adjustment for vertical curve. Positive for crest vertical curves.

G = Sacrificial haunch depth (2" [50 mm]).

H = Total topping thickness at beam bearings = A + 1.8B - 1.85C - D - E - F + G. If F > 1.8B - 1.85C - (D + E) then H = A + G

I = Total topping thickness at mid-span = A + G. If F > 1.8B - 1.85C - (D + E) then I = A - (1.8B - 1.85C) + D + E + F + G.

The gross moment of inertia for the non-composite beam shall be used to calculate the camber and deflection values for B, C and D. The moment of inertia for the composite section should be used to calculate value E.

The designer shall show a longitudinal superstructure cross section in the plans detailing the total topping thickness, including the design slab thickness and the haunch thickness at the centerline of spans and bearings. Provide the camber at the time of release (B-C), camber at the time of erection (1.8B – 1.85C), long term camber (2.45B – 2.40C), and a screed elevation table according to Section 302.2.3.

302.5.2.4 ANCHORAGE

One inch [25 mm] diameter anchors shall be provided at each fixed pier as shown on standard bridge drawing PSID-1-99.

Minimum number of anchors shall be 2 for each beam line.

The anchors shall be a minimum of 2'-0" [600 mm] long. Anchors shall be embedded a minimum of 1'-0" [300 mm] into the pier cap and 1'-0" [300 mm] into the field cast-in-place concrete pour which connects any two discontinuous prestressed I-beams in the same beam line into a continuous member. The anchors should be drilled in place at the centerline of the pier between the ends of adjoining prestressed I-beams. The designer should confirm the pier cap has reinforcing steel clearance to accept these anchors.
302.5.2.5  DECK SUPERSTRUCTURE AND PRECAST DECK PANEL

It is recommended that only cast-in-place concrete decks, Class S or HP Concrete be designed and used.

The precast panel alternative, previously used, has shown cracking problems at the joints between the panels and there are questions on the transfer of stresses in the finished deck sections.

302.5.2.6  DIAPHRAGMS

Maximum spacing of intermediate diaphragms shall be 40'-0" [12 000 mm].

Intermediate diaphragms may be either cast-in-place concrete or galvanized steel. The contractor shall choose the type. Details for each type are provided in the standard bridge drawing PSID-1-99. The design plans shall show the centerline location of each intermediate diaphragm. Payment for the intermediate diaphragms shall be made at the contract price for item 515, each, Intermediate Diaphragms.

Cast-in-place intermediate diaphragms should not make contact with the underside of the deck because they could act as a support to the deck, causing cracking and possible over stressing of the deck. The top of the cast-in-place intermediate diaphragm should start at the bottom vertical edge of the top flange and end at the top of the vertical edge of the bottom flange.

If the Standard Bridge Drawing for I-beams is not referenced by the contract plans, the designer shall add a note to the plans for prestressed I-beam designs requiring cast-in-place intermediate diaphragms to be placed and cured at least 48 hours before deck placement.

Threaded inserts shall be used to connect the cast-in-place diaphragm reinforcing steel to the I-beam. Use of the inserts will ease installation of the diaphragms, allow transfer of load to the beam and help protect the diaphragms against cracking. The threaded inserts and the threaded rods shall be galvanized according to CMS 711.02.

End diaphragms shall be provided. Diaphragms shall be cast-in-place. The top of the end diaphragm shall make complete contact with the deck. The bottom of the end diaphragm shall end at the top vertical edge of the bottom flange. The bottom of the diaphragm shall not extend down to the bottom of the I-beam's bottom flange. Refer to standard bridge drawing PSID-1-99 for typical diaphragm details.

302.5.2.7  DECK POURING SEQUENCE

A deck pour sequence is required for all prestressed I-beam designs made continuous at pier locations. Standard bridge drawing PSID-1-99 establishes one sequence. The designer should
either accept the standard drawing sequence or detail an alternative.

302.5.2.8 CONCRETE MATERIALS FOR I-BEAMS

The designer has a choice of 28-day compressive strengths ranging from 5500 psi [38 Mpa] to 7000 psi [48 Mpa]. The 28-day compressive strength chosen for design shall be listed in the contract plan General Notes.

The designer has a choice of compressive strength at the time of release ranging from 4000 psi [27.5 Mpa] to 5000 psi [34.5 Mpa]. The release strength chosen for design shall be listed in the contract plan General Notes.

Cast-in-place concrete, (composite decks, pier diaphragms, intermediate diaphragms, etc.) Shall be Class S or HP superstructure concrete - 4500 psi [31.0 MPa] at 28 days.

Consult the Office of Structural Engineering for recommendations prior to designing a structure with concrete strengths higher than those shown above.

302.5.2.9 REINFORCING

Unless otherwise specified all reinforcing steel used shall be epoxy coated, Grade 60 [420] \( F_y = 60 \text{ ksi} [420 \text{ MPa}] \).

The fabricator, by specification, is required to use a corrosion-inhibiting admixture to the concrete.

Reinforcing bars projecting from the prestressed members shall be epoxy coated.

Reinforcing steel stirrups shall completely enclose the strands for the entire length of the beam.

For composite designs the total amount of longitudinal reinforcing steel over the piers, for the deck slab shall be determined in accordance to AASHTO.

302.5.2.10 TRANSPORTATION & HANDLING CONSIDERATIONS

In order to prevent damaging the beams during transit and erection, fabricators may require additional strands to be placed in the top flange. These shipping strands keep the top flange in compression until the beams are set into final position. Once set, the shipping strands are cut to release their prestressing force and allow the beams to reach their design ultimate capacity.
303  SUBSTRUCTURE

303.1  GENERAL

If a pier column, wall or other structural member is located in the sloped portion of an embankment, the design active earth pressure shall be applied to an effective width (S) of the member as defined in the following table. The effective width accounts for the earth pressure due to the embankment directly in back of the member and the earth pressure due to the adjacent embankment on each side.

<table>
<thead>
<tr>
<th>Type of Member</th>
<th>S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Column or Wall</td>
<td>a + H</td>
</tr>
<tr>
<td>Interior Columns</td>
<td>c $ H a + H</td>
</tr>
<tr>
<td></td>
<td>c &lt; H a + H - (H - c)^2/H</td>
</tr>
<tr>
<td>Exterior Columns</td>
<td>c $ H a + H</td>
</tr>
<tr>
<td></td>
<td>c &lt; H a + H - (H - c)^2/2H</td>
</tr>
</tbody>
</table>

Where:  
- c = One-half of the distance between adjacent members measured face to face.
- H = Height of the active earth fill measured at the face of the footing.
- a = Width of the member.

The minimum design earth pressure shall be 40 psf [2.0 kPa] unless granular backfill is provided.

303.1.1  SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Specifications for the sealer are defined in CMS 512. Concrete surfaces shall be sealed with a concrete sealer as follows:

A. The front face of abutment backwalls, from top to bridge seat, the bridge seat and the breastwall down to the groundline shall be sealed with an epoxy-urethane or non-epoxy sealer. (Note: Sealing of the backwall shall not be required on prestressed box beam bridges because the beams are installed before the backwall is placed.)

B. The exposed surfaces of all wingwalls and retaining walls, exclusive of abutment type, that are within 30 feet [10 000 mm] of any pavement edge shall be sealed with an epoxy-urethane sealer.

C. Ends and sides of piers exposed to traffic-induced deicer spray, from any direction, shall be sealed with either an epoxy-urethane or non-epoxy sealer. Top of pier caps need only be sealed if there is an expansion joint or the tops are subject to exposure to deicer-laden water.
D. The total vertical surface of piers which are adjacent to traffic lanes shall be sealed with either an epoxy-urethane or non-epoxy sealer. Structures with A588[M] weathering steel superstructures shall also have their piers sealed as stated above with either an epoxy-urethane or non-epoxy sealer.

The designer should include in the plans actual details showing the position, location and area required to be sealed. A plan note to describe the position should not be used as there can be both description and interpretation problems.

The designer has the option to select a specific type of sealer, epoxy-urethane or non-epoxy. The designer also has the alternative to just use a bid item for sealer, with no preference, and allows the contractor to choose based on cost.

See Figures 321, 322 & 323.

In areas where concrete surfaces have a history of graffiti vandalism, the designer may add a sacrificial or permanent graffiti coating meeting the requirements of Supplement 1083 on top of the epoxy-urethane or non-epoxy sealer. A plan note is available in BDM Section 600. The designer should limit the concrete surfaces that are treated with sacrificial or permanent graffiti coatings to those reachable by easy climbing and visible to the traveling public.

303.2 ABUTMENTS

303.2.1 GENERAL

Abutments should be provided with backwalls to protect the superstructure from contact with the approach fill and to assist in preventing water from reaching the bridge seat.

For members designed to retain earth embankments and restrained from deflecting freely at their tops, the computed backfill pressure shall be determined by using at-rest pressure. Examples include: rigid frame bridges, abutment walls keyed to the superstructure, and some types of U-abutments.

For abutment walls of structures designed without provision for expansion between superstructure and substructure and where an appreciable amount of superstructure expansion is anticipated, passive earth pressure should be considered in the design.

To allow for slight tilting of wall type abutments after the backfill has been placed, batter the front face 1/16" for each foot [5 mm for each 1000 mm] of abutment height. Height is measured from bottom of footing to the roadway surface.
303.2.1.1 PRESSURE RELIEF JOINTS FOR RIGID PAVEMENT

If rigid concrete pavement or base is to be used adjacent to the structure, the designer shall confirm that the roadway plans require installation of type A pressure relief joints, as per Standard Construction Drawing BP-2.3.

Pressure relief joints are required to alleviate backwall pressures on abutments with expansion devices and to allow freedom of movement for integral and semi-integral abutments.
303.2.1.2 BEARING SEAT WIDTH

For all continuous slab bridges, the approach slab seat and the bridge slab seat should be placed at the same elevation by providing a haunch on the thinner slab.

For continuous or simple span beam or girder structures the abutment seat width should be 2'-0" [600 mm]. Centerline of bearing should be 1'-0" [300 mm] from face of backwall. Exceptions to this practice may be necessary for highly skewed structures.

The configuration for prestressed I-beam abutment bearing seat widths is similar to those for the steel beam or girder. The seat width will vary due to size of elastomeric bearing, prestressed I-beam flange width and the structure's skew.

Abutment bearing seat widths for prestressed box beams without expansion devices is normally 1'-4" [400 mm] with centerline of bearing 9" [225 mm] from face of breast wall. The seat width for prestressed box beams with expansion devices shall be increased but the centerline of bearing shall remain 9" [225 mm] from face of breast wall.

AASHTO seismic seat width requirements, based on length and height of structure, may require additional seat width. All defined abutment bearing seat widths can be affected due to special considerations for a specific structure or type of bearing. See Section 301.4.3 of this Manual.

303.2.1.3 BEARING SEAT REINFORCEMENT

Bearing areas of abutments may require supplementary reinforcement to resist local compressive and shearing stresses.

The location and spacing of all reinforcing in bridge seats should be chosen to provide adequate clearance for bearing anchors whether pre-set or drilled in place. The designer should recognize that drilled in place anchors use larger holes than the actual anchor.

A note shall be provided on the substructure detail sheets cautioning the contractor to place the reinforcing to avoid interference with the anchor bolts. Also a “Bearing Anchor Plan” to adequately show the location of the bearing anchors with respect to the main reinforcing bars and the edges of the bridge seats shall be provided.

303.2.1.4 PHASED CONSTRUCTION JOINTS

Seal the vertical joint between construction phases on the back side of abutment backwalls and breastwalls from the top of the footing to the approach slab seat with Item 512, Type 2 Waterproofing, 3'-0" [915 mm] wide centered on the joint.
303.2.2 TYPES OF ABUTMENTS

303.2.2.1 FULL HEIGHT ABUTMENTS

If the computed horizontal forces at the bottom of the footing for full height abutments cannot be completely resisted by the friction of the subsoil, by the action of vertical and battered bearing piles, or drilled shafts, or by footing keys, steel sheet piling rigidly attached to the footing may be used to provide additional resistance. See Section 303.4.1.1.

The minimum projection of the steel sheet piling below the bottom of the footing shall be 5 feet [1.5 meters]. If the sheet piling is placed in front of battered bearing piles, it also should be specified to be battered.

Where these short lengths of steel sheet piling are used, the sheet piles should be anchored to the face of the toe of the footing by not less than two #6 [#19M] reinforcing bars attached near the top of each sheet pile and included with the sheet piling for payment. The #6 [#19M] bars shall be long enough to be fully developed in bond.

If a 5 foot [1500 mm] projection of sheet piling below the bottom of the footing is found to be sufficient, the piles should have a minimum section modulus of 7 in³ per foot [375 000 mm³ per meter] of wall. For other lengths of sheet piling the minimum required section modulus should be computed. The plans shall show the minimum required section modulus. See plan notes Section 700.

Vertical rustication grooves may be provided at 48 inch [1200 mm] centers to fit the width of standard plywood forms or liners.

An alternate to vertical rustication grooves is the use of formliners to provide the wall surface with an aesthetic appearance. While a variety of formliners are available the following criteria should be met:

A. Formliners should not be used when they will not be visible to the public. The selected pattern of formliner should be easily visible from a distance. Small or ornate patterns not easily visible from a distance do not enhance the structure and are not cost effective.

B. Minimum cover requirements for reinforcing steel must be met. If a formliner is used minimum concrete cover shall not be violated by patterns or indents of the formliner. This will require additional concrete and in some cases dimensional changes.

C. The cost of formliners selected should add only minimal additional cost to the overall cost of the concrete (1 to 3 percent per yard [meter] of the abutment, pier or wall)

D. Generic formliner patterns shall be specified. An alternative of at least three suppliers listed. Listing of a formliner pattern only available from one supplier will not be accepted.
303.2.2.1.a  COUNTERFORTS FOR FULL HEIGHT ABUTMENTS

For full height abutments exceeding 30 feet [10 000 mm] in height, counterforts should be considered.

Reinforcing steel in the back, sloping, face of the counterfort should be placed in two rows with a 6 inch [150 mm] clearance between rows. Reinforcing steel splices should be staggered a minimum of 3'-0" [1000 mm], by row.

Reinforcing extending from the footing of a counterforted wall into the highly reinforced areas of the counterforts shall have reinforcing steel splices staggered.

In counterforted walls, each pocket formed by the intersection of the counterfort and wall shall be drained.

303.2.2.1.b  SEALING STRIP FOR FULL HEIGHT ABUTMENTS

Use an impervious fabric across the expansion joints in full height abutments or retaining walls to eliminate leakage. The impervious fabric should be CMS 512 Type 2 Waterproofing, 3 feet [1000 mm] wide, centered over, and extending the full length of the joint to the top of the footing. See Section 303.2.5 on requirements for expansion joints in abutments.

303.2.2.2  CONCRETE SLAB BRIDGES ON RIGID ABUTMENTS

For a continuous concrete slab bridge supported on rigid abutments, the joint between the deck slab and the top of the abutment shall be troweled smooth and a continuous strip of elastomeric material shall be recessed into the abutment seat before placement of the superstructure concrete.

The above bearing system for slabs on rigid abutments should conform to temperature movement and bearing design requirements of this Manual.

303.2.2.3  STUB ABUTMENTS WITH SPILL THRU SLOPES

If a stub abutment is to support a bridge having provision for relative movement between the superstructure and the abutment, two rows of piles are required and the front row shall be battered 1:4.

Where two rows of piles are used, the forward row shall have approximately twice the number of piles as the rear row, with the rear piles placed directly behind alternate front piles.

The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4'-0" [1200 mm].
The maximum spacing of piles in a single row or in the front row of a double row shall be 8 feet [2500 mm].

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

### 303.2.2.4 CAPPED PILE STUB ABUTMENTS

For capped pile stub abutments that do not provide for relative movement between the superstructure and the abutment, one row of vertical piles shall be used.

The construction joint at the top of the footing for cap pile abutments should be shown as optional.

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

### 303.2.2.5 SPREAD FOOTING TYPE ABUTMENTS

Where foundation conditions warrant the use of an abutment on a spread footing, the bottom of the footing should be at least 5'-0" [1525 mm] below the surface of the embankment.

The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 5'-0" [1525 mm].

In no case shall the top of the footing be less than 1'-0" [300 mm] below the surface of the embankment.

### 303.2.2.6 INTEGRAL ABUTMENTS

Integral Abutment use is limited as defined in Section 200 of this Manual. Integral design should not be used with curved main members or main members that have bend points in any stringer line.

For an integral design to work properly, the geometry of the approach slab, the design of the wingwalls, (see section 303.2.4) and the transition parapets must be compatible with the freedom required for the integral (beams, deck, backwall, wingwalls and approach slab) connection to rotate and translate longitudinally.

See Figure 324.

The horizontal and vertical joint shall be sealed at the back face of the backwall by use of a 3'-0" [900 mm] wide sheet of nylon reinforced neoprene sheeting. The sheeting should only be
attached on one side of the joint to allow for the anticipated movement of the integral section. A note for the neoprene sheeting is available in Section 600.

Integral abutments shall be supported on a single, row of parallel piles. If an integral abutment design uses steel H piles, they shall be driven so the pile's web is parallel to the centerline of bearing.

The expansion length at the abutment for an integral structure is considered to be two-thirds (2/3) of the total length of the structure.

For phased construction projects, do not design an abutment phase to be supported on less than three (3) piles.

Phased construction integral backwall details shall have a closure section detailed between sections of staged construction to allow for dead load rotation of the main beams or girders.

The standard bridge drawing shows details for integral abutments with a steel beam or girder superstructure. Cantilevered or turn-back wingwalls shall not be used with integral abutments.

303.2.2.7 SEMI-INTEGRAL ABUTMENTS

Semi-integral abutment use is limited as defined in Section 200 of this Manual. Semi-integral abutments require foundation types that are fixed in position (a single row of piles shall not be used). The expansion and contraction movement of the bridge superstructure is accommodated at the end of the approach slab. Semi-integral design should not be used with curved main members or main members that have bend points in any stringer line.

The expansion length at the abutment for a semi-integral structure is considered to be two-thirds (2/3) of the total length of the structure.

Where an existing structure is being rehabilitated into a semi-integral abutment design, the designer should investigate whether the existing fixed bearings and piers can accept the stresses due to the differential movement caused by the concept of movement for semi-integral abutment designs.

Semi-integral details can be used on wall type abutments, spill-thru type abutments on two or more rows of piles, spread footing type abutments or abutments on drilled shafts.

This design allows the superstructure and the approach slab to move together independent of the abutment. Therefore wingwalls should not be attached to the superstructure and the vertical joints between them should be parallel with the centerline of the roadway.

The joints between superstructure and wingwalls are normally filled with 2 inch [50 mm] performed expansion joint filler material, CMS 705.03.
The horizontal joint in the backwall created between the expansion section of the semi-integral abutment and the beam seat is filled with expanded polystyrene sheet or some equal material to act as form work for the placement of the upper semi-integral abutment concrete. Both the horizontal and vertical joints shall be sealed at the back face of the backwall by use of a 3 foot [900 mm] wide sheet of nylon reinforced neoprene sheeting. The sheeting should only be attached on one side of the joint to allow for the anticipated movement of the integral section.

A standard bridge drawing detailing semi-integral abutment is available.

See Figure 325.

For phased construction projects, do not design an abutment phase to be supported on less than three (3) piles or two (2) drilled shafts.

Phased construction semi-integral backwall details shall have a closure section detailed between sections of staged construction to allow for dead load rotation of the main beams or girders.

### 303.2.3 ABUTMENT DRAINAGE

#### 303.2.3.1 BACKWALL DRAINAGE

The porous backfill immediately behind abutments and retaining walls should be provided according to CMS 518. The porous backfill shall be effectively drained by the use of a corrosion resistant pipe system into which water can percolate. See Section 303.2.3.3 for possible exceptions.

Porous backfill shall be wrapped with filter fabric, CMS 712.09, Type A. The fabric shall cover the vertical face between the porous backfill and the excavation, the bottom of the porous backfill and the excavation and include a 6 inch [150 mm] vertical up turn between the porous backfill and the abutment backwall. The porous backfill excavation should extend up to the horizontal plane of the subgrade or 1'-0" [300 mm] below the embankment surface. The bottom of the porous backfill should extend to the bottom of the abutment footing except when the vertical backface of the abutment footing extends more than 1'-0" [300 mm] out from the vertical backface of the abutment backwall. Then the Porous backfill shall extend down only to the top of the abutment footing. Porous backfill should be 2'-0" [600 mm] thick for its full height behind the abutment and wingwalls except where the vertical backface of the abutment footing extends out 1'-0" [300 mm] or less. A pipe drainage system shall be placed at the bottom of the porous backfill and sloped to allow drainage.

While a single outlet for the pipe drainage systems in the porous backfill can be adequate, the designer should evaluate whether the length of the drainage run requires multiple outlets to supply the porous backfill with a positive drainage system.
The pipe drainage system designs shall make use of standard corrugated plastic pipe segments, tees and elbows (either 90° or adjustable). Overlapping bands should connect pipe segments. Ends of runs, unless intended to function as outlets, should have end caps. While galvanized corrugated pipe has been used for years, the inertness and life expectancy of smooth internal wall plastic corrugated pipe makes this the better material to specify. CMS 518 calls for 707.33, corrugated plastic pipe, if called for in the plans.

303.2.3.2 BRIDGE SEAT DRAINAGE

For full-height or spill-thru non-integral type abutments supporting steel beams, steel girders or prestressed I-beams, the drainage of the bearing seat shall be provided by sloping the bearing seat away from the backwall, except at the bearings.

303.2.3.3 WEEP HOLES IN WALL TYPE ABUTMENTS AND RETAINING WALLS

Positive drainage with a pipe system in porous backfill is preferred.

If a location demands the use of weep holes, the weep holes through the abutment and retaining walls should be 6 inches [150 mm] to 12 inches [300 mm] above normal water or ground line. The porous backfill with filter fabric behind the walls should be shown as extending at least 6 inches [150 mm] below the bottom of the weep holes.

Weep hole type drainage systems should not be used with concrete slope protection as the flow undermines the concrete protection, ultimately causing its failure.

Where sidewalks are located immediately adjacent to wall type abutments or retaining walls, some type of porous backfill collection and drainage system, with pipes if necessary, should be used in lieu of weep holes.

303.2.4 WINGWALLS

Wingwalls shall be of sufficient length to prevent the roadway embankment from encroaching on the stream channel or clear opening. Generally the slope of the fill shall be assumed as not less than 1 vertical to 2 horizontal, and wingwall lengths computed on this basis.

Wingwalls shall be designed as retaining walls.

Cantilevered wingwalls shall not be used with integral abutments, as the walls will create additional pressures due to superstructure movement.
303.2.5 EXPANSION AND CONTRACTION JOINTS

Expansion joints should generally be provided every 90 feet [30 000 mm] with the following exceptions:

A. When the total length of wingwalls and breastwall exceeds 90 feet [30 000 mm] in length, vertical expansion joints should be provided just beyond each side of the superstructure.

B. When the length of a breastwall exceeds 90 feet [30 000 mm] in length, no expansion joint shall be placed under the superstructure. An expansion joint shall be positioned as described in the above paragraph.

An expansion joint shall be filled with preformed expansion joint material, CMS 705.03, or other suitable compressible material.

Expansion joints shall be waterproofed as described in Section 303.2.2.1.b of this Manual.

Contraction joints are not required for abutments.

Reinforcing steel shall not project through expansion or contraction joints.

303.2.6 REINFORCEMENT, “U” AND CANTILEVER WINGS

The minimum amount of reinforcing in the wings, their junctions with the backwall and their supports shall be #5 [#16M] bars on 1'-6" [450 mm] centers, both horizontally and vertically, in both faces.

If a secondary member, such as a short cantilevered turn-back wing, is attached to an abutment or other member, reinforcing steel shall be provided in the secondary member at its connection to the main member and in all parts of the main member stressed by the secondary member, even though small, with adequate lap or bond length at the junction between the several kinds of bars. The probable presence of some tensile stress at various locations, due to the secondary member, must be recognized.

303.2.7 FILLS AT ABUTMENTS

The requirements for fills at abutments, time of settlement, and what and when to use special notes to control field construction of fills are dealt with in earlier preliminary sections of this Manual. The designer should consult the Office of Structural Engineering for the recommended notes to use at a specific project site.
303.3  PIERs

303.3.1  GENERAL

A “free-standing” pier is defined as one that does not depend upon its attachment to the superstructure for its ability to resist horizontal loads or forces.

The width of footing for a free-standing pier generally shall be not less than one-fourth the height of the pier where founded on soil and not less than one-fifth the height of the pier where founded on bedrock.

The minimum width of footing supported by a drilled shaft is the diameter of the shaft.

Where piling is used to support free-standing piers, the distance between centers of outside piles, measured across the footing, generally shall be not less than one-fifth the height of the pier.

Widths greater than the above shall be provided if required for proper bearing area or to accommodate the required number of piles.

The height of the pier shall be measured from the bottom of the footing to the bridge seat.

For multiple span bridges with continuity over piers, where the height of pier is more than 50 percent of the length of superstructure from the point of zero movement to such pier, it may be assumed that the pier will bend or tilt sufficiently to permit the superstructure to expand or contract without appreciable pier stress. This assumption is not permissible if the piers are skewed more than 30 degrees. The above rule does not apply to rigid frame or arch bridges.

Slender columns of either concrete or steel may be designed to bend sufficiently to permit the superimposed superstructure to expand and contract, but the resulting bending stresses shall not exceed the allowable.

During phased construction of a capped pile pier, do not design a pier phase to be supported on less than three (3) piles. For cap and column piers, do not design a phase to be supported on less than two (2) columns.

For a new or replacement structure, individual free-standing columns without a cap are not permitted.

303.3.1.1  BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform to Standard Bridge Drawing CPP-2-94. Also see Section 303.3.2.5 of this Manual.

Pier caps on piles, drilled shafts or on columns are normally a minimum of 3'-0" [915 mm] wide.
This is the standard width used for continuous span prestressed box beams and I-beams. Bearing seat widths of 3'-0" [915 mm], while normally adequate must be verified by the designer of the structure. Large bearings, skew angle, intermediate expansion devices, AASHTO earthquake seat requirements, etc. may require additional width.

303.3.1.2 PIER PROTECTION IN WATERWAYS

See Section 200 of this Manual for piling protection requirements and Section 600 for a plan note to be added to design drawings when the Capped Pile Pier Standard Bridge Drawing is not referenced.

303.3.2 TYPES OF PIERS

303.3.2.1 CAP AND COLUMN PIERS

The cantilever arms of cap and column piers shall be designed for the same impact fraction as the superstructure. When designing the cantilever portions of cap and column piers, the design moments shall be calculated at the actual centerline of the column. A reduction in design moments based on AASHTO Section 8.8.2 will not be acceptable for the cantilever portion.

The uppermost layers of longitudinal reinforcing steel in the pier cap shall not be lap spliced at the centerline of a column.

Longitudinal reinforcing shall conform to AASHTO. Round columns shall be reinforced with spiral reinforcing placed directly outside the longitudinal bars.

Round columns are preferred and normally should be 36" [915 mm] diameter.

Cap dimensions should be selected to meet strength requirements and to provide necessary bridge seat widths. Caps should be cantilevered beyond the face of the end column to provide approximately balanced moments in the cap. The end of the cantilevered caps should be formed perpendicular to the longitudinal centerline of the cap to allow for uniform development lengths for the reinforcing steel. Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap. Cantilevered caps may be eliminated for waterway crossing where debris removal access is an issue.

Minimum column diameters of 36" [915 mm] are generally used with spiral reinforcing. Spirals are made up of #4 [#13M] bars at 4½" [115 mm] c/c pitch with a 30" [765 mm] outside core diameter. Using the circumference of the spiral as the out to out of the reinforcing steel bar, this column size normally has a relatively small ratio of the actual axial load to the column’s axial load capacity (i.e. less than 2/3). Therefore, while this spiral reinforcement does not conform to AASHTO requirement 8.18.2.2.3 it is acceptable under AASHTO 8.18.2.1 if the ratio of actual loads to design capacity is under 2/3.
For columns where the ratio of actual axial load to axial capacity is greater than $2/3$, the spiral reinforcing should conform to AASHTO Section 8.18.2.

In no case shall column reinforcement not meet minimum cross section area, shrinkage and temperature requirements of AASHTO.

303.3.2.2 CAP AND COLUMN PIERS ON PILES

Piers supported on piles generally should have separate footings under each column.

Column piers shall have at least 4 piles per footing.

For grade separation structures, the top of the pier's footings should be a minimum of 1'-0" [300 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is located in a raised earth median barrier.

303.3.2.3 CAP AND COLUMN PIERS ON DRILLED SHAFTS

Where columns are supported on a drilled shaft foundation, the drilled shaft should be at least 6 inches [150 mm] larger in diameter than the column. This is to allow for field location tolerances of the drilled shaft. A drilled shaft foundation is defined as starting 1 foot [0.3 meter] below ground level or 1 foot [0.3 meter] above normal water.

303.3.2.4 CAP AND COLUMN PIERS ON SPREAD FOOTINGS

Cap and column piers on spread footings, placed on existing soils or on embankment fills, should have continuous footings which should extend beyond the center of the end column a distance equal to approximately $1/3$ of the distance between the end column and the adjacent column, in order to provide approximately balanced moments.

Cap and Column piers with spread footings on bedrock shall have separate footings under each column.

For grade separation structures, the top of pier footings should be a minimum of 1'-6" [450 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is located in a raised earth median barrier. In no case should the bottom of the footings in existing soil or on embankment fills be above the frost line.

The width of footing for a free-standing pier generally shall be not less than one-fourth the height of the pier where founded on soil and not less than one-fifth the height of the pier where founded on bedrock.
303.3.2.5 CAPPED PILE PIERS

Steel H piles shall be a minimum HP12x53 [HP310 x 79]. The piles should be shown on the plans with the flanges of the H-section perpendicular to the face of the pier cap.

The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 8 inches [200 mm].

The diameter of the exposed portions of cast-in-place reinforced concrete piles generally should be 16 inches [400 mm], but if exposed length, design load or other conditions make it necessary, larger diameter cast-in-place piles should be used. Cast-in-place piles shall be reinforced with a reinforcement cage composed of 8-#6 [#19M] reinforcing bars with a 12 inch [300 mm] outside diameter, #4 [#13M] spiral, with a 12 inch [300 mm] pitch. The cage length should extend from the finished top of the pile to 15 feet [5 meters] below ground level. The reinforcing steel shall be shown in the structure's reinforcing bar list and be included in item 507 for payment. The use of cast-in-place piles greater than 16 inches [400 mm] in diameter will require an increase in the width of the cap of Standard Bridge Drawing CPP-2-94. See Section 303.3.3.

Exposed H piles and unreinforced concrete piles shall have pile protection. Refer to the description in Standard Bridge Drawing CPP-2-94. A plan note is also available. Also See Section 200 for a description of pile protection.

For pile embedment requirements into concrete, see Section 303.3.3.

An optional construction joint shall be shown at the top of pier caps for reinforced concrete slab bridges. This joint is optional as some machine finishing equipment for slab bridge decks require a uniform depth of freshly placed concrete in order to obtain best results.

The design of the cap for a capped pile pier supported on bearing piles should be based on the assumption that any one pile in any three consecutive piles does not have sufficient bearing to support axial loads. The cap design doesn’t need to assume the end piles cannot support axial loads.

Although actual performance of this type of pier indicates this condition to be rare, this conservatism is recommended.

For phased construction projects, do not design a pier or abutment phase to be supported on less than three (3) piles.

303.3.2.6 STEEL CAP PIERS

If at all possible this alternative should not be selected. This is a fracture critical design that has historically shown both steel member and weld metal cracking problems. As specified in Section 301.2, these structure types require a concurrent detail design review to be performed by
If a steel box girder is required as a pier cap, the design shall allow reasonable access to the interior for maintenance, inspection and repair purposes. The physical dimensions of the box shall be large enough to allow access to the interior for inspection. Access hatches of the box girder should be bolted and sealed with a neoprene gasket. Access hatches should also be light enough for an inspector to easily remove them. One recommended lightweight material is ABS plastic.

Designers shall ensure that all governmental agency regulations regarding to enclosed spaces, ventilation, lighting, etc. are complied with within any enclosed steel pier cap design.

Box designs with cut away webs to allow for stringers to continue through the box are generally not considered acceptable alternatives.

Situations that require stringers to be continuous through, and in the same plane with a steel pier cap or crossbeam should be avoided if at all possible.

Designers should review all weld details for possible fatigue problems. Consult the Office of Structural Engineering for assistance in this area.

303.3.2.7 POST-TENSIONED CONCRETE PIER CAPS

Where vertical clearance or geometric considerations require stringers to be continuous through and/or in the same plane as the pier cap, a post-tensioned concrete cap should be investigated as a first option in lieu of a steel pier cap. However, this is a non-redundant design, and, as specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

303.3.2.8 T-TYPE PIERS

The cantilever arms of T-type piers are to be designed for the same impact fraction AASHTO requires for the superstructure.

In the cap of a T-type pier, the top layer of reinforcing bars shall extend the full length of the cap and be turned down at the end face the necessary development length. The second layer of reinforcing steel shall extend into the stem of the pier at least the necessary development length plus the depth of the cantilever at its connection to the stem.

Lateral ties are required for T-type and wall type piers. As a minimum, lateral ties shall consist of #4 [#13M] bars spaced at 3'-0" [915 mm] centers both horizontally and vertically.
303.3.2.9  PIER USE ON RAILWAY STRUCTURES

For clearance requirements see Section 200 of this Manual. Items listed in Section 200 are only general rules and vary from railroad to railroad. The designer shall confirm with the individual railroad the actual physical dimension and design requirements.

303.3.2.10  PIERS ON NAVIGABLE WATERWAYS

Piers in the navigation channel of waterways, unless protected from collision by an adequate fendering system, shall be designed to resist collision forces based on AASHTO Guide Specification for Vessel Collision Design of Highway Bridges.

303.3.2.11  PIER CAP REINFORCING STEEL STIRRUPS

Stirrups for concrete beams of constant depth, such as pier caps, should be detailed using either 2 “U” bars with the vertical legs long enough to furnish the required lap length or a single bar closed type stirrup with 135° bends at both ends of the rebar. The single bar closed type stirrup should only be selected when minimum required lap lengths cannot be provided with the “U” type stirrup. The corner with the 135° bends of the closed type stirrup should be placed in the compression zone of the concrete beam.

303.3.3  FOOTING ON PILES

Piles supporting capped pile piers shall be embedded 1'-6" [450 mm] into the concrete cap. Other substructure units on a single row of piles should have the piles embedded 2'-0" [600 mm] into the concrete. A 1'-0" [300 mm] embedment depth into the concrete footing is required for all other cases. In every case, there shall be at least 1'-6" [450 mm] cover over top of pile.

The distance from the edge of a footing to the center of a pile shall be not less than 1'-6" [450 mm]. The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 8 inches [200 mm].

303.4  FOUNDATIONS

303.4.1  MINIMUM DEPTH OF FOOTINGS

Footings, not exposed to the action of stream currents, should be founded based on the following minimum depths:

A. For grade separation structures the top of footing shall be a minimum of 1 foot [300 mm] below the finished ground line. The top of footing should be at least 1 foot [300 mm] below the bottom of any adjacent drainage ditch.
B. The bottom of footing shall not be less than 4 feet [1200 mm] below, measured normal to, the finished groundline.

C. Due to the probability of stream meander, pier footings of waterway crossings in the overflow section should not be above channel bottom unless the channel slopes are well protected against scour. Founding pier footings at or above the flow line elevation is discouraged.

Where footings are founded on bedrock (note that undisturbed shale is rock) the minimum depth of the bottom of the footing below the stream bed, D, in feet [meters], shall be as computed by the following:

\[ D = T + 0.50Y \]

Where:
- \( T \) = Thickness of footing in feet [meters]
- \( Y \) = distance from bottom of stream bed to surface of bedrock in feet [meters]

The footing depth from the above formula shall place the footing not less than 3 inches [75 mm] into the bedrock.

Adjustment may be made to the minimum depth of the bottom of a footing due to actual frost line at the structure site.

### 303.4.1.1 FOOTING, RESISTANCE TO HORIZONTAL FORCES

The safety factor against horizontal movement at the base of a structure; i.e., the ratio of available resistance to movement to the forces tending to cause movement, shall be not less than 1.5 except as specified below for footings on bearing piles.

The friction resistance between a concrete footing and a cohesionless soil may be taken as the vertical pressure on the base times the coefficient of friction “\( f \)” of concrete on soil.

- For coarse-grained soil without silt.......................................................... \( f = 0.55 \)
- For coarse-grained soil with silt.............................................................. \( f = 0.45 \)
- For silt........................................................................................................ \( f = 0.35 \)

If the footing bears upon clay, the resistance against sliding shall be based upon the cohesion of the clay, which may be taken as one-half the unconfined compressive strength provided, however, that the frictional resistance against sliding should not be considered to be greater than that obtained using the coefficient “\( f \)” of 0.35. If the clay is very stiff or hard, the surface of the clay
shall be roughened before the concrete is placed.

If the footing bears upon bedrock, consideration shall be given to features of the bedrock structure that may constitute planes of weakness such as laminations or inter-bedding. If there is no evidence of such weakness, the coefficient of friction "f" may be taken as 0.55 for shale and 0.7 for rock.

If the frictional or shearing resistance of the supporting material is inadequate to withstand the horizontal force, one or more of the following means shall provide additional resistance:

A. Increase the footing width and/or use footing keys.

B. Make allowance for the passive pressure developed at the face of the footing.

C. Use battered piles, footing struts, sheeting or anchors.

For footings with keys, allowance shall be made for the shearing resistance furnished by the supporting material at the elevation of the bottom of the key. Keys generally shall be located within the middle-half of the footing width.

For footings on piles, no allowance shall be made for the frictional resistance of the footing concrete on soil. For such footings, the horizontal component of the axial load on battered piles shall be taken at full value, without the application of the safety factor of 1.5. The safety factor shall apply for any required additional resistance provided by the passive pressure developed in the soil in front of such foundations. The above may be expressed by the following formula:

\[
\frac{A}{B - C} \geq 1.5
\]

Where:
- \(A\) = passive pressure developed in the soil in front of the footing
- \(B\) = force tending to cause movement
- \(C\) = horizontal component of the axial load in battered piles

For structures on piles or soils, the passive resistance developed on the face of a foundation (assuming a level ground surface) may be based on an equivalent passive fluid weight \(W_p\) (lb/ft\(^3\) [kN/m\(^3\)]) for the undisturbed material encountered or anticipated. The equivalent passive fluid weight may be based on the following equation:

\[
W_p = W \tan^2 (45 + \frac{q}{2}) \text{ lb/ft}^3 \ [\text{kN/m}^3] 
\]

Where:
- \(W\) = unit soil weight, lb/ft\(^3\) [kN/m\(^3\)]
- \(q\) = angle of internal friction, in degrees.
For soft clays to coarse compact sand and gravels, $W_p$ may vary from 100 to 800 lb/ft$^3$ [15.7 to 125.7 kN/m$^3$], respectively. For firm soils $W_p$ may be taken as equal to 300 lb/ft$^3$ [47.1 kN/m$^3$]. The total passive resistance ($P_p$) may therefore be based on the following equation:

$$P_p = 0.5 \times W_p \times (H_1^2 - h_1^2), \text{ lb/ft} \ [\text{kN/m}]$$

Where:

“$H_1$” and “$h_1$” are the effective depth and the surcharge depth respectively, both in feet [meters]. For structures without piles the effective depth “$H_1$” may be measured from the ground surface to the bottom of the footing or footing key. For structures on piles the effective depth may be extended below the bottom of the footing a depth equal to one-fourth the penetrated length of the piles, but not to exceed 5 feet [1.5 meters]. The surcharge depth, “$h_1$”, is the larger of the depth below ground surface affected by seasonal changes or the depth of uncompacted backfill. In estimating the above depths allowance shall be made for the possibility of future loss of surface material by erosion, scour or possible excavation. For a foundation on piling the effective width for computing passive resistance (on the piling) may be equal to the sum of the pile diameters, but not to exceed the length of the footing.

If the preceding methods do not furnish sufficient horizontal resistance, the use of sheet piling to increase the effective depth ($H_1$) of the passive resistance below the elevation of the bottom of the footing may be incorporated in the design. Such sheet piling shall be rigidly attached to the footing and cantilevered downward. This sheet piling shall have sufficient section to resist the cantilever moment produced by the passive resistance developed in adjacent soil and shall have a connection to the footing adequate to provide the required fixed condition. See Section 303.2.2.1.

Alternate methods of analysis may be acceptable.

### 303.4.1.2 LOCATION OF RESULTANT FORCES ON FOOTINGS

Footings shall be designed to distribute the combined total vertical and horizontal forces in such a manner that the required structural stability is obtained and that the allowable unit bearing values of the subfoundation materials are not exceeded.

For footings on soils, the resultant of all forces generally should intersect the base of the footing within its middle third.

For footings on bedrock, the resultant of all forces generally should intersect the base of the footing within its middle half or the footing should be embedded in bedrock to a depth sufficient to prevent footing rotation.

Where the structural stability of the member is obtained by its attachment to some other stable portion of the structure, the limitations of the preceding two paragraphs may not apply.
303.4.1.3  **REINFORCING STEEL IN FOOTINGS**

Secondary reinforcing steel in a footing generally should be placed under the main steel.

For footings on piles the reinforcing bars shall be placed near the bottom of the footing rather than at the top of the piles.

If the footing dowels (footing to wall or column) are provided, a bent portion of the dowel should lie in the plane of the bottom footing bars.

For piers in embankment slopes the minimum dowel size and spacing should be #8 [#25M] at 1'-0" [300 mm] centers. For full length wall type piers not in embankment slopes and without earth overturning forces the minimum dowel size and spacing should be #6 [#19M] at 1'-0" [300 mm] centers.

At locations where the concrete unit tensile stress approaches the allowable for un-reinforced concrete, reinforcing steel should be provided. This applies particularly to the bottom of the toe and the top of the heel of a footing for a cantilever-type retaining wall or abutment where the footing is thin in proportion to the toe and heel projections. It may also apply to the tops of footings for tall piers where unanticipated longitudinal or lateral movements may induce tension in the tops of the footings.

303.4.2  **PILE FOUNDATIONS**

303.4.2.1  **PILES, PLAN SHEET REQUIREMENTS**

For record and project use, a unique number shall individually identify each pile for a structure. The designer may choose to number each pile on the individual substructure plan sheet or on a separate pile layout sheet.

Listed below are definitions to commonly specified pile lengths:

A.  Estimated Length = Pile Cutoff Elevation - Pile Tip Elevation
   Round Estimated Length up to the nearest 5 ft [1 m]. Section 200 requires the Designer to provide the Estimated Length on the site plan.

B.  Order Length = Estimated Length + 5 ft [1.5 m]
   The Designer shall provide the order length for each pile in the Structure General Notes. Refer to Section 600.

C.  Furnished Length = Order Length x No. of Piles
   Include in the table of Estimated Quantities.

D.  Driven Length = Estimated Length x No. of Piles
Include in the table of Estimated Quantities.

303.4.2.2 PILES, NUMBER & SPACING

The designer shall comply with the following maximum center-to-center spacing of piles:

A. In capped pile piers, 7.5 feet [2300 mm].
B. In capped pile abutments, 8 feet [2500 mm].
C. In stub abutments, front row, 8 feet [2500 mm].
D. In wall type abutments and retaining walls, front row, 7 feet [2100 mm].
E. Cap and column piers should have at least 4 piles per individual footing.

303.4.2.3 PILES BATTERED

The path of battered piles should be checked to see that the piles remain within the right-of-way and do not interfere with piles from adjacent and existing substructure units nor conflict with portions of staged construction.

In general, a batter of 1:4 is considered desirable, but in cases where sufficient resistance is not otherwise attainable, a batter of 1:3 may be specified.

Piles should be battered to resist the stream forces. Battered piles also should be provided where necessary to avoid settlement due to group action by increasing the periphery of the soil mass.

Abutment piles should be battered normal to the centerline of bearings.

Piles fewer than 15 feet [5 meters] in length should not be battered.

When friction battered piles are specified, include note [30d] from Section 600 in the plans.

303.4.2.4 PILES, DESIGN LOADS

The pile’s Ultimate Bearing Value, based on calculation of dead and live load transferred to the piles shall be given in the structure General Notes.

Ultimate Bearing Value load is equal to the actual unfactored design load multiplied by a safety factor of two (2).

The largest of these calculated individual pile Ultimate Bearing Value loads for each
substructure unit should be used as the Ultimate Bearing Value for that substructure unit. This value for each substructure shall be listed in the structure General Notes.

The following table for H-piles should be used for selecting the required pile size based on the calculated Ultimate Bearing Value load for each substructure unit.

<table>
<thead>
<tr>
<th>H Pile Size</th>
<th>Maximum Design Load</th>
<th>Ultimate bearing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP10x42 [HP250X62]</td>
<td>75 Tons [650 kN]</td>
<td>150 Tons [1300 kN]</td>
</tr>
<tr>
<td>HP12x53 [HP310X79]</td>
<td>95 Tons [850 kN]</td>
<td>190 Tons [1700 kN]</td>
</tr>
<tr>
<td>HP14x73 [HP360X108]</td>
<td>130 Tons [1150 kN]</td>
<td>260 Tons [2300 kN]</td>
</tr>
</tbody>
</table>

Design load values for H piles are based on a maximum service load stress of 12.5 ksi [86 MPa] for Grade 50 steel.

The following table for pipe piles should be used for selecting the required pile size based on the calculated Ultimate Bearing Value load for each substructure unit.

<table>
<thead>
<tr>
<th>Pipe Pile Diameter</th>
<th>Maximum Design Load</th>
<th>Ultimate bearing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>12&quot; [300 mm]</td>
<td>50 Tons [450 kN]</td>
<td>100 Tons [900 kN]</td>
</tr>
<tr>
<td>14&quot; [350 mm]</td>
<td>70 Tons [650 kN]</td>
<td>140 Tons [1300 kN]</td>
</tr>
<tr>
<td>16&quot; [400 mm]</td>
<td>90 Tons [800 kN]</td>
<td>180 Tons [1600 kN]</td>
</tr>
</tbody>
</table>

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the pile size selected, whether H pile or Pipe pile, but the calculated Ultimate Bearing Value load of the substructure unit or units.

Maximum specified pile spacings and maximum allowable Ultimate Bearing loads should be utilized to minimize the number of piles.

303.4.2.5 PILES, STATIC LOAD TEST

The Designer shall specify a Static Load Test when the total estimated pile length for an individual structure exceeds 10,000 ft [3000 m] for piling of the same size and Ultimate Bearing Value. Static load testing is not required for piling driven to refusal on bedrock.

The Designer shall specify one subsequent static load test for each additional 10,000 ft [3000 m] increment of estimated pile length. Each static load test requires two dynamic testing items and two restrike items. Restrikes are a useful tool to determine if a driven pile gains or loses capacity over time.
The results of both the static and dynamic testing shall be forwarded to the Office of Structural Engineering to the attention of the Foundations Engineer. Refer to Section 600 for a General Note to include in the plans.

303.4.2.6 PILES, DYNAMIC LOAD TEST

The Department now requires dynamic load testing to establish the driving criteria (i.e. blow count) for all piling not driven to refusal on bedrock. The dynamic testing and resulting wave analysis has replaced the Engineering News Record Formula, used in previous issues of the CMS.

For an individual structure, the Designer shall specify one dynamic load testing item for each pile size. If multiple pile capacities are required for a given pile size, the Designer shall specify one testing item for each Ultimate Bearing Value. When static load tests are required, provide two dynamic load testing items and two restrike items for each static load test item.

The driving criteria for battered piles will be determined in the field as a function of a dynamically tested vertical pile of the same Ultimate Bearing Value. When battered piles are specified, refer to Section 600 for a General Note to include in the plans.

One dynamic load testing item consists of testing a minimum of 2 piles and performing a CAPWAP analysis on one of the two piles. One restrike item consists of performing dynamic testing on two piles and performing a CAPWAP analysis on one of the two piles.

303.4.2.7 PILE FOUNDATION – DESIGN EXAMPLE

The following example for a 6-span bridge shall be used as a guide for specifying pile testing and estimated quantities for pile foundations.

Rear Abutment ~
30 - 12" C.I.P. Reinforced Concrete Piles
20 piles installed vertical & 10 piles battered
Ultimate Bearing Value = 76 ton
Estimated Length = 65 ft
Order Length = 70 ft (Total Length = 2100 ft)

Requires 1 dynamic load-testing item.

Piers 1, 2, 3, & 4 ~
80 - 14" C.I.P. Reinforced Concrete Piles at each pier
56 piles installed vertical & 24 piles battered
Ultimate Bearing Value = 125 ton
Estimated Length = 70 ft
Order Length = 75 ft (Total Length = 24,000 ft)

The total length (24,000 ft) requires 1 static load test item and 1 subsequent static load test. Each static load test requires 2 dynamic load testing items and 2 restrike items.

Pier 5 ~
52 - 14" C.I.P. Reinforced Concrete Piles
36 piles installed vertical & 16 piles battered
Ultimate Bearing Value = 135 ton
Estimated Length = 85 ft
Order Length = 90 ft (Total Length = 4680 ft)

The difference in Ultimate Bearing Value between piers 1, 2, 3 & 4 and pier 5 requires 1 dynamic testing item.

Forward Abutment ~
30 - 12" C.I.P. Reinforced Concrete Piles
20 piles installed vertical & 10 piles battered
Ultimate Bearing Value = 76 ton
Estimated Length = 75 ft
Order Length = 80 ft (Total Length = 2400 ft)

No additional dynamic load testing items are required.

For this example, the Designer should include notes [30], [30c] and [30d] from Section 606.2 in the General Notes. Note [30] should be modified as follows:

**PILE DESIGN LOADS (ULTIMATE BEARING VALUE):** The Ultimate Bearing Value is 76 ton per pile for the rear and forward abutment piles. The Ultimate Bearing Value is 125 ton per pile for Pier 1, 2, 3, and 4 piles and 135 ton per pile for Pier 5 piles.

Abutment Piles:
30 piles 70 ft long, order length (Rear)
30 piles 80 ft long, order length (Forward)
1 dynamic load testing item

Pier 1, 2, 3, and 4 Piles:
320 piles 75 ft long, order length
1 static load test item
1 subsequent static load test item
4 dynamic load-testing items
4 restrike items
Pier 5 Piles:
52 piles 90 ft long, order length
1 dynamic load testing item

The Designer should provide the following items in the Estimated Quantities:

<table>
<thead>
<tr>
<th>Item</th>
<th>Extension</th>
<th>Total</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>506</td>
<td>11100</td>
<td>Lump Sum</td>
<td>Static Load Test</td>
<td></td>
</tr>
<tr>
<td>506</td>
<td>12200</td>
<td>1 Each</td>
<td>Subsequent Static Load Test</td>
<td></td>
</tr>
<tr>
<td>507</td>
<td>00500</td>
<td>4200 ft</td>
<td>12&quot; Cast-In-Place Reinforced Concrete Piles, Driven</td>
<td></td>
</tr>
<tr>
<td>507</td>
<td>00550</td>
<td>4500 ft</td>
<td>12&quot; Cast-In-Place Reinforced Concrete Piles, Furnished</td>
<td></td>
</tr>
<tr>
<td>507</td>
<td>00600</td>
<td>26,820 ft</td>
<td>14&quot; Cast-In-Place Reinforced Concrete Piles, Driven</td>
<td></td>
</tr>
<tr>
<td>507</td>
<td>00650</td>
<td>28,680 ft</td>
<td>14&quot; Cast-In-Place Reinforced Concrete Piles, Furnished</td>
<td></td>
</tr>
<tr>
<td>523</td>
<td>20000</td>
<td>6 Each</td>
<td>Dynamic Load Testing</td>
<td></td>
</tr>
<tr>
<td>523</td>
<td>20500</td>
<td>4 Each</td>
<td>Restriking</td>
<td></td>
</tr>
</tbody>
</table>

303.4.3 DRILLED SHAFTS

3'-6" [1065 mm] diameter drilled shafts for piers and 3'-0" [915 mm] diameter shafts for abutments are normally used.

The diameter of bedrock sockets of a drilled shaft are generally 6 inches [150 mm] less in diameter than the diameter of the drilled shaft above the bedrock elevation. The 6 inch [150 mm] downsize can be eliminated for abutment shafts. Reinforcing steel cages should be based on the bedrock socket diameter.

The drilled shaft diameter for the abutment shafts can be shown as one constant diameter for the full length of the drilled shaft (through bedrock and through soil).

Spiral reinforcement used in the drilled shaft is normally a #4 [#13M] bar at a 4½ inch [115 mm] pitch with a spiral diameter of 6 inches [150 mm] less, out to out of spiral cage than the drilled shaft diameter. (Note AASHTO specifications do not recognize a 4½ inch [115 mm] pitch as meeting spiral requirements definition 8.18.2.2.3) When steel casing is left in place, a pitch of 12 inches [300 mm] should be used for the spiral reinforcing.

The minimum clear distance between longitudinal and lateral reinforcement shall not be less than 5 times the maximum aggregate size. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

Drilled shafts with diameters of less than 3'-0" [915 mm] are not recommended.

The diameter of the drilled shafts should be 6 inches [150 mm] larger than the pier column diameter so that if the drilled shaft is slightly misaligned, the pier column can still be placed at plan location, although the pier column would not be exactly centered on a misaligned-drilled shaft.
For record and project use, each drilled shaft for a structure shall be individually identified by a unique number. The designer may choose to number the drilled shafts on the individual
substructure plan sheet or on a separate drilled shaft foundation layout sheet.

A construction joint between the drilled shaft and any column will be required. Therefore the designer will need to specify reinforcing steel, incorporating the required lap splices, at the construction joint.

The designer should develop a lap splice that will allow both for required lap and minimum cover due to mis-alignment of the drilled shaft versus the column. Possible alternatives are two cages, one for the drilled shaft diameter and a second splice cage for the lap to the column.

When the exposed length of the pier columns is relatively short, one full length reinforcing steel cage, from the bottom of the drilled shaft up into the pier cap, should be designed. The steel cage should be designed to provide a 3 inch [75 mm] concrete cover within the pier column.

When the drilled shaft is socketed into bedrock, the quantity of the reinforcing steel in the drilled shaft, including the portion extending into the rock socket, should be included with Item 524 “Drilled Shaft, Above Bedrock” for payment. For drilled shafts with friction type design where the tip elevation is known, the reinforcing steel should be paid under Item 524, Drilled Shafts. The Designer shall also specify the reinforcing steel to be epoxy coated according to 709.00.

A general note as listed in Section 600 will be required.

The top of the drilled shaft is defined as 1 foot [0.3 meter] above normal water elevation, for piers in water, and 1 foot [0.3 meter] below the ground surface for piers not in water.

### 303.5 DETAIL DESIGN REQUIREMENTS FOR PROPRIETARY RETAINING WALLS

Supplemental Specification 840 defines the requirements for construction and design for internal stability for Mechanically Stabilized Earth (MSE) walls. The project plans shall include a reference to SS 840 when MSE walls are shown. Special provisions are required for other types of proprietary walls.
303.5.1 WORK PERFORMED BY THE DESIGN AGENCY

The Design Agency is responsible for providing sufficient information in the plans such that, prior to submitting a bid, the Contractor can select a proprietary company to design the internal stability of the wall after the project is awarded. Detail each wall on a project separately. As a minimum, the project plans for each wall location shall provide the following:

A. Plan View of the wall showing: (Refer to Figure 329)

1. Wall location with station and offset with respect to the centerline of construction for each critical point
2. All complex geometry information
3. Pay limits for wall and roadway quantities
4. North Arrow
5. Locations of typical sections for (C.) below
6. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions (Refer to Section 209.3 for drainage and Section 301.7 for utility locations)
7. Parapet/barrier locations
8. Limits of proposed wall excavation
9. Locations of sheeting and bracing

   If sheeting and bracing is required according to BDM Section 208, provide a pay item for Item 503 – Cofferdams and Excavation Bracing.

10. Select Granular Backfill drainage locations

    Perforated plastic pipe, CMS 707.33, wrapped with filter fabric shall be located as low as possible within the select granular backfill while still providing positive gravity flow in the pipe to an outlet. The pipe shall be located near the back side of the leveling pad and near the free end of the soil reinforcement. The pipe shall be continuous and sloped to provide a positive gravity flow to an outlet. The approximate location of the outlet shall be shown on the plan view. Drainage pipe without perforations shall be used outside the limits of the select granular backfill. If the proprietary wall supports an abutment, provide backfill drainage in accordance with Section 303.2.3.1.
B. Elevation of the wall showing: (Refer to Figure 329)

1. Station and elevation for each critical point on the wall
2. Finished ground surface elevations for each critical point on the wall
3. Leveling pad showing the minimum dimension from the finished ground line to the top of the pad.
4. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions
5. Backfill drainage
6. Approximate locations of slip joints

C. Typical Sections showing: (Refer to Figures 330, 331, 332, 333, 333A & 333B)

1. Coping details
2. Parapet and sleeper slab details
3. Abutment footing details including the dimensions from the back of the proprietary wall to the centerline of bearing at the abutment, dimensions from the back of the proprietary wall to the toe of the abutment footing, and dimensions from the back of the proprietary wall to the centerline of the nearest row of piles.
4. Minimum clearance between the bottom of the footing/sleeper slab and the uppermost wall reinforcement strap. Six inches [150 mm] is preferred.
5. Locations of abutment footing, piles, utilities, catch basins, and other possible obstructions
6. Backfill drainage
7. Soil reinforcements attached to abutments

Regardless of abutment type and foundation type, one row of soil reinforcements shall be attached to the backside of the abutment footing. These additional reinforcements are necessary to resist horizontal bridge and backwall forces, and prevent load transfer to the coping and facing panels. To estimate select granular backfill quantities, Designers may assume these additional reinforcements are the same length as those attached to the facing panels.
8. Limits of select granular backfill

Show the limit of the select granular. The top elevation of the select granular backfill shall be at least six inches above the uppermost layer of soil reinforcement, but not lower than six inches above the bottom of the abutment footing.

9. Limits of wall excavation

Supplemental Specification 840 requires a minimum one foot undercut beneath the leveling pad elevation for all MSE walls. If more undercut is required, show it on the plans. The backfill material is specified in SS 840.

10. Pay limits of wall and roadway quantities

11. Pile sleeves (if required)

Pile sleeves shall be shown extending from the bottom of the abutment footing to the bottom of the wall excavation

12. Location of sheeting and bracing (if required)

13. Limits of concrete sealer

D. Requirements for wall surface textures or other aesthetic treatments (i.e. show panel size and shape restrictions specific to the project in the plans)

E. Wall design criteria including:

1. Allowable bearing capacity at the base of the reinforced soil mass

2. Vertical dead and live loads, horizontal loads and actual bearing pressure applied to the reinforced soil mass from the bridge

Plan notes are provided in Section 600.

F. Final copy of the Special Provisions for proprietary wall types other than MSE walls.

G. Estimated Quantities Table (list each wall on a project separately)

Include all pay items listed in SS 840. Also include as necessary; Item 203 – Embankment; Item 512 – Sealing of concrete surfaces (epoxy urethane); and Item 503 - Cofferdams and Excavation Bracing.
303.5.2 WORK PERFORMED BY THE PROPRIETARY WALL COMPANIES

The proprietary wall companies will be responsible for designing the internal stability of the wall in accordance with the project plans and either Supplemental Specification 840 for MSE walls or the special provisions for other proprietary wall types.
304 RAILING

304.1 GENERAL

All railing on bridges with the type of “Proposed Work” listed below, shall meet acceptance criteria contained in NCHRP Report 350 or AASHTO “Manual for Assessing Safety Hardware” (MASH). The minimum acceptance level shall be TL-3 unless supported by the AASHTO “Guide Specifications for Bridge Railing”, 1989.

“Proposed Work” that requires NCHRP Report 350 or MASH compliance includes:

A. New construction on all routes.
B. Complete deck replacements on all routes.
C. Replacement of deteriorated deck edges on all routes.
D. Superstructure widenings on all routes.
E. Rigid concrete overlays on NHS routes.
F. Rigid concrete overlays on non-NHS routes with design speeds of 40 mph or greater.

For other types of “Proposed Work”, including roadway railing upgrade projects, NCHRP Report 350 or MASH compliance of the bridge railing is not warranted, but a positive connection between the approach railing and the existing bridge railing is required.

Bridge railings that have been found acceptable under the crash testing acceptance criteria defined in NCHRP Report 230 and the AASHTO Guide Specification for Bridge Railing, 1989 including all interims, will be considered as meeting the requirements of NCHRP Report 350 without further testing as indicated in the following table.

<table>
<thead>
<tr>
<th>Bridge Railing Testing Criteria</th>
<th>Acceptance Equivalencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCHRP 350</td>
<td>TL-1</td>
</tr>
<tr>
<td>NCHRP 230</td>
<td>MSL-1</td>
</tr>
<tr>
<td>AASHTO Guide Specification</td>
<td>PL-1</td>
</tr>
</tbody>
</table>

Section 301.1 of this Manual lists standard ODOT bridge railing types available along with the corresponding NCHRP Report 350 level of acceptability. For non-standard railing designs, a review submission, concurrent with the Structure Type Study, shall be made to the Office of Structural Engineering as stated in Section 201 of this Manual. The design of all non-standard railing systems shall be based on the NCHRP Report 350 or MASH level of acceptability. Designers may be required to submit actual crash test report data to verify the level of acceptability of a proposed design.
Modifications to the ODOT standard railing types or other NCHRP 350 or MASH approved railing system should be avoided. Additional structural steel tubing added to satisfy pedestrian concerns does not require additional crash testing provided these elements do not protrude nearer to the roadway than the rail elements on the tested design and they do not present any type of snagging potential to an impacting vehicle. If an accepted crash tested railing system is modified, the face geometry (i.e. offset, rail height, spacing, etc.) shall match the tested design and the static strength and deflections shall remain at least equal to the tested design. Include with the preliminary design submission to the Office of Structural Engineering, strength and deflection calculations to support these modifications. The calculations shall follow the procedure defined in the AASHTO LRFD Bridge Design Specifications, 2nd Edition, Sections A13.1-3. The intent of any modification shall be to maintain the original NCHRP 350 or MASH acceptability level.

All railing elements fabricated with ASTM A500 steel tubing shall specify a drop-weight tear test per CMS 707.10. Provisions shall be made at tube splices for expansion and contraction. Steel railing systems shall also allow for structural movement at expansion joints without adversely affecting the system’s level of acceptability.

Aesthetically pleasing railing systems have been successfully crash tested but are for use only where TL-2 acceptability requirements are allowed. These systems include the Texas Classic Traffic Railing, Type T411 with open windows, a smooth stone masonry barrier with reinforced concrete core wall and an artificial stone precast concrete barrier. Detailed information regarding the latter two systems may be found in FHWA Report No. FHWA-RD-90-087 “Guardrail Testing Program: Final Report”, June 1990 and FHWA Report No. FHWA-SA-91-051 “Summary Report on Selected Aesthetic Bridge Rails and Guardrails”, June 1992.

The recommended railing design for bridges with combination vehicular and pedestrian traffic is detailed in Standard Bridge Drawing BR-2-98. Other designs are allowed as previously mentioned above, provided the following requirements are met:

A. The curb height shall be 8".
B. The sidewalk width shall be 5'-0" or greater.

A pedestrian railing may be used in lieu of a crash tested barrier at the deck edge provided a crash tested barrier system meeting the minimum requirements for the specific location is used to separate the vehicular and pedestrian traffic. Pedestrian railing shall be designed in accordance with AASHTO.
### 304.2 STANDARD RAILING TYPES

<table>
<thead>
<tr>
<th>Drawing No.</th>
<th>Description</th>
<th>NCHRP Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>BR-1</td>
<td>36&quot; Deflector Parapet Type</td>
<td>TL-4</td>
</tr>
<tr>
<td>BR-1</td>
<td>42&quot; Deflector Parapet Type</td>
<td>TL-5</td>
</tr>
<tr>
<td>BR-2-98</td>
<td>Bridge Sidewalk Railing with Concrete Parapet</td>
<td>TL-4</td>
</tr>
<tr>
<td>DBR-2-73</td>
<td>Deep Beam Bridge Guardrail</td>
<td>TL-2</td>
</tr>
<tr>
<td>DBR-3-11</td>
<td>Deep Beam Bridge Retrofit Railing</td>
<td>TL-3</td>
</tr>
<tr>
<td>PCB-91</td>
<td>Portable Concrete Barrier (Fully Anchored)</td>
<td>TL-4</td>
</tr>
<tr>
<td></td>
<td>Portable Concrete Barrier (Unanchored)</td>
<td>TL-3</td>
</tr>
<tr>
<td>SBR-1-99</td>
<td>42&quot; Single Slope Parapet Type</td>
<td>TL-5</td>
</tr>
<tr>
<td>TBR-91</td>
<td>Thrie Beam Bridge Railing (Retrofit)</td>
<td>TL-4</td>
</tr>
<tr>
<td>TST-1-99</td>
<td>Twin Steel Tube Bridge Railing</td>
<td>TL-4</td>
</tr>
</tbody>
</table>

### 304.3 WHEN TO USE

#### 304.3.1 PARAPET TYPE (BR-1 & SBR-1-99)

The department currently has three (3) standard concrete parapet type bridge railing systems: a 36" New Jersey shape, a 42" New Jersey shape and a 42" single slope shape. These systems are for use on roadway and railroad overpass structures with no sidewalks and structures where the finished deck surface is 25 ft [7.62 m] or more above the ground line or water surface. Details for these parapet types, including end transitions to terminal assemblies, are provided in the Standard Bridge Drawings. The transition section may be placed on a structure’s turned back wingwalls, widened approach slab or directly on the actual structure. If the transition section is placed directly on the structure, a curb is required for the full length of the approach slab.

The 36" barrier section is for use on structures located on two (2) lane routes with an ADTT in one direction less than 2500.

The 42" barrier sections are for use on structures located on interstates, divided highways of four (4) lanes or more, and two (2) lane routes with an ADTT in one direction of 2500 or more. Final decision of which section to use rests with the districts and should be finalized during the preliminary structural design review. The single slope barrier section is unaffected by the placement of future overlays, but weighs 23% more than the 42" New Jersey type parapet.

A 50" deflector type median barrier and a 57" single slope median barrier are for use on...
structures where protection against oncoming headlight glare is required. The structure’s barrier height and type shall match the design of the adjoining roadway median barrier.

For each of the above listed barrier types, designers are required to confirm the structural adequacy of the concrete deck slab as described in the “Concrete Deck Design” Section 302.2 of this manual.

All concrete parapet type barriers shall be designed and detailed as follows:

A. All horizontal reinforcing steel shall be detailed as continuous for the total length of the structure.

B. Crack control joints shall be sawed into the concrete parapets. The distance between the saw-cut joints on the structure shall be between 6'-0" and 10'-0". The detailed locations of the crack control joints and vertical reinforcing bars shall be shown in the contract plans for all parapet types.

C. The saw-cut crack control joint shall be detailed as 1 ¼ inch deep and shall be filled with a polyurethane or polymeric material conforming to ASTM C920, Type S. The bottom one-half inch of both the inside and outside face shall be left unsealed to allow any water that enters the joint to escape. This requirement is established in the Standard Bridge Drawings; however, a plan note is required for special designs. See Section 600.

304.3.2 DEEP BEAM BRIDGE GUARDRAIL (DBR-2-73)

This railing configuration does not meet the Department’s minimum NCHRP 350 or MASH acceptance criteria (i.e. TL-3) for use on any project described in Section 304.1 of this manual. In no case, shall this railing system be used on an overpass structure or a project where the finished deck surface is greater than 25 feet above the normal water surface elevation or final ground line.

When a structure is included in a project, as defined in Section 304.1 of this manual, existing Deep Beam Bridge railing shall be upgraded in accordance with Standard Bridge Drawing, DBR-3-11.

The standard configuration for this rail type does not meet the minimum requirements specified by AASHTO for pedestrian and bicycle railings and shall not be used where pedestrian or bicycle traffic is expected. A modified railing design meeting these requirements and using the Type 1 post design may be justified.

Use of Type A anchors, as detailed on the Standard Bridge Drawing, is not recommended. The Type B alternative is recommended because they are easier to install in a deck or box beam and easier to replace if damaged in a collision.

Designers should recognize that variable post lengths may be required along the length of a
structure due to beam camber. A design data sheet is available from the Office of Structural Engineering to address these concerns.

304.3.3 TWIN STEEL TUBE BRIDGE RAILING (TST-1-99)

This railing configuration was developed as a replacement to the Deep Beam Bridge Guardrail system on projects requiring a higher NCHRP acceptance level. The Twin Steel Tube Bridge Railing is for use over rural stream crossings on two (2) lane routes with an ADTT in one direction less than 2500 where the finished deck surface is less than 25 feet above the normal water surface elevation or final ground line. This system shall not be used on an overpass structure.

The standard configuration for this rail type does not meet the minimum requirements specified by AASHTO for pedestrian and bicycle railings and shall not be used where pedestrian or bicycle traffic is expected. A modified railing design meeting these requirements may be justified.

The required bridge terminal assembly section used to transition from Type 5 or 5A approach roadway guardrail to the bridge railing is detailed on Standard Construction Drawing GR-3.6.

The typical post spacing is 6'-3". The standard drawing enables the designer to reduce the first, last and one additional post spacing per span on each side of the bridge to account for construction clearances. The designer should carefully review the position of the posts that are near the corner of a structure for possible interference with wingwalls, tie rods, etc. For box beam bridge types, post spacing dimensions shall be referenced to each box beam end.

The site plan shall show the station of the center of the first inlet-mounted post on each corner of the bridge.

304.3.4 BRIDGE RETRO-FIT RAILING, THRIE BEAM BRIDGE RAILING FOR BRIDGES WITH SAFETY CURBS (TBR-91)

Thrie beam railing, as described on Standard Bridge Drawing TBR-91, is for use as a provisional upgrade on structures with safety curb and parapets where a safety upgrade is required under Section 304.1, and the structure will be rehabilitated or replaced in the near future.

The Office of Structural Engineering does not generally recommend this alternative because of the potential for high maintenance costs. A more suitable alternative is concrete refacing of existing safety curb and parapets to either a New Jersey or Single Slope shape. See Section 400 of this Manual for additional information on refacing of safety curb and parapets.

304.3.5 PORTABLE CONCRETE BARRIER (PCB-91)

This system is for use on construction projects to protect project personnel and to provide a
temporary barrier system when a permanent bridge railing system does not exist. Application guidelines for PCB-91 are provided in Design Data Sheet, PCB-DD, available at the Office of Structural Engineering website.

The designer is required to detail the installation requirements, including the number of anchor bolts per barrier, in the bridge plans. The pay item for this barrier system is Item 622 - Portable Concrete Barrier, 32 inch, Bridge Mounted. Although temporary railing is to be specified and completely described in the bridge plans, temporary railing is a roadway item and shall be included in the roadway quantities.

On projects where maintaining minimum lane widths during a construction phase is not possible due to limited bridge width, the use of a top mounted steel post and tubular steel rail system, similar to the Twin Steel Tube bridge guardrail, may be justified. The railing, post and anchorage designs of these systems are to be in accordance with the AASHTO LRFD Bridge Design Specifications, 2nd Edition, Sections A13.1-3.

304.3.6 BRIDGE SIDEWALK RAILING WITH CONCRETE PARAPETS (BR-2-98)

This railing system is for use on bridges with sidewalks at least 5'-0" wide and a curb height of 8 inches. Although this system is essentially a combination railing system, it may also be used without a sidewalk in applications where pedestrian traffic is not a concern.

Where Vandal Protection Fencing is required, the fencing shall be installed behind the steel tubing as shown in Figure 327. However, the steel tubing may be omitted if the concrete parapet height is 32" or greater. See Figure 326. If the tubing is omitted, the fencing should extend the full length of the concrete parapet and the additional 18" parapet height at each end, as detailed in the standard, is not required.

The concrete parapet shall be designed and detailed as follows:

A. All horizontal reinforcing steel shall be detailed as continuous for the total length of the structure.

B. Crack control joints shall be sawed into the concrete parapets. The distance between the saw-cut joints on the structure shall be between 6'-0" and 10'-0". The detailed locations of the crack control joints and vertical reinforcing bars shall be shown in the contract plans.

C. The saw-cut crack control joint shall be detailed as 1 ¼ inch deep and shall be filled with a polyurethane or polymeric material conforming to ASTM C920, Type S. The bottom one-half inch of both the inside and outside face shall be left unsealed to allow any water that enters the joint to escape. This requirement is established in the Standard Bridge Drawing; however, a plan note is required for special designs. See Section 600.
304.3.7 DEEP BEAM BRIDGE RETROFIT RAILING (DBR-3-11)

This system has been accepted by FHWA as a TL-3 compliant railing. The DBR-3-11 railing system is intended solely as an upgrade for existing Deep Beam Bridge Guardrail (DBR-2-73) installations and shall be specified on all bridge projects with existing DBR-2-73 railing that require at least TL-3 acceptance level according to BDM Section 304.1. The use of the AASHTO “Guide Specifications for Bridge Railing” will not be permitted.
305 FENCING

305.1 GENERAL

The primary purposes of protective fencing are to provide for the security of pedestrians and to discourage the throwing or dropping of objects from bridges onto lower roadways, railways, boat lanes or occupied property. In addition, fence may be needed on high level bridges where wind may threaten to blow pedestrians or occasional stranded motorists off the bridge and on bridges where there is a danger that the outside parapet may be mistaken for a median barrier, causing persons to jump over the parapet in emergency situations in periods of darkness. These situations should be treated on a case-by-case basis.

Since a falling object problem could occur at any bridge accessible to pedestrians, it is necessary to consider installation of protective fencing at such locations.

Generally, fencing attached to bridge structures for the protection of traffic and pedestrians should conform to the Vandal Protection Fencing Standard Bridge Drawing. The designer may need to enhance this standard to deal with requirements for the specific structure.

305.2 WHEN TO USE

Designers shall investigate the need for fencing during the Red Flag Summary when a bridge is included in the one of the following construction project types: new construction, complete deck replacements, replacement of deteriorated concrete deck edges, superstructure widenings and rigid concrete overlays. Pedestrian Fencing may be required when a total of 10 points or greater is achieved for a structure according to the following criteria. The designer should use this procedure as a general guide as to the need for fencing. The affected district should also be consulted for their input. The list is not to be construed as all-inclusive. Other rationale may be used on a case-by-case basis. Similarly, retrofitting of bridges that qualify according to the total index number is not mandatory if adequate justification for not doing so can be documented.
A. Overpass within an urbanized area of 50,000 or more population  
B. Overpass with sidewalks but not in an urbanized area as defined in (A)  
   (“Sidewalk” does not include safety curbs 2'-3" [685 mm] or less in width)  
C. Overpass which is unlighted  
D. Overpass not a main thoroughfare, i.e., on collectors or local streets  
E. Overpass within ½ mile [0.8 km] of another overpass exclusive of pedestrian  
   bridges, having or requiring protection  
F. Overpass within ½ mile [0.8 km] of another overpass having previous reports  
   of falling objects  
G. Overpass within 1 mile [1.6 km] of a school, playground or other pedestrian  
   attraction  
H. Bridges over any feature which has a high count of boat, rail, vehicular or  
   pedestrian traffic, or includes damage-sensitive property  
I. Overpass which has had prior reported incident of falling objects  
J. Overpass which is used exclusively by pedestrians

“OVERPASS” is a bridge over a highway or a railroad.

Justification Items (E), (F) and (G) do not apply to overpasses carrying Freeway routes as defined in ORC 4511.01 where pedestrians are prohibited per ORC 4511.051.

305.3 FENCING CONFIGURATIONS

For structures with sidewalks, the top of fence should be a minimum height of 8 feet [2450 mm] above the sidewalk. For a greater degree of protection against objects being thrown from the bridge, the fence may be curved to overhang the sidewalk. For curved fence the posts should be vertical for approximately 8 feet [2450 mm] above the sidewalk before curving inward over the sidewalk. The overhang should be at least 1 foot [300 mm] less than the width of the sidewalk, with a maximum overhang of 3'-7" [1100 mm]. The slope of the straight overhanging portion should be 1 vertical to 4 horizontal. The radius of the connecting arc should be 32 inches [815 mm]. See Figures 326 & 327.

For narrow pedestrian bridges, bent pipe frames are generally used with pipe bend radii of 24" [600 mm] at the upper corners and the start of the radii about 8 feet [2450 mm] above the sidewalk surface. The fabric should start at the deck line, top of curb or parapet and may stop at the upper end of the bent portion of the frame.

Fabric on the top horizontal area of the frame is sometimes not installed because adventurous youngsters tend to walk on the top of the enclosure. See Figure 328 for an illustration of this configuration. To try to eliminate the adventurous youngster problem, some pedestrian bridges have used a frame design that comes to a peak at the center of the structure, similar to a house.
roofline.

Chain link fabric should not have an opening at the bottom through which large objects could be pushed. A detail to close the bottom of a fencing section is included on the standard bridge drawing. The closure plate detail is required for all fence configurations that have tension wire at the bottom of the fence fabric.

Posts and frames may be either plumb or perpendicular to the longitudinal grade of the bridge, subject to considerations of aesthetics or practicality of construction. Complete details of base plates, pipe inserts or other types of base anchorage shall be provided on the plans. If applicable to the specific project, details from the standard bridge drawing may be referred to in the project plans.

### 305.4 SPECIAL DESIGNS

The following information is given the designer as a basis for specialized designs. It is not intended for designers to develop their own requirements in lieu of the standard bridge drawing.

For fence installation projects on new structures, the installation of a traffic railing (steel tubing) is not required if the top concrete parapet or concrete wall is 32" [813 mm] above roadway for structures without sidewalks or 32" [813 mm] above the top of sidewalk for structures with sidewalks. See Figure 326.

For special fence designs, plan notes shall be required to define materials, traffic maintenance, construction procedures and other requirements. The designer should follow the example of standard bridge drawing for development of required notes.

### 305.5 FENCE DESIGN GENERAL REQUIREMENTS

Fencing mesh should consist of either of the following materials:

A. Chain-link wire mesh with one inch [25 mm] diamonds. The core wire shall be 11 gage [3.05 mm] with a Polyvinyl chloride coating. (C&MS 710.03)

B. Welded wire fabric with $\frac{1}{2}" \times 3"$ [12 mm x 75 mm] opening size. The core wire shall be 10.5 gage [3.25 mm]; galvanized after welding (1.2 oz zinc/ft²), and PVC coated (10 mil [0.25 mm]).

Brace and bottom rails shall be clamped to posts or post frames.

The top rail, if any, of a free-standing fence should be continuous over two or more posts and suitable cap fittings provided.

Bent pipe frames for narrow pedestrian bridges are permitted. Bent pipe frames for narrow
pedestrian bridges should be fabricated in two or more sections and field spliced at the top with sleeves bolted to the frame sections.

To prevent pipe blow-ups during galvanizing, both ends of pipe should be open. Therefore base plates should have holes in them almost equal to the pipes’ inside diameter.
305.5.1 WIND LOADS

The design wind pressure (P) in lb/ft² [kPa] shall be calculated using:

\[ P = 27.69C_h, \text{ derived from the formula: } \]
\[ P = 0.00256(1.3V)^2C_sC_hC_i \quad (1) \]

Where:
\[ V = 50 \text{ yr. mean wind vel.} \quad (2) = 80 \text{ mph} \]
\[ 1.3 = 30\% \text{ Wind Gust Factor} \]
\[ C_s = \text{Shape Coefficient} = 1.0 \]
\[ C_h = \text{Height Coefficient} \text{ (See table)} \]
\[ C_i = \text{Ice Coefficient} = 1.0 \]

\[ P = 1.326C_h, \text{ derived from the formula: } \]
\[ P = 0.0471(1.3V)^2C_sC_hC_i/1000 \quad (1) \]

Where:
\[ V = 50 \text{ yr. mean wind vel.} \quad (2) = 129 \text{ km/h} \]
\[ 1.3 = 30\% \text{ Wind Gust Factor} \]
\[ C_s = \text{Shape Coefficient} = 1.0 \]
\[ C_h = \text{Height Coefficient} \text{ (See table)} \]
\[ C_i = \text{Ice Coefficient} = 1.0 \]

<table>
<thead>
<tr>
<th>( C_h )</th>
<th>Height (ft)</th>
<th>Height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>0 - 15</td>
<td>0 - 4500</td>
</tr>
<tr>
<td>1.0</td>
<td>15 - 30</td>
<td>4500 - 9000</td>
</tr>
<tr>
<td>1.1</td>
<td>30 - 50</td>
<td>9000 - 15 000</td>
</tr>
<tr>
<td>1.25</td>
<td>50 - 100</td>
<td>15 000 - 30 000</td>
</tr>
<tr>
<td>1.40</td>
<td>100 - 150</td>
<td>30 000 - 46 000</td>
</tr>
<tr>
<td>1.50</td>
<td>150 - 200</td>
<td>46 000 - 61 000</td>
</tr>
</tbody>
</table>

The centroid of the horizontally projected area of the fence is to be used to determine the height above normal terrain and the value of \( C_h \).

The projected area for wind forces on 11 gage [3.05 mm] polyvinyl chloride coated one inch [25 mm] wire mesh shall be 20% of the gross horizontally projected area.

Additional area for posts, rails and other hardware need not be considered.

Ref. (1) Specifications for the Design and Construction of Structural Supports for Highway Signs, AASHTO.
Ref. (2) Isotach's of the U.S. The 80 mph [129 km/h] line covers the northwestern portion of Ohio and shall be used herein for all of Ohio.

306 EXPANSION DEVICES

306.1 GENERAL

Expansion devices should provide a total seal against penetration and moisture. Standard bridge drawings are available for expansion devices for typical bridge superstructure types.

Expansion devices as shown in the standard bridge drawings and their support systems are designed for an HS25 [MS22.5] loading with 100% impact. Special expansion devices including finger joints and modular joints and their support systems shall also be designed for an HS25 [MS22.5] loading with 100% impact.

For fabricated steel expansion devices, the designer should specify the type of steel required. Type of steel should be included as a plan note if requirements in the plans are not covered by a selected standard bridge drawing.

To protect steel expansion devices, metallizing of the exposed surfaces with a 100% zinc coating shall be specified. Standard bridge drawings define the requirements for metallizing. The design agency will need to develop plan notes for special expansion devices, such as finger joints and modular joints. Use the note for shop-applied metallizing located in the appendix as a guideline. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans.

306.1.1 PAY ITEM

Expansion devices, except as specifically listed in this section, shall be paid for as Item 516.

For sealed expansion devices the elastomeric seal, either strip or compression, shall be included in the pay Item 516.

The plans shall clearly show what components are included with the expansion devices, Item 516. As an example, cross frames, which are field welded to both the superstructure girders and the expansion devices, are part of the 513 structural steel item. The seal is considered part of the expansion device and should be included in the 516 pay item.

306.1.2 EXPANSION DEVICES WITH SIDEWALKS

On structures with sidewalks, the expansion devices shall be the same type as furnished for main bridge deck expansion joint.
Sidewalk details for standard expansion devices (strip seals) are shown on the standards. For non-standard devices, a curb plate and sidewalk cover plate will be required. The Curb and sidewalk plates should be separated at the interface of the sidewalk and curb. See details on Standard Bridge Drawings: EXJ-2-81, EXJ-3-82, EXJ-4-87, EXJ-5-93 and EXJ-6-95 for sidewalk plates.

### 306.1.3 EXPANSION DEVICES WITH STAGE CONSTRUCTION

On projects involving stage construction, joints in the seal armor must be located and shown in the plans. At the stage construction lines, expansion devices should require complete penetration welded butt joints. If butt welds will be in contact with a sealing gland the butt-welded joint shall be ground flush at the contact area.

### 306.2 EXPANSION DEVICE TYPES

#### 306.2.1 ABUTMENT JOINTS IN BITUMINOUS CONCRETE, BOX BEAM BRIDGES

This poured joint seal system is capable of small expansion movements, up to 3/16" [5 mm]. A plan insert sheet, Abutment Joints in Bituminous Concrete Box Beam Bridges, is available through the Office of Structural Engineering’s web page. This device requires three bid items: Item Special - Sawing and Sealing Bituminous Concrete Joints; Item 516 - Joint Sealer, As Per Plan; and Item 516 - 1" Preformed Expansion Joint Filler.

#### 306.2.2 ABUTMENT JOINTS AS PER AS-1-81

A group of no or small movement joints used for sealing and rotational purposes are detailed on Standard Bridge Drawing, AS-1-81.

#### 306.2.3 EXPANSION JOINTS USING POLYMER MODIFIED ASPHALT BINDER

This device is generally for use on structures with concrete or asphalt overlays and where expected expansion is 0 to 1½" [40 mm]. A detail & plan note insert sheet, Polymer Modified Asphalt Expansion Joint System, is available through the Office of Structural Engineering’s web page. This item is bid as a special.

Thickness of the polymer-modified joint shall be a minimum of 2" [50 mm]. A thickness greater than 5" [125mm] should be avoided.
306.2.4  STRIP SEAL EXPANSION DEVICES

The seal size is limited to a 5" [125 mm] maximum. Unpainted A588[M] weathering steel should not be used in the manufacture of this type expansion device as A588[M] does not perform well in the atmospheric conditions an expansion device is subjected to. Standard Bridge Drawings, EXJ-4-87, EXJ-5-93 and EXJ-6-95, are available. The designer must ensure that all details are covered in the plans because the standard drawing is not inclusive for all structure types.

The strip seal shall be of one piece across the total width of the structure. No splices will be acceptable.

306.2.5  COMPRESSION SEAL EXPANSION DEVICES

Maximum allowable seal size is 4" [100 mm]. A 5" [125 mm] wide seal shall not be used since installation problems have been encountered. Compression seal expansion devices are limited to structures with a maximum skew of 15 degrees. Movement should be limited so that the seal is not compressed greater than 60 percent or less than 20 percent.

The compression seal shall be of one piece across the total width of the structure. No splices will be acceptable. Standard Bridge Drawings EXJ-2-81 & EXJ-3-82 give generally used details.

306.2.6  STEEL SLIDING PLATE ENDDAMS, RETIRED STANDARD DRAWING SD-1-69

In general steel sliding plate enddams are not recommended for new structures. This expansion device is limited to total movement of 4" [100 mm], including movement in both directions.

Sliding plates should be configured to prevent binding and bearing when the superstructure is supported on elastomeric bearings.

Unpainted A588[M]/A709[M] Grade 50W materials are not recommended for construction of this type of joint.

306.2.7  MODULAR EXPANSION DEVICES

Modular expansion devices may be required for structures when total required movements exceed movement capacity of a strip or compression seal. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans.

Modular devices main load bearing beams, support beams and welds shall be designed for fatigue.
The manufacturer of the expansion device shall be required by plan note to submit design calculations showing that the device can meet the impact and fatigue design requirements.

Modular devices have been known to fail at connections due to welding and fatigue. Therefore it is recommended the following general requirements be included in any project plan notes:

A. Spacing of support beams shall be limited to 3'-0" [1000 mm] centers under main load bearing beams unless fatigue testing of the actual welding connection details has been performed to show that a greater spacing is acceptable. The fatigue cycles should be 2,000,000 + truck load cycles or truck traffic count over the expected life of the structure.

B. Shop or field welds splicing main beams, or connections to the main beams shall be full penetration welded and 100 percent non-destructively tested in accordance with AWS D1.5 Bridge Welding Code. Any required field splices or joints and non-destructive testing shall be located and defined in the plans.

C. Fabricators of modular devices shall be pre-qualified 513 Level UF fabricators. Review Section 302.4.1.3 and contact the Office of Structural Engineering for recommendations.

D. Approved manufacturer/fabricator shall supply a qualified technical representative to the jobsite during all installation procedures.

E. Seals shall be one continuous piece through the total length of the structure.

Design of support for the modular device and deck thickness should allow for multiple styles or designs of modular devices. Contact suppliers and become familiar with the modular devices available.

Contact the Office of Structural Engineering for sample notes used on other projects.

306.2.8 TOOTH TYPE, FINGER TYPE OR NON-STANDARD SLIDING PLATE EXPANSION DEVICES

Finger or sliding plate joints are another alternative type of expansion device where movements exceed the capacity of either strip or compression seal devices. This type of expansion device generally competes against Modular joints. Their advantage is their simplicity of design. Their disadvantage is their inability to seal against intrusion of water and debris. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans. Example plan notes are provided in the appendix.

Use of a tooth type expansion device also requires neoprene drainage troughs and a suitable drainage system to carry away the water. Both the neoprene trough and downspout to drainage trough connection must be detailed completely. Special attention should be paid to developing a complete seal at the downspout to trough connection.
Vulcanization is recommended over adhesive for sealing.

Finger devices shall be designed for fatigue and conform to fracture critical requirements if the design has fracture critical components in it.

Fabricators of finger or sliding plate devices shall be pre-qualified 513 Level UF fabricators. Review Section 302.4.1.3 and contact the Office of Structural Engineering for recommendations.

### 306.3 EXPANSION DEVICE USES – BRIDGE OR ABUTMENT TYPE

#### 306.3.1 INTEGRAL OR SEMI-INTEGRAL TYPE ABUTMENTS

No allowance for temperature need be made.

The vertical joint between abutment backwall and approach slab should be finished as per Standard Bridge Drawing AS-1-81, Detail B.

#### 306.3.2 REINFORCED CONCRETE SLAB BRIDGES

The following table specifies joint requirements. Expansion length is defined as the total length if no fixed bearing exists, or length from fixed bearing to proposed expansion device location, if one exists.

<table>
<thead>
<tr>
<th>Expansion Length</th>
<th>Joint Required</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft)</td>
<td>(m)</td>
<td></td>
</tr>
<tr>
<td>0 - 40</td>
<td>0 - 12</td>
<td>None</td>
</tr>
<tr>
<td>40 - 200</td>
<td>12 - 60</td>
<td>None (1) PM (2)</td>
</tr>
<tr>
<td>200 +</td>
<td>60 +</td>
<td>PM</td>
</tr>
</tbody>
</table>

|   (1) = flexible abutments and piers (CPP-2-94 and CPA-5-94) |
|   (2) = abutments and/or piers fixed or rigid |
|   PM = Polymer Modified Asphalt Joint |

#### 306.3.3 STEEL STRINGER BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.
### Expansion Length (ft) (m) | Joint Required | Approach Slab Joint
---|---|---
0 - 30 | 0 - 10 | None | AS-1-81 detail B
30 - 125 | 10 - 38 | PM (1) or EXJ-4-87 | AS-1-81 detail B
125 - 400 | 38 - 125 | EXJ-4-87 | AS-1-81
400 + | 125 + | TTED or MED |

PM = Polymer Modified Asphalt Joint  
TTED = Tooth Type expansion device  
MED = Modular Expansion Device  
(1) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems.

### 306.3.4 PRESTRESSED CONCRETE I-BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.

<table>
<thead>
<tr>
<th>Expansion Length (ft) (m)</th>
<th>Joint Required</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 40</td>
<td>0 - 12</td>
<td>None</td>
</tr>
<tr>
<td>40 - 225</td>
<td>12 - 65</td>
<td>PM (1) or EXJ-6-95</td>
</tr>
<tr>
<td>225 - 500</td>
<td>65 - 150</td>
<td>EXJ-6-95</td>
</tr>
<tr>
<td>500 +</td>
<td>150 +</td>
<td>TTED or MED</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint  
TTED = Tooth Type expansion device  
MED = Modular Expansion Device  
(1) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems

### 306.3.5 NON-COMPOSITE PRESTRESSED BOX BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.
### Expansion Length

<table>
<thead>
<tr>
<th>Expansion Length (ft)</th>
<th>Joint Required (2)</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 40</td>
<td>PM</td>
<td></td>
</tr>
<tr>
<td>40 - 225</td>
<td>PM (1) or EXJ-5-93</td>
<td>AS-1-81 detail A,C,E</td>
</tr>
<tr>
<td>225 - 500</td>
<td>EXJ-5-93</td>
<td>AS-1-81 detail C</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint

(1) = Bridges with sidewalks should not use polymer modified expansion joint systems

(2) = Joint requirements are for rigid or fixed abutments. For flexible abutments requiring no expansion movement a PM joint is recommended except for (1).

#### 306.3.6 COMPOSITE PRESTRESSED CONCRETE BOX BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.

<table>
<thead>
<tr>
<th>Expansion Length (ft)</th>
<th>Joint Required (2)</th>
<th>Approach Slab Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 40</td>
<td>PM</td>
<td></td>
</tr>
<tr>
<td>40 - 225</td>
<td>PM (1) or EXJ-5-93</td>
<td>AS-1-81 detail C,D,F</td>
</tr>
<tr>
<td>225 - 500</td>
<td>EXJ-5-93</td>
<td>AS-1-81 detail C</td>
</tr>
</tbody>
</table>

PM = Polymer Modified Asphalt Joint

(1) = Bridges with sidewalks should not use polymer modified expansion joint systems

(2) = Joint requirements are for rigid or fixed abutments. For flexible abutments requiring no expansion movement a PM joint is recommended except for (1).

#### 306.3.7 ALL TIMBER STRUCTURES

No allowance for temperature need be made.
307  BEARINGS

307.1  GENERAL

The Department’s policy is, whenever possible, use laminated elastomeric bearings.

Justification, including design calculations showing elastomeric bearings will not be adequate for the structure, must be available.

When specialized bearings, such as pot, disc or spherical, are required, detail notes shall be included in the contract plans. A plan note for pot bearings is provided in the appendix and may require modification by the designer based on the specific structure. If a cost evaluation shows that either spherical or disc bearings could be competitive against pot bearings, those bearings should be included in the plans and special notes developed.

For specialized bearings, the designer's detail plan notes shall require the contractor to coordinate the required substructure bearing seat elevations or dimensions with the selected bearing manufacturer. A note is available in Section 700.

307.2  BEARING TYPES

307.2.1  ELASTOMERIC BEARINGS

The preferred design of elastomeric bearings is AASHTO’s Method A. Non-laminated elastomeric bearings are only acceptable if actual design calculations support their use.

AASHTO design Method B is recommended for use when specialized bearings are being considered. Method B designs do require additional material and bearing testing over Method A designs. The designer shall investigate this additional cost versus the cost savings when compared to the use of specialized bearings.

Elastomeric bearings should be designed based on a selected durometer of either 50 or 60.

Elastomeric bearings should generally be limited to a 5 inch [125 mm] maximum elastomeric height excluding internal laminates with a minimum total height of one inch [25 mm]. The designer should evaluate greater height elastomeric bearings, or elastomeric bearings with sliding surfaces, before arbitrarily selecting specialized, high priced pot spherical or Disc type bearings.

Elastomeric bearings for steel beam and girder bridges will require a load plate. Field welding of a beam or girder to the bearing load plate should be controlled so that the temperature of the elastomer is subjected to does not exceed 300°F [150°C].

Elastomeric bearings with a load plate shall have the plate beveled if the rotation and or grade exceed the limitations of AASHTO Section 14. The load plate thickness required by design shall be the minimum thickness of the beveled plate. A nominal minimum thickness of 1½ inches [38

3-99
mm] is recommended but not mandatory.

Elastomeric bearings should not bear on unbonded steel surfaces. Therefore all steel plates in contact with an elastomeric bearing shall be vulcanized (bonded) to the bearing.

Vertical deformations of the bearings greater than 1/8" [3 mm] are to be compensated for in the elevations of the bridge bearing seats. A note shall be required in the design plans.

Detail plans shall include the unfactored dead load, live load and total load reactions for each elastomeric bearing design.

307.2.2 STEEL ROCKER & BOLSTER BEARINGS, RB-1-55

This bearing type should only be used in rehabilitation projects where a match to the existing bearing is required.

This bearing type is presented on Standard Bridge Drawing RB-1-55. The standard drawing also includes material and maximum load capacity requirements for this bearing type.

This bearing is limited to a 2" [50 mm] movement in one direction from the vertical.

The assumed rolling and sliding resistance of rockers is 0.25 DL times r/R, where: DL is the dead load reaction on the rockers, “r” is the radius of the pin, in inches [mm], and “R” is the radius of the rocker, in inches [mm].

For structures where the grade at the bearing is greater than 2 percent, the upper load plate shall require beveling to match the required grade. The designer shall provide a plan detail of the beveled, upper load plate. The thickness of the upper load plate at the centerline of the bearing (dimension C in the standard drawing) should be held.

Pier and abutment seats should allow the bearing base plate to achieve full seat area. The designer may choose, on structures of extreme skew, the alternative of clipping the corner of the bearing plate to save adding additional width to the pier and abutment seats. The maximum clip shall not remove more than 3 in² [1900 mm²] of bearing base plate surface area. The designer shall investigate that the substructure can accept the increased loading.

307.2.3 SLIDING BRONZE TYPE & FIXED TYPE STEEL BEARINGS

This bearing type should only be used in rehabilitation projects where a match to the existing bearing is required. The sliding bronze type expansion bearing is known to freeze up, therefore, not providing the required freedom of movement. This bearing type is normally not recommended even on rehabilitation projects.
This bearing type is found on older steel beam or girder structures and is shown on Standard Bridge Drawing FSB-1-62. This standard is not currently active but copies are available through the Department. The fixed type bearing shown on Standard Bridge Drawing FB-1-82, originated from the old Standard Bridge Drawing FSB-1-62.

In the design of these bearings for steel bridges the assumed coefficient of friction of lubricated bronze sliding bearings is 0.10.

307.2.4 SPECIALIZED BEARINGS

307.2.4.1 POT TYPE BEARINGS

Pot type bearings are capable of sustaining high vertical loads and multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, pot bearings are capable of large expansion movements. A plan note is provided in the appendix. The Designer may modify the note depending upon the specific structure.

AASHTO has both a design and construction section for pot bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a pot bearing.

If requested, the Designer shall provide the Department justification for the use of pot bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.

Pot bearings shall not be used with other bearing types.

Pot bearings are not considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

The minimum vertical load on a pot bearing shall not be less than 20% of total vertical design load.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The plan notes available in the Appendix include requirements for the sliding surfaces and materials. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).
307.2.4.2 DISC TYPE BEARINGS

Disc type bearings are capable of sustaining high vertical loads and multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, disc bearings are capable of large expansion movements.

AASHTO has both a design and construction section for disc bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a disc bearing. Generic plan notes for disc bearings are not available through the Office of Structural Engineering. The designer will need to develop specific notes based on the specific structure. Consult the Office of Structural Engineering for previously used notes.

If requested, the Designer shall provide the Department justification for the use of disc bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.

Disc bearings shall not be used with other bearing types.

Disc bearings are no longer considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).

307.2.4.3 SPHERICAL TYPE BEARINGS

Spherical type bearings are capable of sustaining high vertical loads and large multi-directional rotations. Included with PTFE (Teflon) sliding surfaces, spherical bearings are capable of large expansion movements.

AASHTO has both a design and construction section for spherical bearings. The Designer should use these sections and this Manual as a guide in designing, selecting and specifying a spherical bearing. Generic plan notes for spherical bearings are not available through the Office of Structural Engineering. The designer will need to develop specific notes based on the specific structure. Consult the Office of Structural Engineering for previously used notes.
If requested, the Designer shall provide the Department justification for the use of spherical bearings. This justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure. Cost comparisons to other specialized bearing types shall also be included.

Spherical bearings shall not be used with other bearing types.

Spherical bearings are not considered proprietary and, therefore, alternate bearing designs are not required.

Design plans shall show design requirements for both vertical and horizontal loads, required movements, required rotations and maximum friction factor for the sliding surfaces.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 1/4 inch [6 mm].

To accommodate the required horizontal movements, this type of bearing utilizes PTFE (Teflon) to stainless steel sliding surfaces. The Designer should be aware that Teflon to stainless steel friction factors vary with the applied load (i.e. the lower the load, the higher the friction factor).

307.3 GUIDELINES FOR USE

307.3.1 FIXED BEARINGS

307.3.1.1 FIXED TYPE STEEL BEARINGS (RB-1-55 OR FB-1-82)

These types of fixed bearings have been used in the past for steel beam or girder bridges.

Fixed bearings, Standard Bridge Drawing FB-1-82, have also been used in conjunction with laminated elastomeric bearings acting as expansion bearings. This is especially true in rehabilitation work where this existing fixed bearing type could possibly be salvaged.

Generally, steel fixed bearings should be limited to steel beam and girder bridge structures with a maximum 15° skew and 60 foot [20 meter] deck width.

Bolster-type fixed bearings (Standard Bridge Drawing RB-1-55) are not recommended for selection on new structures, replacement structures or total superstructure rehabilitation. They may be chosen on widening projects to match existing bearings.

307.3.1.2 FIXED LAMINATED ELASTOMERIC BEARINGS FOR STEEL BEAM BRIDGES

Fixed laminated elastomeric bearings are recommended for use on new steel structures, replacement steel structures or total superstructure rehabilitation.
Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Laminated elastomeric bearings will require analysis by the designer to fit the specific structure.

Laminated elastomeric bearings for steel beam and girder bridges shall be designed with a load plate.

For additional information, see Section 307.2.1 on elastomeric bearings.

### 307.3.1.3 FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS

Laminated elastomeric bearings shall be used for prestressed concrete box beam bridges.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

A fixed bearing condition may be assumed to be obtained by the use of 1 inch [25 mm] thick laminated elastomeric bearing pads and the installation of anchor dowels with grout.

For additional information see Section 307.2.1 on elastomeric bearings.

### 307.3.1.4 FIXED LAMINATED ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS

Laminated elastomeric bearings shall be used for prestressed concrete I-beam bridges.

The Department has no standards for fixed or expansion bearings for prestressed I-beam superstructures; therefore, the designer is required to design the bearing.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

The designer should note that prestressed I-beam bridges, whether single or multiple continuous spans, will generally follow the same constraints as prestressed box beam bridges.

In designing the bearing for an I-beam bridges the designer should verify that the attachment of the bearing to the I-beam or any lateral restraining devices for the bearing do not interfere with placement of the diaphragm. Interference of the bearing with the diaphragms may cause spalling of the diaphragm and future maintenance problems.

For additional information, see Section 307.2.1 on elastomeric bearings.
307.3.2 EXPANSION BEARINGS

307.3.2.1 ROCKER BEARINGS (RB-1-55)

This type expansion bearing (rocker) in the past has been used on steel beam or girder bridges. Generally, this type steel expansion bearing is limited in use to steel beam and girder bridge structures with a maximum 15° skew, 60 foot [20 meter] deck width.

Rocker type expansion bearings are not recommended for selection on new structures, replacement structures or total superstructure rehabilitations. They may be chosen on widening projects to match existing bearings.

Twin structures, being rehabilitated, which have RB-1-55 type bearings, should not be tied together if overall finished width is to be greater than 60 feet [20 meters].

307.3.2.2 BRONZE TYPE STEEL EXPANSION BEARINGS

This sliding type bearing was used in the past on some steel beam or girder structures. Based on deleted Standard Bridge Drawing FSB-1-62 and normally used with FB-1-82 type fixed bearing.

This bearing is not recommended for use on new projects but may be required due to a special widening project requiring a match of existing bearings. This bearing type has shown problems with freezing. If jacking is being performed on a structure the designer should consider replacing this existing type of bearing with elastomeric bearings.

Twin structures, being rehabilitated, which have FSB-1-62 type bearings, should not be tied together if the total combined deck width exceeds 60 feet [20 meters]. This bearing is not designed to accept transverse movement.

307.3.2.3 EXPANSION ELASTOMERIC BEARINGS FOR BEAM AND GIRDER BRIDGES

This bearing type is recommended for use on new structures, replacement structures or total superstructure rehabilitation.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Laminated elastomeric bearings will require analysis by the designer to fit the specific structure.

Laminated elastomeric bearings for steel beam and girder bridges shall be designed with a load plate.

When the decks of twin structures are being tied together, resulting in a total structure width in excess of 60 feet [20 meters] laminated elastomeric bearings shall be required.
307.3.2.4 EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED BOX BEAMS

Box beam bridges shall have two elastomeric bearing pads at each end of each beam. At least 1 inch [25 mm] minimum thickness is required but the bearing shall be designed for the required movement and rotation.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

On skewed bridges, a 1/8" [3 mm] thick preformed bearing shim material, CMS 711.21, the same plan dimensions as the bearing, should be provided to accommodate any non-parallelism between bottom of beam and bridge seat. This non-parallelism between bottom of beam and bridge seat can result from camber and beam warpage due to skew and fabrication. Generally, half as many preformed bearing pads should be specified as the number of bearings. The preformed bearing pads should be incorporated in an item 516 in the Estimated Quantities.

307.3.2.5 EXPANSION ELASTOMERIC BEARINGS FOR PRESTRESSED I-BEAMS

Unless special limitations exist, elastomeric bearings should be selected to handle load, expansion and rotation requirements for prestressed concrete I-beam bridges.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

In designing the bearing for an I-beam bridges the designer should verify that the attachment of the bearing to the I-beam or any lateral restraining devices for the bearing do not interfere with placement of the diaphragm. Interference of the bearing with the diaphragms may cause spalling of the diaphragm and future maintenance problems.

307.3.3 SPECIALIZED BEARINGS

Where specialized bearings, such as pot, disc or spherical, are required, plan notes are needed. A plan note for pot bearings is provided in the appendix. For spherical or disc bearings, plan notes will need to be developed. If a cost evaluation shows that either spherical or disc bearings could be competitive against pot bearings, those bearings should be included in the plans and special notes developed.

For specialized bearings, the designer’s detail plan notes must require the contractor to coordinate required substructure bearing seat elevations or dimensions with the selected bearing manufacturer. A note is available in Section 700.
307.3.3.1 POT BEARINGS

See Section 307.2.4.1 for specific requirements on Pot Bearings.

307.3.3.2 DISC TYPE BEARINGS

See Section 307.2.4.2 for specific requirements on Disc Bearings.

307.3.3.3 SPHERICAL BEARINGS

See Section 307.2.4.3 for specific requirements on Spherical Bearings.
HS25 TRUCK

CONCENTRATED LOAD - 22,500 LBS. FOR MOMENT
- 32,500 LBS. FOR SHEAR

UNIFORM LOAD 800 LBS. PER LINEAR FOOT OF LOAD LANE

HS25 LANE LOADING
CLEARANCE AND
LOAD LANE WIDTH

Figure 301
THE BAR SIZE NUMBER IS SPECIFIED ON THE PLANKS IN THE BAR MARK COLUMN. THE FIRST DIGIT WHERE THREE DIGITS ARE USED, AND THE FIRST TWO DIGITS WHERE FOUR ARE USED, INDICATES THE BAR SIZE NUMBER. FOR EXAMPLE, P601 IS A NO. 6 BAR, BAR DIMENSIONS SHOWN ARE OUT TO OUT UNLESS OTHERWISE INDICATED. R INDICATES INSIDE RADIUS. UNLESS OTHERWISE NOTED, "STD." WRITTEN IN PLACE OF A DIMENSION INDICATES A STANDARD BEND AT THE END OF THE BAR.

ALL REINFORCING STEEL TO BE EPOXY COATED

Figure 302
# Tension Splices (In.)

<table>
<thead>
<tr>
<th>BAR LOCATION</th>
<th>EPOXY</th>
<th>NON-EPOXY</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLR. SIZE</td>
<td>TOP</td>
<td>OTHER</td>
</tr>
<tr>
<td>1</td>
<td>35</td>
<td>31</td>
</tr>
<tr>
<td>5</td>
<td>43</td>
<td>41</td>
</tr>
<tr>
<td>6</td>
<td>52</td>
<td>49</td>
</tr>
<tr>
<td>7</td>
<td>66</td>
<td>62</td>
</tr>
<tr>
<td>8</td>
<td>87</td>
<td>76</td>
</tr>
<tr>
<td>9</td>
<td>110</td>
<td>97</td>
</tr>
<tr>
<td>10</td>
<td>139</td>
<td>123</td>
</tr>
<tr>
<td>11</td>
<td>171</td>
<td>151</td>
</tr>
</tbody>
</table>

**Notes:**

1. For epoxy bars with cover less than \(*3d_b\) or clear spacing between bars less than \(*6d_b\) (8.25.2.3)

2. Top bars refers to only top row of reinforcement.

3. For bars spaced laterally at least 6 inches on center with at least 3 inches clear cover measured in the direction of the spacing, reduce value by 20% \((x0.80)\) (8.25.3.1), but not less than 12" per 8.32.3.1.

4. Values shown are for class "C" lap with \(f_c = 4,000\) P.S.I. and \(f_y = 60,000\) P.S.I. (8.32.3.2)

* Bar diameter

Figure 303
### Development Length for Uncoated Standard Hooks in Tension

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>MOD. FACTOR</th>
<th>BASIC LENGTH, L_{bd}</th>
<th>CONCRETE COVER 1</th>
<th>TIES OR STIRRUPS 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>10</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>6</td>
<td>14</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>7</td>
<td>17</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>8</td>
<td>19</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>9</td>
<td>21</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>10</td>
<td>24</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>11</td>
<td>27</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>12</td>
<td>32</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>13</td>
<td>43</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

#### Hooked-Bar Details for Development of Standard Hooks

- **4d_{b}** or **2\frac{1}{2}” min.**
- **d_{b}**
- **4d_{b}** through **8**
- **L_{dh}**
- **6d_{b}**
- **d_{b}**
- **4d_{b}** through **11**
- **6d_{b}** through **14 and 18**

### Compression Lap Splices

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>STANDARD</th>
<th>WITHIN SPIRALS</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td>5</td>
<td>19</td>
<td>14</td>
</tr>
<tr>
<td>6</td>
<td>23</td>
<td>17</td>
</tr>
<tr>
<td>7</td>
<td>26</td>
<td>20</td>
</tr>
<tr>
<td>8</td>
<td>30</td>
<td>23</td>
</tr>
<tr>
<td>9</td>
<td>34</td>
<td>25</td>
</tr>
<tr>
<td>10</td>
<td>38</td>
<td>29</td>
</tr>
<tr>
<td>11</td>
<td>42</td>
<td>32</td>
</tr>
</tbody>
</table>

---

1. FOR NO. 11 BARS AND SMALLER WITH NOT LESS THAN 2 1/2 IN. COVER ON SIDE OF HOOK AND 2 IN. OVER END OF HOOK. (8.29.3.2)  
2. FOR NO. 11 BARS AND SMALLER ENCLOSED WITHIN TIES OR STIRRUPS SPACED NO GREATER THAN 3d_{b} ALONG DEVELOPMENT LENGTH.  
3. BRIDGE DESIGN MANUAL (3.3.3.2.I) CALLS FOR A 1/2" DIAMETER SPIRAL BAR WITH A 4 1/2" PITCH FOR 36" DIAMETER COLUMNS WITH LIMITED RATIO OF ACTUAL AXIAL LOAD TO AXIAL LOAD CAPACITY. COLUMNS SO REINFORCED MAY BE CONSIDERED TO CONFORM TO THE LATERAL REINFORCEMENT REQUIREMENTS OF AASHTO 1989 SPECIFICATION 8.18.2.2.  
4. COMPRESSION LAP SPLICES WITHIN TIES MAY BE MULTIPLIED BY 0.83 PER AASHTO 8.32.4.I, BUT IN NO CASE LESS THAN 12 INCHES.  
5. FOR EPOXY COATED REINFORCING STEEL, VALUES SHALL BE MULTIPLIED BY 1.2.
# Development Length for Reinforcing Steel (in.)

<table>
<thead>
<tr>
<th>BAR TYPE</th>
<th>TENSION REINFORCEMENT</th>
<th>COMPRESSION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EPOXY</td>
<td>NON-EPOXY</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MIN. LENGTH</td>
</tr>
<tr>
<td></td>
<td>TOP BARS</td>
<td>OTHER BARS</td>
</tr>
<tr>
<td>AASHTO SECTION</td>
<td>8.25.2.1</td>
<td>8.25.2.3</td>
</tr>
<tr>
<td>MOD. FACTOR</td>
<td>1.4 (1.5)</td>
<td>1.4 (1.15)</td>
</tr>
<tr>
<td>BAR NO.</td>
<td>12</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>38</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>48</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>59</td>
<td>101</td>
</tr>
<tr>
<td></td>
<td>81</td>
<td>137</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>177</td>
</tr>
</tbody>
</table>

(See notes fig. 306)
NOTES:

1. FOR EPOXY-COATED BARS WITH COVER LESS THAN $3 d_b$ OR CLEAR SPACING BETWEEN BARS LESS THAN $6 d_b$. (8.25.2.3)

2. TOP BARS REFERS TO ONLY TOP ROW OF REINFORCEMENT.

3. FOR BARS SPACED LATERALLY AT LEAST 6 INCHES ON CENTER WITH AT LEAST 3 INCHES CLEAR COVER MEASURED IN THE DIRECTION OF THE SPACING, REDUCE VALUE BY 20\% ($x0.80$) (8.25.3.1), BUT NOT LESS THAN 12 INCHES PER (8.25.4)

4. BRIDGE DESIGN MANUAL SECTION 3.3.3.2.1 CALLS FOR A 1/2" DIAMETER SPIRAL BAR WITH A 4 1/2" PITCH FOR 36" DIAMETER COLUMNS WITH LIMITED RATIO OF ACTUAL AXIAL LOAD TO ALLOWABLE AXIAL LOAD CAPACITY. COLUMNS SO REINFORCED MAY BE CONSIDERED TO CONFORM TO THE LATERAL REINFORCEMENT REQUIREMENTS OF AASHTO 1989 SPECIFICATION, SECTION 8.18.2.2, SPIRAL REINFORCEMENT.

5. FOR BARS IN COMPRESSION MINIMUM DEVELOPMENT LENGTH SHALL BE $\geq 8$" (8.26)

6. VALUES SHOWN ARE FOR CLASS "C" LAP WITH $f'_c = 4,000$ P.S.I. AND $f_y = 60,000$ P.S.I. (8.32.3.2)

* BAR DIAMETER
<table>
<thead>
<tr>
<th>BAR SIZE DESIGNATION</th>
<th>NOMINAL AREA SQ. INCHES</th>
<th>NOMINAL WEIGHT POUNDS PER FT.</th>
<th>NOMINAL DIAMETER INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td># 3</td>
<td>0.11</td>
<td>0.376</td>
<td>0.375</td>
</tr>
<tr>
<td># 4</td>
<td>0.20</td>
<td>0.668</td>
<td>0.500</td>
</tr>
<tr>
<td># 5</td>
<td>0.31</td>
<td>1.043</td>
<td>0.625</td>
</tr>
<tr>
<td># 6</td>
<td>0.44</td>
<td>1.502</td>
<td>0.750</td>
</tr>
<tr>
<td># 7</td>
<td>0.60</td>
<td>2.044</td>
<td>0.875</td>
</tr>
<tr>
<td># 8</td>
<td>0.79</td>
<td>2.670</td>
<td>1.000</td>
</tr>
<tr>
<td># 9</td>
<td>1.00</td>
<td>3.400</td>
<td>1.128</td>
</tr>
<tr>
<td># 10</td>
<td>1.27</td>
<td>4.303</td>
<td>1.270</td>
</tr>
<tr>
<td># 11</td>
<td>1.56</td>
<td>5.313</td>
<td>1.410</td>
</tr>
<tr>
<td># 14</td>
<td>2.25</td>
<td>7.65</td>
<td>1.693</td>
</tr>
<tr>
<td># 18</td>
<td>4.00</td>
<td>13.60</td>
<td>2.257</td>
</tr>
</tbody>
</table>
# BAR BENDING DATA (DIMENSIONS IN INCHES)

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>180° Bend</th>
<th>90° Bend</th>
<th>135° Bend</th>
<th>BAR NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>A</td>
<td>H</td>
<td>C</td>
</tr>
<tr>
<td>3</td>
<td>2⅓</td>
<td>5</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>6</td>
<td>4½</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>3¾</td>
<td>7</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>4½</td>
<td>8</td>
<td>5¾</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>5¼</td>
<td>10</td>
<td>7½</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>11</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>9</td>
<td>9½</td>
<td>15</td>
<td>10</td>
<td>11¾</td>
</tr>
<tr>
<td>10</td>
<td>10¾</td>
<td>17</td>
<td>11½</td>
<td>13½</td>
</tr>
<tr>
<td>11</td>
<td>12</td>
<td>19</td>
<td>12¾</td>
<td>14¾</td>
</tr>
<tr>
<td>14</td>
<td>18¼</td>
<td>27</td>
<td>17½</td>
<td>21¾</td>
</tr>
<tr>
<td>18</td>
<td>24</td>
<td>36</td>
<td>23¼</td>
<td>28½</td>
</tr>
</tbody>
</table>

**BENDING TOLERANCES:** Refer to Section CMS 509
<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>STANDARD BEND (DEGREES)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
</tr>
<tr>
<td>*3</td>
<td>1½”</td>
</tr>
<tr>
<td>*4</td>
<td>2”</td>
</tr>
<tr>
<td>*5</td>
<td>2½”</td>
</tr>
<tr>
<td>*6</td>
<td>3”</td>
</tr>
<tr>
<td>*7</td>
<td>3½”</td>
</tr>
<tr>
<td>*8</td>
<td>4”</td>
</tr>
<tr>
<td>*9</td>
<td>6¾”</td>
</tr>
<tr>
<td>*10</td>
<td>7⅛”</td>
</tr>
<tr>
<td>*11</td>
<td>8”</td>
</tr>
<tr>
<td>*14</td>
<td>12½”</td>
</tr>
<tr>
<td>*18</td>
<td>16”</td>
</tr>
</tbody>
</table>

**NOTE:**

"D" IS THE DIAMETER OF THE BEND PER CONSTRUCTION AND MATERIAL SPECIFICATIONS ITEM 509.05
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE

CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET

PRESTRESSED BOX BEAM DECK WITH DEFLECTOR PARAPET

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 310
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE  
CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET  
PRESTRESSED BOX BEAM DECK WITH OVER THE SIDE DRAINAGE

SEALING OF CONCRETE-surfaces, SUPERSTRUCTURE

Figure 311
Sample problem: Using load factor design procedures determine the slab thickness and main reinforcement for a deck slab with an 9'-6" stringer spacing and an HS25-44 loading.

\[ S = 9'-6" \text{ minus } 6" = 9'-0" \]
\[ T_{\text{min}} = (S + 17) / (36) = 0.72' = 8.67" > 8\frac{1}{2}" \]
\[ f_c = 4500 \text{ psi} \]
\[ f_y = 6000 \text{ psi} \]
\[ \phi = 0.9 (8.16, 1.2.2) \]
\[ Z = 130 \text{ k/in(top), 170 k/in(bottom) (8.16.8.4)} \]
\[ n = 8 \]
\[ \text{Impact} = 30\% \]

Dead load \( W \):
\[ \text{Slab} = (0.73')(1.0')(0.15 \text{ k/ft}^2) = 0.110 \text{ k/ft} \]
\[ \text{FWS} = 60 \text{ p.s.f.}(1.0') = 0.060 \text{ k/ft} \]
\[ \text{TOTAL DEAD LOAD} \ (W) = 0.170 \text{ k/ft} \]

**Design Moments**:
\[ DL = (0.125)(W)(S^2)(0.8) = (0.125)(0.170)(9.0^2)(0.8) = 1.38 \text{ ft}-k \]
\[ LL + I = (S + 2)(1.2)(1.3)(0.8)/32 = 7.15 \text{ ft}-k \] \( (3.24, 3.1) \)
\[ Mu = 1.3[DL+1.67(LL+I)] = 1.3[1.38+1.67(7.15)] = 17.32 \text{ ft}-k \] \( (3.22) \)
\[ Mw = \text{Service load moment} = DL + LL + I = 8.53 \text{ ft}-k \]

**Top Reinforcement**

\[ R = Mu/\phi bd^2 \]
\[ = 650.77 \text{ psi} \]
\[ \rho = 0.01196 \]
\[ A_s = (0.01196)(12)(5.438) = 0.78 \text{ in}^2/\text{ft} \]

Try #5 bars at 4.75" in \((A_s=0.78 \text{ in}^2/\text{ft})\)

**Bottom Reinforcement**

\[ R = (17.32)(1000)/(0.9)(1)(5.938^2) \]
\[ = 545.79 \text{ psi} \]
\[ \rho = 0.00985 \]
\[ A_s = (0.00985)(12)(5.938) = 0.70 \text{ in}^2/\text{ft} \]

Try #5 bars at 5.25" in \((A_s=0.71 \text{ in}^2/\text{ft})\)

**Check steel spacing** (8.16.8.4)

\[ d_c = 2(0.625/2) - 2.312 \text{ in} \]
\[ A_s = 2(0.372)(4.75) - 2(0.96) = 0.78 \text{ in}^2/\text{ft} \]

\[ f_y (\text{all.}) = 60 \text{ ksi} \]

\[ f_y (\text{act.}) = 52.23, \text{ use } 36.0 \text{ ksi max.} \]

\[ f_y (\text{act.}) = (8.53)(12)/(0.78)(0.89)(5.438) = 27.1 \text{ ksi} \]

\[ A_w = 2(0.812)(5.25) - 19.01 \text{ in}^2/\text{ft} \]

\[ f_y (\text{act.}) = (8.53)(12)/(0.71)(0.89)(5.938) = 27.28 \text{ ksi} \]

\[ \text{Use #5 bars @ 4.75" c/c (As=0.78 in}^2/\text{ft}) \]

\[ \text{Top and bottom bars shall coincide based on BDM Section 302.2.4.2} \]

Figure 312
EXAMPLE: Stringer spacing of 9'-6"

$S = 9'-6" \text{ minus } 6" = 9'-0"

$T = 8\frac{3}{4}"$, $A_{(\text{top})} = 0.78 \text{ in}^2/\text{ft}$, $A_{(\text{bott.})} = 0.71 \text{ in}^2/\text{ft}$

PRIMARY REINFORCEMENT: (SEE SEC. 302.2.4.2)
Use #5 bars (top & bott.), both at 4.75" c/c, $A = 0.78 \text{ in}^2$

DISTRIBUTIONAL REINFORCEMENT:
As $(\text{top}) = 0.33(0.78) = 0.26 \text{ in}^2/\text{ft}$
Use #4 bars at 13 equal spaces ($A = 0.27 \text{ in}^2/\text{ft}$)

As $(\text{bott.}) = \frac{220}{\sqrt{5}} = 73.33\%$, use 67% max.\hspace{2cm} (3.24.10.2)

$= (0.67)(0.78) = 0.52 \text{ in}^2/\text{ft}$ in mid-half of span

$= (0.50)(0.52) = 0.26 \text{ in}^2/\text{ft}$ in each outer quarter (3.24.10.3)

Use 9 #5 bars at 7" c/c in mid-half of span and 2 #5 bar in each outer quarter.

\* AASHTO

\* By load factor procedures. For design data and sample problem, see Fig. 312
Note: This Figure is for the design of a reinforced concrete deck on new steel beams/girders using HS25.

**Slab Thickness “T”**

Area of Steel “As” (sq. in. per ft.)

- 7 in.
- 8 in.
- 9 in.
- 10 in.
- 11 in.
- 12 in.
- 13 in.
- 14 in.

Effective Span “S” (feet)

- 8½” min.
- 8¾”
- 9”
- 9¼”
- 9½”
- 9¾”
- 10”
- 10¼”
- 10½”

As, Top of slab
(d-T-3.3125)

As, Bottom of slab
(d-T-2.8125)
Note: This Figure is for the design of a reinforced concrete deck on existing steel beams/girders using HS20-44.

Slab Thickness "T"

8½” min. 8¾” 9” 9¼” 9½” 9¾” 10” 10¼” 10½”

Area of Steel "As" (Sq. In per ft.)

0.4 0.5 0.6 0.7 0.8 0.9

Effective Span "S" (feet)

5 6 7 8 9 10 11 12 13 14

As, Top of slab
(d=T-3.3125)

As, Bottom of slab
(d=T-2.8125)
Sample Problem: Using load factor design procedures, determine whether the reinforcing steel design given in the previous example is adequate to sustain a 3'-0" cantilever slab carrying a 36" deflector parapet and an HS25-44 loading.

\[ P_1 = 20 \text{ Kip} (3.24.3) \]
\[ P_2 = 10 \text{ kip} (2.71.3) \]

**Truck Load Distribution Factor:**
\[ E_1 = 0.8 \times X_1 + 3.75 \text{ (3.24.5.1.1)} \]
\[ E_1 = 0.8 \times (0.5) + 3.75 = 4.15 \]

**Railing Load Distribution Factor:**
\[ E_2 = 0.8 \times X_2 + 5.0 \text{ (3.24.5.2)} \]
\[ E_2 = 0.8 \times (2.5) + 5.0 = 7.0 \]

Uniform Dead Load: (per ft of length)
- Slab = \( w_1 = (0.83)(1.0)(0.15) = 0.125 \text{ Kip/ft} \)
- F.W.S. = \( w_2 = 60 \text{ psf}(1.0) = 0.060 \text{ Kip/ft} \)

Concentrated Dead Load: (per ft of length)
- Parapet = \( P = 0.47 \text{ Kip located @ CG} \)

Dead Load Moment:
\[ DLM = \frac{1}{2} w_1 L^2 + \frac{1}{2} w_2 (L - 1.5)^2 + P(L - 0.5) \]
\[ DLM = \frac{1}{2} (0.125)(3.00)^2 + \frac{1}{2} (0.06)(3.0 - 1.5)^2 \]
\[ + 0.47(3.00 - 0.5) = 1.81 \text{ Kip-ft} \]

Live Load Moment:
- Truck Load Moment + Impact = TLM + \( I = 1.3 \times X_1 \left( \frac{P_1}{E_1} \right) \)
\[ TLM + I = 1.3 \times (0.5) \times \frac{20}{4.15} = 3.13 \text{ Kip-ft} \]

Railing Load Moment = RLM = \( h \left( \frac{P_2}{E_2} \right) \)
\[ RLM = 3.0 \left( \frac{10.0}{7.0} \right) = 4.29 \text{ Kip-ft} \]

Live Load Moment = Greater of TLM+1 & RLM = 4.29 kip-ft

Design Moments:
\[ M_u = 1.3 [ \text{ DLM + 1.67 LLM } ] \]
\[ M_u = 1.3 [ 1.81 + 1.67 (4.29) ] = 11.67 \text{ Kip-ft} \]

\[ M_w = \text{ Service Load Moment} = \text{ DLM + LLM} \]
\[ M_w = 1.81 + 4.29 = 6.10 \text{ Kip-ft} \]

CHECK TOP REINFORCEMENT

\[ \odot \text{AASHTO} \]
CANTILEVER SLAB DESIGN

\( f'_c = 4500 \text{ psi} \)
\( f_y = 60000 \text{ psi} \)
\( \phi = 0.9 \quad (8.16.1.2.2) \)
\( Z = 130 \text{ kip/in (top steel)} \quad (8.16.8.4) \)

\[ R = \frac{M_y}{\phi b d^2} = \frac{11.67 (1000)}{0.9 (1) (6.6875)^2} = 289.94 \text{ psi} \]
\[ p = 0.00503 \quad (\text{see fig. 312}) \]
\[ A_s = \rho bd = 0.00503 (12) (6.6875) = 0.40 \text{ in}^2 / \text{ft} < 0.78 \text{ in}^2 / \text{ft} \quad \star \text{ ok} \]

Check Steel Spacing (8.16.8.4) \(
\[ f_s (\text{ALL}) = \frac{Z}{(d_c - A_s)^{1/3}} < 0.6f_y \]
\[ \frac{130}{(2.312)(21.96)}^{1/3} = 35.11 \text{ ksi} \]
\[ f_s (\text{ACT}) = \frac{M_y}{A_s \cdot d_t} \]
\[ f_s (\text{ACT}) = \frac{6.10 (12)}{(0.78)(0.91)(6.6875)} = 15.42 \text{ ksi} < 35.11 \text{ ksi} \text{ ok} \]

\( \star \) - Steel reinforcing ratio for top steel taken from Transverse Slab Design example (#5 bars @ 4.75”).

AASHTO
TYPICAL CONCRETE
DECK HAUNCH DETAIL
TYPICAL CONCRETE DECK HAUNCH DETAIL
Plan View

Crossframes for Dog-legged Splices

Figure 319
FIT-UP SHOULD NOT BE INCLUDED IN ESTABLISHING THE NUMBER OF BEAMS.
IN ACTUAL CONSTRUCTION FIT-UP WILL BE ABSORBED IN THE OVERHANG

DO - DESIGN OVERHANG (MINIMUM 2", MAXIMUM 8") BOX BEAM DESIGN
WIDTH SHOULD BE SELECTED SO DO STAYS WITHIN ACCEPTABLE RANGE.
ABUTMENT SEALING LIMITS (FOR STEEL BEAM BRIDGE)

ABUTMENT SEALING LIMITS (FOR PRESTRESSED BOX BEAM BRIDGE)

WINGWALL SEALING LIMITS (TURNBACK WALL ON U-TYPE ABUTMENT)

WINGWALL SEALING LIMITS (STRAIGHT WING ABUTMENT)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 321
Seal end of pier cap

Seal entire surface area of column

Seal entire surface area

SECTION A-A

Ground Line

ELEVATION

PIER SEALING LIMITS
(EXPOSED TO DEICER SPRAY)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 322
ABUTMENT SEALING LIMITS
(FOR INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

ABUTMENT SEALING LIMITS
(FOR SEMI-INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 323
INTEGRAL ABUTMENT

BRIDGE LIMIT

CONCRETE DECK SLAB

APPROACH SLAB

CONSTR. JOINT

STEEL STRINGER

NEOPRENE WATERPROOFING

CONSTR. JOINT

POROUS BACKFILL WITH FABRIC

PILEs-PLACE PILE WEB PARALLEL TO C BEARING.

Figure 324
SEMI-INTEGRAL ABUTMENT

- Φ BEARING
- BRIDGE LIMIT
- CONCRETE DECK SLAB
- APPROACH SLAB
- ELASTOMERIC BEARING PAD
- CONSTR. JOINT
- SUPPORT POST
- WATERPROOFING MATERIAL
- STEEL STRINGER
- POLYSTYRENE
- SLOPE PROTECTION
- 1
- 2
- POROUS BACKFILL W/ FILTER FABRIC
- Φ PILES - PLACE PILE WEB PERPENDICULAR TO Φ BEARING.

Figure 325
BRIDGE WITH SIDEWALK - VERTICAL FENCE

1'-0" min. horizontal distance between face of curb and edge of fence.

1'-0" to 3'-7" max. overhang

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 326
BRIDGE WITH SIDEWALK - VERTICAL FENCE

1'-0" min.
horizontal distance
between face of
curb and edge of
fence.

1'-0" to 3'-7"
max. overhang

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 327
Railing shall be designed in accordance with the AASHTO standard specification for pedestrian/bicycle railing.

Pedestrian Fencing on Structures

Deflector Parapet with Fencing

See Std Drwg. VPF-1-90 for fencing details.
FIGURE 339

SECTION A-A

(ALL DIMENSIONS PERPENDICULAR TO MSE WALL)
MSE WALL AND COPING DETAIL

COPING EXPANSION JOINTS

MSE WALL COPING

ALL REINFORCING STEEL TO BE EPOXY COATED
If x is less than 8 ft., use select granular backfill material between soil reinforcement.

See roadway plans for pavement build up.
The web splice plate shall not encroach upon the beam fillet.

The inside splice place shall not encroach upon the beam fillet.

Figure 334
MS22.5 TRUCK

CONCENTRATED LOAD - 100 kN FOR MOMENT
- 145 kN FOR SHEAR

UNIFORM LOAD 11.7 kN PER LINEAR METER OF LOAD LANE

MS22.5 LANE LOADING
CLEARANCE AND LOAD LANE WIDTH

Figure 30IM
<table>
<thead>
<tr>
<th>MARK</th>
<th>NUMBER</th>
<th>LENGTH (mm)</th>
<th>WEIGHT (kg)</th>
<th>TYPE</th>
<th>DIMENSIONS (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>REAR</td>
<td>FWD</td>
<td>TOTAL</td>
<td>A</td>
</tr>
<tr>
<td>PIERs</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPI6W01</td>
<td>1</td>
<td>1</td>
<td>8610</td>
<td>13</td>
<td>12</td>
</tr>
<tr>
<td>SPI6W02</td>
<td>1</td>
<td>1</td>
<td>4040</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>PI6W01</td>
<td>14</td>
<td>14</td>
<td>2290</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>PI6W02</td>
<td>12</td>
<td>12</td>
<td>2360</td>
<td>44</td>
<td></td>
</tr>
<tr>
<td>PI9W01</td>
<td>17</td>
<td>17</td>
<td>6095</td>
<td>232</td>
<td></td>
</tr>
<tr>
<td>PI9W02</td>
<td>15</td>
<td>15</td>
<td>1800</td>
<td>60</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>3</td>
<td>6</td>
<td>2270</td>
<td></td>
</tr>
<tr>
<td>PI9W03</td>
<td>ser</td>
<td>ser</td>
<td>ser to</td>
<td>32</td>
<td>890</td>
</tr>
<tr>
<td></td>
<td>of 6</td>
<td>of 6</td>
<td>of 6</td>
<td>2470</td>
<td></td>
</tr>
<tr>
<td>DPI9W01</td>
<td>28</td>
<td>28</td>
<td>865</td>
<td>54</td>
<td></td>
</tr>
<tr>
<td>DPI9W02</td>
<td>16</td>
<td>16</td>
<td>990</td>
<td>35</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>526</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ABUTMENT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A16W01</td>
<td>8</td>
<td>8</td>
<td>16</td>
<td>3160</td>
<td>78</td>
</tr>
<tr>
<td>A16W02</td>
<td>4</td>
<td>4</td>
<td>8</td>
<td>3050</td>
<td>38</td>
</tr>
<tr>
<td>A16W03</td>
<td>12</td>
<td>12</td>
<td>24</td>
<td>3560</td>
<td>136</td>
</tr>
<tr>
<td>A16W04</td>
<td>7</td>
<td>7</td>
<td>3280</td>
<td>36</td>
<td>6</td>
</tr>
<tr>
<td>A19W01</td>
<td>16</td>
<td>16</td>
<td>8585</td>
<td>307</td>
<td></td>
</tr>
<tr>
<td>A19W02</td>
<td>16</td>
<td>16</td>
<td>2560</td>
<td>92</td>
<td>1</td>
</tr>
<tr>
<td>TOTAL</td>
<td>687</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SUPERSTRUCTURE</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S16W01</td>
<td>619</td>
<td>9145</td>
<td>8657</td>
<td>Str.</td>
<td></td>
</tr>
<tr>
<td>S16W02</td>
<td>61</td>
<td>4065</td>
<td>385</td>
<td>Str.</td>
<td></td>
</tr>
<tr>
<td>S16W03</td>
<td>530</td>
<td>12650</td>
<td>10405</td>
<td>Str.</td>
<td></td>
</tr>
<tr>
<td>S16W04</td>
<td>466</td>
<td>1098</td>
<td>794</td>
<td>18</td>
<td>660</td>
</tr>
<tr>
<td>S25W01</td>
<td>240</td>
<td>876</td>
<td>835</td>
<td>17</td>
<td>230</td>
</tr>
<tr>
<td>S25W02</td>
<td>16</td>
<td>2540</td>
<td>161</td>
<td>1</td>
<td>300</td>
</tr>
<tr>
<td>TOTAL</td>
<td>2133</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The bar size number is specified on the plans in the Bar Mark column. The first two digits indicate the bar size number. For example, a 15W01 is a 15W bar. Bar dimensions shown are to cut or cut unless otherwise indicated. R indicates inside radius, unless otherwise noted, "STD" written in place of a dimension indicates a standard bend at the end of the bar.

All reinforcing steel to be epoxy coated.
### TENSION SPLICES (mm)

<table>
<thead>
<tr>
<th>BAR SIZE</th>
<th>EPOXY</th>
<th>NON-EPOXY</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLC, CLR. SIZE</td>
<td>TOP</td>
<td>OTHER</td>
</tr>
<tr>
<td>13</td>
<td>890</td>
<td>840</td>
</tr>
<tr>
<td>16</td>
<td>1090</td>
<td>1040</td>
</tr>
<tr>
<td>19</td>
<td>1320</td>
<td>1240</td>
</tr>
<tr>
<td>22</td>
<td>1680</td>
<td>1570</td>
</tr>
<tr>
<td>25</td>
<td>2210</td>
<td>2080</td>
</tr>
<tr>
<td>29</td>
<td>2790</td>
<td>2640</td>
</tr>
<tr>
<td>32</td>
<td>3530</td>
<td>3350</td>
</tr>
<tr>
<td>36</td>
<td>4340</td>
<td>4110</td>
</tr>
</tbody>
</table>

**NOTES:**

1. **FOR EPOXY BARS WITH COVER LESS THAN \(3d_b\) OR CLEAR SPACING BETWEEN BARS LESS THAN \(6d_b\) (8.25.2.3)**

2. **TOP BARS REFERS TO ONLY TOP ROW OF REINFORCEMENT.**

3. **FOR BARS SPACED LATERALLY AT LEAST 150 mm ON CENTER WITH AT LEAST 75 mm CLEAR COVER MEASURED IN THE DIRECTION OF THE SPACING, REDUCE VALUE BY 20% \(\times 0.80\) (8.25.3.1), BUT NOT LESS THAN 300 mm PER 8.32.3.1**

4. **VALUES SHOWN ARE FOR CLASS “C” LAP WITH \(f’c = 28 \text{ MPa. AND } F_y = 420 \text{ MPa (8.32.3.2)****}}**

**BAR DIAMETER**

---

*Figure 303M*
### Development Length for Uncoated Standard Hooks in Tension

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>MOD. FACTOR</th>
<th>BASIC LENGTH, lbd</th>
<th>CONCRETE COVER 1</th>
<th>TIES OR STIRRUPS 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>250</td>
<td>180</td>
<td>0.7</td>
<td>0.8</td>
</tr>
<tr>
<td>16</td>
<td>300</td>
<td>200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>360</td>
<td>250</td>
<td>280</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>430</td>
<td>300</td>
<td>360</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>480</td>
<td>330</td>
<td>380</td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>530</td>
<td>380</td>
<td>430</td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>610</td>
<td>430</td>
<td>480</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>690</td>
<td>480</td>
<td>560</td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>810</td>
<td>—</td>
<td></td>
<td></td>
</tr>
<tr>
<td>57</td>
<td>1100</td>
<td>—</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. For No. 36 bars and smaller, with not less than 65 mm cover on side of hook and 50 mm over end of hook (8.29.3.2).

2. For No. 36 bars and smaller enclosed within ties or stirrups spaced no greater than 3d_b along development length.

3. Bridge Design Manual Section 303.3.2.i calls for *"13 spiral bar with a 115 mm pitch for 915 mm diameter columns with limited ratio of actual axial load to axial load capacity. Columns so reinforced may be considered to comply with the lateral reinforcement requirements of AASHTO 1997 Specification 8.18.2.2.*

4. Compression lap splices within ties may be multiplied by 0.83 per AASHTO 8.32.4.i, but in no case less than 300 mm.

5. For epoxy coated reinforcing steel, values shall be multiplied by 1.2.
<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>BAR TYPE</th>
<th>TENSION</th>
<th>REINFORCEMENT</th>
<th>EPOXY</th>
<th>NON-EPOXY</th>
<th>COMPRESSION</th>
<th>MIN. LENGTH</th>
<th>WITHIN SPIRAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO SECTION</td>
<td>8.25.2.1 8.25.2.3</td>
<td>8.25.2.1 8.25.2.3</td>
<td>8.25.2.3</td>
<td>8.25.2.3</td>
<td>8.25.2.3 8.25.3.3</td>
<td>8.25.2.3 8.25.3.3</td>
<td>8.25.2.1</td>
<td>8.25.1</td>
</tr>
<tr>
<td>BASIC LENGTH</td>
<td>1.4 (1.5) ≤ 1.7</td>
<td>1.4 (1.15)</td>
<td>1.5</td>
<td>1.5 (1.75)</td>
<td>1.5 (1.75)</td>
<td>1.4</td>
<td>1.0</td>
<td>1.0 (1.75)</td>
</tr>
<tr>
<td>13</td>
<td>300</td>
<td>510</td>
<td>480</td>
<td>460</td>
<td>360</td>
<td>360</td>
<td>300</td>
<td>430</td>
</tr>
<tr>
<td>16</td>
<td>380</td>
<td>660</td>
<td>610</td>
<td>560</td>
<td>430</td>
<td>430</td>
<td>530</td>
<td>380</td>
</tr>
<tr>
<td>19</td>
<td>460</td>
<td>790</td>
<td>740</td>
<td>690</td>
<td>530</td>
<td>510</td>
<td>410</td>
<td>640</td>
</tr>
<tr>
<td>22</td>
<td>580</td>
<td>990</td>
<td>940</td>
<td>860</td>
<td>660</td>
<td>660</td>
<td>510</td>
<td>810</td>
</tr>
<tr>
<td>25</td>
<td>760</td>
<td>1300</td>
<td>1220</td>
<td>1140</td>
<td>860</td>
<td>860</td>
<td>660</td>
<td>1070</td>
</tr>
<tr>
<td>29</td>
<td>970</td>
<td>1650</td>
<td>1550</td>
<td>1450</td>
<td>1120</td>
<td>1040</td>
<td>840</td>
<td>1350</td>
</tr>
<tr>
<td>32</td>
<td>1220</td>
<td>2080</td>
<td>1980</td>
<td>1830</td>
<td>1400</td>
<td>1370</td>
<td>1070</td>
<td>1700</td>
</tr>
<tr>
<td>36</td>
<td>1500</td>
<td>2570</td>
<td>2410</td>
<td>2260</td>
<td>1730</td>
<td>1700</td>
<td>1300</td>
<td>2110</td>
</tr>
<tr>
<td>43</td>
<td>2060</td>
<td>3480</td>
<td>3300</td>
<td>3070</td>
<td>2360</td>
<td>2310</td>
<td>1780</td>
<td>2870</td>
</tr>
<tr>
<td>57</td>
<td>2640</td>
<td>4500</td>
<td>4270</td>
<td>3960</td>
<td>3050</td>
<td>2970</td>
<td>2290</td>
<td>3710</td>
</tr>
</tbody>
</table>

(SEE NOTES FIG. 306M)
NOTES:

1. FOR EPOXY COATED BARS WITH COVER LESS THAN $3d_b$ OR CLEAR SPACING BETWEEN BARS LESS THAN $6d_b$, (8.25.2.3)

2. TOP BARS REFERS TO ONLY TOP ROW OF REINFORCEMENT.

3. FOR BARS SPACED LATERALLY AT LEAST 150mm ON CENTER WITH AT LEAST 75 mm CLEAR COVER MEASURED IN THE DIRECTION OF THE SPACING, REDUCE VALUES BY 20% ($0.80$) (8.25.3.1), BUT NOT LESS THAN 300 mm PER (8.25.4)

4. BRIDGE DESIGN MANUAL SECTION 303.3.2.1 CALLS FOR A $16M$ SPIRAL BAR WITH A 115 mm PITCH FOR 915 mm DIAMETER COLUMNS WITH LIMITED RATIO OF ACTUAL AXIAL LOAD TO ALLOWABLE AXIAL LOAD CAPACITY. COLUMNS SO REINFORCED MAY BE CONSIDERED TO CONFORM TO THE LATERAL REINFORCEMENT REQUIREMENTS OF AASHTO 1997 SPECIFICATION, SECTION 8.18.2.2, SPIRAL REINFORCEMENT.

5. FOR BARS IN COMPRESSION THE MINIMUM DEVELOPMENT LENGTH SHALL BE $\geq 200mm$ (8.26)

6. VALUES SHOWN ARE FOR CLASS "C" LAP WITH $f'_c = 28$ MPa AND $F_y = 420$ MPa (8.32.3.2)
## ASTM Standard

### Reinforcing Bars

<table>
<thead>
<tr>
<th>Bar Size Designation</th>
<th>Nominal Area (mm²)</th>
<th>Nominal Weight (kg/m)</th>
<th>Nominal Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 10</td>
<td>71</td>
<td>0.56</td>
<td>9.5</td>
</tr>
<tr>
<td># 13</td>
<td>129</td>
<td>0.994</td>
<td>12.7</td>
</tr>
<tr>
<td># 16</td>
<td>200</td>
<td>1.552</td>
<td>15.9</td>
</tr>
<tr>
<td># 19</td>
<td>284</td>
<td>2.235</td>
<td>19.1</td>
</tr>
<tr>
<td># 22</td>
<td>388</td>
<td>3.042</td>
<td>22.2</td>
</tr>
<tr>
<td># 25</td>
<td>511</td>
<td>3.973</td>
<td>25.4</td>
</tr>
<tr>
<td># 29</td>
<td>646</td>
<td>5.06</td>
<td>28.7</td>
</tr>
<tr>
<td># 32</td>
<td>821</td>
<td>6.404</td>
<td>32.3</td>
</tr>
<tr>
<td># 36</td>
<td>1008</td>
<td>7.907</td>
<td>35.8</td>
</tr>
<tr>
<td># 43</td>
<td>1454</td>
<td>11.385</td>
<td>43</td>
</tr>
<tr>
<td># 57</td>
<td>2585</td>
<td>20.239</td>
<td>57.3</td>
</tr>
</tbody>
</table>

Figure 307M
### BAR BENDING DATA (DIMENSIONS IN mm)

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>180° Bend</th>
<th>90° Bend</th>
<th>135° Bend</th>
<th>BAR NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>A</td>
<td>H</td>
<td>C</td>
</tr>
<tr>
<td>10M</td>
<td>60</td>
<td>130</td>
<td>100</td>
<td>80</td>
</tr>
<tr>
<td>13M</td>
<td>75</td>
<td>155</td>
<td>115</td>
<td>101</td>
</tr>
<tr>
<td>16M</td>
<td>95</td>
<td>180</td>
<td>125</td>
<td>127</td>
</tr>
<tr>
<td>19M</td>
<td>115</td>
<td>205</td>
<td>145</td>
<td>153</td>
</tr>
<tr>
<td>22M</td>
<td>135</td>
<td>255</td>
<td>190</td>
<td>179</td>
</tr>
<tr>
<td>25M</td>
<td>150</td>
<td>280</td>
<td>205</td>
<td>200</td>
</tr>
<tr>
<td>29M</td>
<td>240</td>
<td>380</td>
<td>255</td>
<td>298</td>
</tr>
<tr>
<td>32M</td>
<td>275</td>
<td>430</td>
<td>290</td>
<td>339</td>
</tr>
<tr>
<td>36M</td>
<td>305</td>
<td>485</td>
<td>325</td>
<td>377</td>
</tr>
<tr>
<td>43M</td>
<td>465</td>
<td>685</td>
<td>440</td>
<td>551</td>
</tr>
<tr>
<td>57M</td>
<td>610</td>
<td>915</td>
<td>590</td>
<td>724</td>
</tr>
</tbody>
</table>

**BENDING TOLERANCES:** Refer to Section CMS 509
### STD. BAR LENGTH DEDUCTIONS FOR COMMON BENDS, (mm)

<table>
<thead>
<tr>
<th>BAR NO.</th>
<th>STANDARD BEND (DEGREES)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>45</td>
<td>D</td>
<td>90</td>
<td>D</td>
<td>135</td>
<td>D</td>
</tr>
<tr>
<td>10M</td>
<td>40</td>
<td>6</td>
<td>60</td>
<td>25</td>
<td>40</td>
<td>25</td>
<td>60</td>
</tr>
<tr>
<td>13M</td>
<td>50</td>
<td>6</td>
<td>75</td>
<td>25</td>
<td>50</td>
<td>32</td>
<td>75</td>
</tr>
<tr>
<td>16M</td>
<td>65</td>
<td>10</td>
<td>95</td>
<td>40</td>
<td>65</td>
<td>41</td>
<td>95</td>
</tr>
<tr>
<td>19M</td>
<td>75</td>
<td>10</td>
<td>115</td>
<td>50</td>
<td>75</td>
<td>51</td>
<td>115</td>
</tr>
<tr>
<td>22M</td>
<td>90</td>
<td>13</td>
<td>135</td>
<td>50</td>
<td>90</td>
<td>57</td>
<td>135</td>
</tr>
<tr>
<td>25M</td>
<td>100</td>
<td>13</td>
<td>150</td>
<td>65</td>
<td>100</td>
<td>64</td>
<td>150</td>
</tr>
<tr>
<td>29M</td>
<td>160</td>
<td>16</td>
<td>240</td>
<td>90</td>
<td>160</td>
<td>86</td>
<td>240</td>
</tr>
<tr>
<td>32M</td>
<td>180</td>
<td>19</td>
<td>275</td>
<td>100</td>
<td>180</td>
<td>95</td>
<td>275</td>
</tr>
<tr>
<td>36M</td>
<td>205</td>
<td>19</td>
<td>305</td>
<td>100</td>
<td>205</td>
<td>108</td>
<td>305</td>
</tr>
<tr>
<td>43M</td>
<td>310</td>
<td>25</td>
<td>465</td>
<td>150</td>
<td>310</td>
<td>143</td>
<td>465</td>
</tr>
<tr>
<td>57M</td>
<td>405</td>
<td>35</td>
<td>610</td>
<td>200</td>
<td>405</td>
<td>191</td>
<td>610</td>
</tr>
</tbody>
</table>

**NOTE:**
“D” is the diameter of the bend per construction and material specifications item 509.05
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE

CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET

PRESTRESSED BOX BEAM DECK WITH DEFLECTOR PARAPET

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 310M
CONCRETE DECKS WITH OVER THE SIDE DRAINAGE

CONCRETE DECKS WITH CURBS, SIDEWALKS AND PARAPET

CONCRETE DECK WITH DEFLECTOR PARAPET

PRESTRESSED BOX BEAM DECK WITH OVER THE SIDE DRAINAGE

SEALING OF CONCRETE SURFACES, SUPERSTRUCTURE

Figure 311M
Sample problem: Using load factor design procedures determine the slab thickness and main reinforcement for a deck slab with an 2.9 m stringer spacing and an MS22.5 loading.

$$S = 2900 \text{ mm minus } 150 \text{ mm} = 2750 \text{ mm}$$

$$T = (S+5200)/(36) \approx 220.8 \approx 215 \text{ mm}, \text{ use } 225 \text{ mm}$$

$$f'_c = 31 \text{ MPa}$$

$$f_y = 420 \text{ MPa}$$

$$\phi = 0.9 (8.16.1.2.2)$$

$$Z = 23000 \text{ KN/m (top)}, 30000 \text{ KN/m (bottom)} (8.16.8.4)$$

$$n = 8$$

$$\text{Impact} = 30\%$$

**Dead load** \( W \):

- Slab \(= (0.225\text{m})(1.0\text{m})(23.57\text{kn/m}^3) = 5.303 \text{ KN/m} \)
- FWS \(= 2.870 \text{ KN/m}^2(1.0\text{m}) = 2.87 \text{ KN/m} \)

**TOTAL DEAD LOAD** \( (W) = 8.173 \text{ KN/m} \)

**Design Moments**:

\[ DL = (0.125)(W)(S^2)(0.8) = (0.125)(8.173)(2.75^2)(0.8) = 6.181 \text{ KN-m} \]

\[ LL + L = (S+610)(100)(1.3)(0.8)/(9750) = 32.256 \text{ KN-m} \]

\[ Mu = 1.3[DL+1.67(LL+L)] = 1.3[6.181+1.67(32.256)] = 78.06 \text{ KN-m} \]

\[ Mw = \text{Service load moment} = DL + (LL+L) = 38.44 \text{ KN-m} \]

**Top Reinforcement**

\[ R = Mu/\phi bd^2 \]

\[- 4.489 \text{ MPa} \]

\[- 0.01180 \]

\[- 1640 \text{ mm}^2 / \text{m} \]

Try 16M bars at 120 mm (As=1667 mm$^2$/m)

**Bottom Reinforcement**

\[ R = (78.06)(1000)/(0.9)(1)(1392) \]

\[- 3.754 \text{ MPa} \]

\[- 0.00969 \]

\[- 1473 \text{ mm}^2 / \text{m} \]

Try 16M bars at 135 mm (As=1482 mm$^2$/m)

**Check steel spacing** (8.16.8.4)

\[ d_c = 50+8 = 58 \text{ mm} \]

\[- 1.32 \]

\[- 1.2 \text{ m} \]

\[- 13920 \text{ mm}^2 / \text{m} \]

\[ f_y = 420 \text{ MPa} \]

\[ 0.6 \text{ (f_y)} \]

\[ 0.62 \]

\[- 12960 \text{ mm}^2 / \text{m} \]

\[ f_s (\text{all.}) = 23000/[(58)(13920)] \leq 0.6(420) \]

\[- 247 \text{ MPa} \leq 252 \text{ MPa} \]

\[- 0.252 \]

\[- 252 \text{ MPa} \]

\[- 51.43 \]

\[- 186.40 \text{ MPa} \text{ (OK)} \]

\[ f_s (\text{act.}) = Ma/As \]

\[- 30000/[(48)(12960)] \leq 252 \text{ MPa} \]

\[- 351.43 \]

\[- 191.73 \text{ MPa} \text{ (OK)} \]

\[ f_s (\text{act.}) = 38.44(1000^2)/(1667)(0.89)(139) \]

\[ = 186.40 \text{ MPa} \text{ (OK)} \]

\[ f_s (\text{act.}) = 38.44(1000^2)/(1482)(0.89)(152) \]

\[ = 191.73 \text{ MPa} \text{ (OK)} \]

**AASHTO**

\[ \text{Use 16M bars @ 120mm c/cl(As=1667 mm}^2) \]

\[ \text{Top and bottom bars shall coincide based on BDM Section 302.2.4.2} \]
EXAMPLE: Stringer spacing of 2900 mm
\[ S = 2900 \text{ mm} - 150 \text{ mm} = 2750 \text{ mm} \]
\[ T = 225 \text{ mm}, \text{As (top)} = 1667 \text{ mm}^2, \text{As (bott.)} = 1482 \text{ mm}^2 \]

PRIMARY REINFORCEMENT: (SEE SEC. 302.2.4.2)
Use #16M bars (top & bott.), both at 120 mm c/c, As=1667 mm²

DISTRIBUTIONAL REINFORCEMENT: (SEE SEC. 302.2.4.1)
As (top)=(0.33)(l667)= 550 mm²/m
Use #13M bars at 13 equal spaces (As= 578 mm²)
As (bott.) : \( \frac{3840}{\sqrt{S}} = 73.22\% \), use 67% max. \( (3.24.10.2) \)
\[ = (0.67)(1667) = 1117 \text{ mm}^2 / \text{m in mid-half of span} \]
\[ = (0.50)(1117) = 559 \text{ mm}^2 / \text{m in each outer quarter} \ (3.24.10.3) \]
Use 9 #16M bars at 180 mm c/c in mid-half of span and 2 #16M bar in each outer quarter.

\( \text{® AASHTO} \)

* By load factor procedures. For design data and sample problem, see Fig. 312M

Figure 313M
Note: This Figure is for the design of a reinforced concrete deck on new steel beams/girders using MS22.5.
Figure 314BM

Note: This Figure is for the design of a reinforced concrete deck on existing steel beams/girders using MS18.

Slab Thickness "T" (mm)

Area of Steel "As" (Sqw. mm per m)

Effective Span "S" (m)

As, Top of slab
(d = T - 91 mm)

As, Bottom of slab
(d = T - 48 mm)
Sample Problem: Using load factor design procedures, determine whether the reinforcing steel design given in the previous example is adequate to sustain a 915 mm cantilever slab carrying a 915 mm deflector parapet and an MS 22.5 loading.

\[ P_1 = 89.0 \text{ KN (3.243)} \]
\[ P_2 = 44.5 \text{ KN (2.71.3)} \]

**Truck Load Distribution Factor:**
\[ E_1 = 0.8 \times X_1 + 1.140 \text{ (3.245,1,1)} \]
\[ E_1 = 0.8 \times 0.15 + 1.140 = 1.260 \]

**Railing Load Distribution Factor:**
\[ E_2 = 0.8 \times X_2 + 1.524 \text{ (3.245,2)} \]
\[ E_2 = 0.8 \times 0.765 + 1.524 = 2.136 \]

**Uniform Dead Load:** (per meter of length)
\[ \text{Slab} = \omega_1 = (0.255)(1.0)(23.57) = 6.01 \text{ KN/m} \]
\[ \text{F.W.S.} = \omega_2 = (2.87)(1.0) = 2.87 \text{ KN/m} \]

**Concentrated Dead Load:** (per meter of length)
\[ \text{Parapet} = P = 6.84 \text{ KN (located @ CG)} \]

**Dead Load Moment:**
\[ \text{DLM} = \frac{1}{2} \omega_1 l^2 + \frac{1}{6} \omega_2 (L - 0.458)^2 + P(L - 0.15) \]
\[ \text{DLM} = \frac{1}{2} (6.01)(0.915)^2 + \frac{1}{6} (2.87)(0.915 - 0.458)^2 + 6.84(0.915 - 0.15) = 8.05 \text{ KN-m} \]

**Live Load Moment:**
\[ \text{Truck Load Moment} + \text{Impact} = \text{TLM} + I = 1.3 \times X_1 \left( \frac{P_1}{E_1} \right) \]
\[ \text{TLM} + I = 1.3 \times 0.15 \times \frac{89}{1.26} = 13.77 \text{ KN-m} \]
\[ \text{Railing Load Moment} = \text{RLM} = h \left( \frac{P_2}{E_2} \right) \]
\[ \text{RLM} = 0.915 \times \frac{44.5}{2.136} = 19.06 \text{ KN-m} \]

**Design Moments:**
\[ M_u = 1.3 \ [ \text{DLM} + 1.67 \text{ LLM} ] \]
\[ M_u = 1.3 \ [ 8.05 + 1.67 (19.06) ] = 51.84 \text{ KN-m} \]
\[ M_w = \text{Service Load Moment} = \text{DLM} + \text{LLM} \]
\[ M_w = 8.05 + 19.06 = 27.11 \text{ KN-m} \]

**CHECK TOP REINFORCEMENT**

**AASHTO**

*Figure 315M*
$f'_c = 31 \text{ MPa}$

$\phi = 0.9 \quad (8.16.1.2.2)$

$Z = 23000 \text{ KN/m (top steel)} \quad (8.16.8.4)$

$R = M_u / \phi b d^2 = 51.84(1000) / 0.9 (1) (169)^2 = 2.017 \text{ MPa}$

$p = 0.00500 \quad (\text{see fig. 312M})$

$A_s = \rho b d = 0.00500 (1000) (169) = 845.54 \text{ mm}^2 / \text{m} \quad < 1667 \text{ mm}^2 / \text{m} \quad \star \text{ ok}$

Check Steel Spacing (8.16.8.4)

$f_s (\text{ALL}) = \frac{Z}{(d_c A)^{1/3}} < 0.6 f_y$

$\frac{23000}{((58)(13920))^{1/3}} = 247.0 \text{ MPa}$

$\star$ - Steel reinforcing ratio for top steel taken from Transverse Slab Design example (16M bars @ 120 mm).
TYPICAL CONCRETE

DECK HAUNCH DETAIL

Figure 317M
TYPICAL CONCRETE DECK HAUNCH DETAIL

Figure 318M
PLAN VIEW

Crossframes for
Dog-legged Splices

Figure 319M
FIT-UP SHOULD NOT BE INCLUDED IN ESTABLISHING THE NUMBER OF BEAMS.

IN ACTUAL CONSTRUCTION FIT-UP WILL BE ABSORBED IN THE OVERHANG

DO = DESIGN OVERHANG (MINIMUM 50 mm, MAXIMUM 200 mm)  BOX BEAM DESIGN WIDTH SHOULD BE SELECTED SO DO STAYS WITHIN ACCEPTABLE RANGE.
Sealing of Concrete Surfaces, Substructure

Figure 321M
Seal entire surface area of column

Seal end of pier cap

Ground Line

SECTION A-A

ELEVATION

PIER SEALING LIMITS
(EXPOSED TO DEICER SPRAY)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 322M
ABUTMENT SEALING LIMITS
(FOR INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEAL END OF WINGWALL ABOVE GROUND LINE.

CROSSHATCHED AREA REPRESENTS THE CONCRETE SEALING LIMITS.

GROUND LINE

SEAL ENTIRE SURFACE AREA

ABUTMENT SEALING LIMITS
(FOR SEMI-INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEAL END OF WINGWALL ABOVE GROUND LINE.

CROSSHATCHED AREA REPRESENTS THE CONCRETE SEALING LIMITS.

GROUND LINE

SEAL ENTIRE SURFACE AREA

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 323M
INTEGRAL ABUTMENT

CONCRETE DECK SLAB

APPROACH SLAB

CONSTR. JOINT

STEEL STRINGER

NEOPRENE WATERPROOFING

CONSTR. JOINT

POROUS BACKFILL WITH FABRIC

SLOPE PROTECTION

Ø PILES-PLACE PILE WEB PARALLEL TO Ø BEARING.

Figure 324M
Figure 325M
BRIDGE WITH SIDEWALK - VERTICAL FENCE

305mm-1092mm max. overhang

300mm min. horizontal distance between face of curb and edge of fence.

1524mm

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 326M
BRIDGE WITH SIDEWALK - VERTICAL FENCE

300mm min. horizontal distance between face of curb and edge of fence.

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 327M
PEDESTRIAN FENCING ON STRUCTURES

DEFLECTOR PARAPET WITH FENCING

Figure 328M
SECTION 400 – REHABILITATION & REPAIR

401 GENERAL

The technology of bridge rehabilitation and repair is constantly changing. In addition, many of the defects encountered vary from bridge to bridge requiring individual unique solutions. Consequently, this section of the Manual merely presents an overview of bridge rehabilitation and some of the more common types of repairs. The repairs that are discussed are all proven to be reasonably successful and are approved by FHWA for use on federally funded projects. ODOT’s District maintenance teams and the Office of Structural Engineering are continually experimenting with new techniques, many of which appear promising, but have not yet reached a point where conclusions can be drawn with regard to their longevity. Until these products and procedures are evaluated, they will not be included in this Manual and they should not be used on Federal aid projects.

401.1 DESIGN CONSIDERATIONS

For individual members, it will be necessary to determine whether the best option is to repair or replace. In making this decision, cost shall be considered along with factors such as traffic maintenance, convenience to the public, longevity of the structure, whether the rehab is long term or short term, and the practicality of either option.

Due to the variation in the types of problems encountered, the designer shall perform an in depth inspection of the structure to identify the defects that exist, and develop a solution which is unique to the problems found. This field inspection should include color photographs and sketches showing pertinent details and field verified dimensions.

It is imperative that an in depth, hands on, inspection of bridges be made, by the Design Agency preparing the repair or rehab plans, to determine the extent of structural steel and concrete repairs. This inspection shall be made concurrent with plan development. Large quantity and cost overruns result when this inspection is not adequately performed resulting in substantial delays to completion of the project.

All pertinent dimensions that can be physically seen shall be field verified or field measured by the designer and incorporated into the plans. It is not permissible to take dimensions directly from old plans without checking them in the field because deviations from plans are common. Every attempt shall be made to prepare plans that reflect the actual conditions in the field. However, it is recognized that uncertainties may exist. Consequently, the note entitled “EXISTING STRUCTURE VERIFICATION”, found in Section 600 of this Manual, should be included in the plans with the understanding that the designer is still responsible for making a conscientious effort to provide accurate information based on field observations.

Sketches of various details have been provided throughout this chapter. These sketches are not
complete nor are they to be taken as standard details. They are offered as suggestions or ideas for the designer to use in developing his or her own solutions to the unique problems they encounter.

A bibliography has been included at the end of this section. While these references contain much information and many innovative ideas, designers are advised to discuss untested solutions with the Office of Structural Engineering before completing detail plans.

401.2 STRENGTH ANALYSIS

When analyzing existing superstructures, substructures and foundations for strength, the live load is to be the HS20-44 [MS18] truck (or lane load) and the Alternate Military Loading as defined in Section 3.7 of AASHTO.

In analyzing the strength of existing superstructures, substructures and foundations for bridges that are to receive a new deck, a future wearing surface of 60 psf [2.87 kPa] shall be included in the dead load.

402 STRUCTURAL STEEL

402.1 DAMAGE OR SECTION LOSS

It may be necessary to repair a section of a steel member that has been damaged by rust or other means. Welded repairs are not permitted in tension zones. Damaged sections in tension zones normally shall be repaired by bolting new steel to existing steel. The specifics of the details are left to the ingenuity of the designer due to the vast number of possible solutions. If it is absolutely necessary to perform welded repairs in a tension zone, consult the Office of Structural Engineering for recommendations. The designer will be responsible for describing the welding procedures, non-destructive testing (NDT) requirements, etc. in plan notes.

Welding is permitted in compression zones provided the designer ensures that the chemistry of the existing steel is such that it can be welded. This will require either review of old mill certifications or actual sampling of the material for chemical analysis. The designer shall make this determination. Pay close attention to American Welding Society (AWS) Specifications. Field NDT of the welds will be required and it will be necessary to specify the type and location of the NDT in the plans.

402.2 FATIGUE

402.2.1 GENERAL

Fatigue damage, as it pertains to bridges, is typically categorized as due to either load-induced or distortion-induced (displacement-induced) stresses. While both types of damage are actually load-induced, the former is directly related to the application of live load and the resulting
stresses (in plane), while the latter is the result of secondary stresses (out of plane) typically transmitted by a secondary member as it tends to change the shape of, or distort, the primary member because of displacement. Load-induced damage is dependent on stress range, type of detail and the number of applications of live load and can be accounted for during design. Distortion-induced damage is not directly quantified in the design of a bridge but can be minimized through proper detailing. Two conditions are necessary for distortion-induced damage to occur: a periodic out-of-plane force or displacement; and an abrupt local change in stiffness where the force/displacement is applied.

Only load-induced fatigue is directly addressed by this section. Since the repairs for distortion-induced fatigue damage are specifically dependent upon the existing detail itself, the repair details are largely up to the designer; are beyond the scope of this section; and may require a special analysis. The designer should consult with the Office of Structural Engineering when developing these types of repairs.

If cracks can be visually detected, the vast portion of the fatigue life has been exhausted and retrofitting of the detail may be necessary.

When Ohio began using the concept of continuous spans, one commonly used standard detail has now become a fatigue concern. The detail consisted of simple span rolled beams butt welded together at the piers with short plates welded to the top and bottom flanges, roughly centered on the piers. The weld line of the beam webs had coped holes at the top and bottom flange, serving as access holes for the flange welds. The plates are not structural in that they are not serving the part of moment plates but were termed “splice plates” and were meant to reinforce the welded flanges, using a “belt and suspenders” logic. These details are illustrated on retired Standard Bridge Drawing SD-1-63, sheet 1, and were generally used with the rolled beams shown in retired Standard Drawing CSB-1-63. When beams of differing depths were used, the web of the shorter beam was cut horizontally and its flange was raised to match the depth of the adjacent beam and the gap in the web was then filled with weld. This method of joining typically resulted in the top splice plate being kinked or bent at the center of the pier. Henceforth, this detail will be referred to as the “Beam Continuity Weld” throughout Section 402.

FATIGUE EVALUATION

A fatigue evaluation of existing steel members/details to be re-used or rehabilitated is required. The evaluation consists of screening the member/detail and performing a remaining life analysis when necessary.

A fatigue evaluation submission shall be made to the Department, as part of the STRUCTURE TYPE STUDY, for final determination as to whether the members require fatigue related upgrading.
402.2.3 DETAILS TO CONSIDER

Evaluate longitudinal rolled steel beams/welded steel girders having either positive moment or negative moment welded coverplates, transverse floor beams with welded coverplates, and/or Beam Continuity Welds with and without splice plates.

If the details of concern have experienced severe corrosion, more than two heat-straightenings, mechanical damage, previously repaired fatigue damage or are wrought iron instead of steel, a special analysis is necessary and is not covered in this section. Contact the Office of Structural Engineering for guidance.

Do not evaluate base metal of rolled beams, welded plate girders without coverplates, cross-frames, cross-frame connections to beams/girders, transverse web stiffeners, longitudinal web stiffeners, lateral bracing, uncracked web welds, riveted members and riveted connections except if required by the Scope of Services.

A special analysis is necessary to evaluate welded truss members, box girders, curved beams, welded steel pier caps, dog-legged rolled beams, details/members with cracks and load carrying diaphragms of curved structures. Contact the Office of Structural Engineering for guidance.

402.2.4 EVALUATION PROCESS

Use the following retrofitting policy when evaluating the re-use of the existing steel beams/girders for deck replacement projects:

402.2.4.1 COVERPLATES AND BEAM CONTINUITY WELDS WITH SPLICE PLATES

A. All ends of coverplate/splice plate locations shall be retrofitted on bridges, without performing remaining fatigue life calculations, when:

1. The bridge is known to contain fatigue cracks at these locations.

2. The bridge is located on the interstate (urban and rural), rural principal arterial - other, and urban principal arterial - other freeway/expressway routes [functional classifications 1, 2, 11 and 12 respectively]. Information regarding functional classification can be obtained from the Office of Urban and Corridor Planning website.

3. The bridge is located on a route that is identified as a route carrying unusually heavy truck loads (i.e., quarry, mining, logging, landfills, heavy manufacturing, etc.). The District shall identify these routes at the time of the pre-scope.

An economic analysis shall be completed comparing the cost of utilizing the existing steel members with all associated repairs, retrofitting, strengthening, painting and widening issues.
considered, versus superstructure replacement to justify the decision to retrofit. For this analysis, approximately $ 8.00 per pound (in 2005 dollars) may be used for the estimated cost of “Structural Steel Repair”. For an estimation of the required weight of steel required for the retrofit, the plate sizes from the retired Bolted Splice Standard Drawing (Standard Bridge Drawing BS-1-93, sheet 2 of 3) may be used with the resulting weight increased by 25 percent if the web is not to be retrofitted and with no increase if the web is to be retrofit. When using the standard drawing, no deduction for bolt holes is necessary. The cost estimate should be based on initial cost.

B. Consider retrofitting the ends of coverplate/splice plate locations of a bridge, or parallel bridges (left-right) that share a continuous deck, that has a one way present day ADTT (Average Daily Truck Traffic) exceeding 1000 trucks. For bridges meeting this criterion, a fatigue life analysis shall be completed to calculate the remaining fatigue life using the procedures shown herein.

1. When the remaining fatigue life is equal to or exceeds 50 years, retrofitting is not required.

2. When the remaining fatigue life of any one location is less than 50 years, both ends of the plate at that location shall be retrofitted. Only retrofit those locations with insufficient remaining fatigue life. An economic analysis shall be completed as detailed earlier.

C. Bridges not meeting criteria (A) or (B) do not require a fatigue analysis. No retrofit is required.

When retrofitting the ends of splice plates that are used in conjunction with the Beam Continuity Weld, the top splice plate shall also be retrofitted at the location of the flange splice weld.

Retrofit web welds only when cracked.

On deck repair projects, including overlays and railing upgrades, retrofitting is not required unless known fatigue cracks exist.

**402.2.4.2 BEAM CONTINUITY WELD WITHOUT SPLICE PLATES**

The flange and web welds are to be retrofitted only if cracks can be visually detected or if there is physical evidence of cracking such as rust stains, damaged paint, etc. In addition to their own field review of the structure as required by Section 401.1 of this Manual, the Designer shall also review the bridge inspection records (available from the District) to ascertain if cracking was noted during an inspection.

The locations that are to be retrofit are only those where the welds are cracked, or show signs of cracking.
402.2.5  CLASSIFICATION OF DETAILS

The end of the coverplates and splice plates and the region of a splice plate directly above the Beam Continuity Weld shall be classified as either an E or an E’ detail. In order to qualify as a Category E detail, all of the following criteria must be met otherwise it is to be classified as a Category E’ detail:

A. The thickness of the coverplate must be less than 1.0 inch.  
B. The thickness of the top flange of rolled beam must be less than or equal to 0.8 inch.  
C. The coverplate is narrower than the beam flange (with or without welds across the ends of the plate) or the coverplate is wider than the beam flange with welds across the ends of the plate.

The Beam Continuity Welds are typically classified as a Category B detail. However, due to the typical poor weld quality and the lack of testing, for the purpose of a remaining life analysis, these details shall be evaluated based upon the requirements of a Category C detail.

For the purpose of a remaining life analysis, riveted built-up sections, riveted members and riveted connections shall be evaluated based upon the requirements of a Category C detail.

All other details shall be classified according to AASHTO Standard Specification Table 10.3.1B.

402.2.6  REMAINING LIFE ANALYSIS

The remaining life is to be calculated by dividing the total fatigue life into two periods in which the truck volume and fatigue truck weight remain constant over the individual time period as detailed in AASHTO’S “Guide Specification for Fatigue Evaluation of Existing Steel Bridges”, 1990 and all Interims, Section 3.2, Alternate 1:

A. A past period from the opening of the bridge to the present, Yp, based on the existing configuration of the beam,

B. A future period from the present to the end of the fatigue life, Yf, based on the proposed configuration of the beam.

Use MEAN LIFE for redundant main members and SAFE LIFE for non-redundant members which, for the purposes of the remaining fatigue life analysis only, are defined as floor beams and superstructures with three or less main longitudinal load carrying members.

The following modifications are to be made to the procedures outlined in the Guide Specification for the calculation of the remaining life:

A. IMPACT (Guide Specification section 2.4)
The gross weight of the fatigue truck is to be increased by 15 percent to account for impact, unless specific site conditions would require an increase up to a maximum of 30 percent. Examples of site conditions would include a severe change in longitudinal grade either directly on or coming onto the bridge or an unusual bunching of trucks.

B. MEMBER SECTION PROPERTIES (Guide Specification section 2.7)

No increase in section properties is to be used to account for any unanticipated possible composite action except when the deck has/is/will be mechanically connected to the beams by means of shear connectors or other structural shapes and meet the reinforcing requirements of AASHTO Standard Specification section 10.50.2.3. Rivets and bolts used in flange splices/connections are not considered a mechanical connection.

C. AVERAGE DAILY TRUCK VOLUME ESTIMATE (Guide Specification section 3.5)

1. Present Truck Volume, \( T_P \)

   The average daily truck volume for the past time period is to be estimated using the current ADTT. If the ADTT in one direction is not available, multiply the total ADTT by 0.55 and use the resulting value.

2. Future Truck Volume, \( T_N \)

   The average daily truck volume for the future time period is to be estimated using the design ADTT (“20 years hence”), multiplied by a factor of 1.85. If the ADTT in one direction is not available, multiply the total ADTT by 0.55 and use the resulting value.

D. Fatigue Truck weight ratios, \( W_P/W \) and \( W_N/W \), shall be taken as unity (1.0).

402.2.7 WIDENING PROJECTS

If an existing fascia beam is to be used as an interior beam on the new widened deck, the fatigue life analysis is to reflect this (i.e. the estimate of the existing damage for the past time period is to be based on the beam being a fascia beam and the estimate of the future damage for the future time period is to be based on the beam being used as an interior beam). Appropriate distribution factors are to be used.

402.2.8 PERMIT LOADS

In addition to the Interstates, the following routes have a large number of permit loads: US 23 in Franklin, Delaware, Marion, Wyandot, Wood and Lucas counties; US 30 in Van Wert, Putnam and Allen counties; and US 224 in Hancock, Seneca, Huron, Ashland and Medina counties.
In order to account for permit loads, if the bridge under consideration is located on any of the routes listed, the ADTT (present and future) is to be increased by 15 percent prior to performing the evaluation and calculating the remaining life.

**402.2.9 VERTICAL CLEARANCES**

The retrofit of a bottom flange could potentially reduce the amount of vertical clearance provided below an overhead structure due to the plate or the bolt head and needs to be checked by the designer. If the clearance is reduced and is unacceptable to the District, a modified retrofit will be necessary. The retrofit consists of a combination of a partial bolted plate along with a mechanical treatment of the existing weld. Contact the Office of Structural Engineering for further guidance in developing the details.

**402.2.10 FATIGUE SUBMISSION INFORMATION**

The fatigue submission shall include the following as a minimum:

A. Recommendations from the consultant regarding whether or not the details require retrofitting and are suitable for retrofitting.

B. Recommendations from the consultant regarding any necessary strengthening of members to meet loading requirements.

C. A summary table showing the following at each detail and location being evaluated: remaining fatigue life; moments; stress ranges; fatigue detail category; live load distribution factor; ADTT, whether ADTT is for one way or total traffic; directional distribution factor; present age of structure in years; impact percentage.

Also include the detailed calculations showing the remaining life computations and the economic analysis.

DO NOT submit voluminous pages of meaningless computer output. Provide the information in summary table form only.

**402.2.11 SPECIAL ANALYSIS**

For structures where the fatigue evaluation procedure outlined in this section, or portions of it, does not apply, a “special analysis” may be necessary. The “special analysis” might consist of: an evaluation procedure based upon specific research findings (e.g. corrosion); the solving of a two-dimensional or three dimension model, as discussed in Section 301.3 of this Manual, in order to determine distribution factors or member stresses or secondary stresses (distortion); a fracture mechanics based analysis; instrumentation to determine member stresses under normal traffic; or in extreme situations, a load test combined with instrumentation in order to determine
distribution factors and member stresses. Since each situation is unique, the Designer shall develop an appropriate approach in order to conduct a comprehensive fatigue evaluation while maintaining close coordination with the District and Office of Structural Engineering for guidance and input. The Designer shall submit the “FATIGUE EVALUATION PROCEDURE” to the District for review and approval. The Designer shall then conduct the evaluation accordingly.

402.3 FATIGUE RETROFIT

402.3.1 RETROFIT DESIGN

402.3.1.1 GENERAL

Details that have cracked or whose remaining fatigue life has been determined to be less than the desired service life are to be retrofitted. The retrofit is done by bolting splice plates at the end of the existing plates and/or over the Beam Continuity Weld in accordance with the procedures outlined in the following sections.

When computing the initial moments for composite steel members, use assumptions that will produce the greatest stiffness at the detail being evaluated (AASHTO Standard Specification 10.38.1.6).

402.3.1.2 END OF COVER PLATES AND SPLICE PLATES (BEAM FLANGES)

A. The design is to be in accordance with the AASHTO Standard Specification for Highway Bridges (current edition and all Interims) and the Bridge Design Manual except as noted in this section.

B. The flange splice plates are to be designed for the entire moment, both from dead load and live load at the section and any contribution from the web is to be discounted, except for when the web is also retrofitted.

C. The connection is to be designed as a slip-critical connection.

D. The live load truck shall be the standard HS20 truck (or lane loading as appropriate) as shown in Figure 3.7.7A (3.7.6B) of the AASHTO Standard Specifications for Highway Bridges. A future wearing surface of 60 pounds per square foot is to be included. Standard AASHTO distribution and impact factors are to be used. The loads are to be calculated at the location of the detail.

E. The stress range in the flange, and in the splice plates, must not exceed the allowable stress range for a Category B detail with over 2,000,000 cycles.

F. The bolted splice designs shown on the retired Standard Bridge Drawing, BS-1-93, are not to be used directly for the design but may be used as a preliminary guide in the initial sizing of
the plate and determination of the number of bolts.

G. For coverplates with tapered ends, the number of bolts required for design shall be placed before the beginning of the taper and beyond the end of the taper. Place additional seal bolts in the range of the coverplate taper.

H. Filler plates equal to the thickness of the coverplate/splice plate shall be used and detailed in the plans, as required.

I. If the flange is cracked and the crack extends into the beam web, a properly installed high strength bolt shall be placed in a hole drilled through the web to eliminate the crack tip. If the web crack is longer than one sixth of the beam depth, the web shall be spliced. The plans shall detail the drilling of the hole and the installation of the bolt.

J. Investigate if vertical clearances are affected when retrofitting bottom flange plates.

K. This design procedure applies equally to coverplate and splice plate ends on the bottom and top flanges.

L. When plating the top flange splice plate for the Beam Continuity Weld detail, the designer may be tempted to use one long continuous plate in lieu of three smaller individual plates. However, because of the possible bend in the splice plate, bolting the new plate into position would be difficult. A single plate could be locally yielded when forced into position and the resulting connection would most likely not be slip-critical. Three individual splice plates shall be used.

402.3.1.3 BEAM WEBS

A. The design is to be in accordance with the Standard Specification for Highway Bridges and the Bridge Design Manual except as noted in this section.

B. When the web retrofit is located under an end of a coverplate retrofit, the web splice plates are to be designed for the applied shear, from both dead load and live load. All of the moment is assumed to be taken by the flange. The plates are to be centered around the ends of the coverplate.

If the web retrofit is located at a Beam Continuity Weld, such as over a pier, and the beam flange is plated, the web splice plates are to be designed for the entire shear, the portion of the moment that is carried by the web and the moment caused by the eccentricity of the bolts.

C. The connection is to be designed as a slip-critical connection.

D. The live load truck shall be the standard HS20 truck (or lane loading as appropriate) as shown in Figure 3.7.7A (3.7.6B) of the AASHTO Standard Specifications for Highway Bridges. A future wearing surface of 60 pounds per square foot is to be included. Standard
AASHTO distribution and impact factors are to be used. The loads are to be calculated at the location of the detail.

E. The stress range in the retrofitted web, and in the splice plates, must not exceed the allowable stress range for a Category B detail with over 2,000,000 cycles.

F. The bolted splice designs shown on the retired Standard Bridge Drawing, BS-1-93, are not to be used directly for the design but may be used as a preliminary guide in the initial sizing of the plate and determination of the number of bolts.

G. Filler plates equal to the difference in thickness of the beam webs shall be used and detailed in the plans, as required.

H. The crack is to be arrested by drilling a hole at the crack tip.

402.3.1.4  FILLER PLATES

Filler plates shall be considered underdeveloped and their effects shall be considered in the design of the fasteners (AASHTO Standard Specification 10.18.1.2). The design reduction factor shall be applied to the fasteners on both sides of the connection to ensure a symmetrical splice. Filler plates need not extend beyond the plates.

The minimum thickness for a filler plate shall be 1/8 inch, which inherently means that when the difference in thickness of the two sections being plated is less than 1/8 inch, no filler plate is necessary.

When plating the top flange splice plate for a Beam Continuity Weld detail, the designer will need to determine the thickness of the filler plate by examining the details shown on sheet 1 of retired Standard Bridge Drawing SD-1-63. A copy of the drawing can be obtained by contacting the Bridge Standards Engineer in the Office of Structural Engineering.

Filler plates shall be detailed in the plans.

402.3.1.5  FATIGUE CRACKS IN OTHER MEMBERS AND DETAILS

Since the repair of fatigue cracks in members and details is dependent on the specifics of a situation, it is not practical to provide specific guidelines or repair details, so only a general discussion will be made. In order to properly devise a repair scheme, it must be determined if the crack resulted from load-induced or distortion-induced stresses. The source of the cracking should also be identified. More than one factor could contribute to cracking. Defects from poor fabrication may need to be considered. Cracking can also occur during shipment and/or erection. The retrofit may need to include modifying the stiffness of an existing connection. The repair should be logically thought out and its influence on the other steel members investigated.
poorly designed repair may actually worsen the situation as repaired cracks can reinitiate and propagate further into the member, the repair detail itself may experience cracking and new cracking may occur elsewhere if load patterns were modified or member response to loads has been modified.

402.3.2 BOX GIRDER PIER CAPS

Often box girders were constructed using non-continuous back-up bars that were stitch welded in place. The discontinuity in the back-up bar is of major concern since it acts like a crack in the member and is the source of crack propagation into the flange or web. One possible solution is to drill a horizontal hole through the web and back-up bar at the points of discontinuity. See Figure 401 for a sample detail. The stitch welds may or may not be a problem depending on the stress ranges at their location.

402.3.3 MISCELLANEOUS FATIGUE RETROFITS

Various retrofits have been used for fatigue prone details such as small web gaps which result in stress concentration and subsequent cracking, intersecting welds, lateral connection plates, longitudinal stiffeners, cracks and many others. The Office of Structural Engineering may be contacted for assistance in determining the best retrofit for specific details.

402.4 STRENGTHENING OF STRUCTURAL STEEL MEMBERS

Welded stud shear connectors shall be installed full length on all steel beam or girder bridges in which the deck is being removed and replaced. The stud spacing shall be designed in accordance with AASHTO Section 10.38.5.

Bolted cover plates in tension zones or field welded cover plates in compression zones can be used to increase strength. Field welding is to be performed in strict compliance with AWS Specifications. Field NDT of the welds will be required and it will be necessary to specify the type and location of testing in the plans. Also, practicality of field welding shall be evaluated. Overhead welding is not practical.

Consider jacking the stringers to relieve stresses prior to installing cover plates. In this manner the cover plates will carry dead load and live load stresses. If the plates are installed without relieving the stresses, they will carry live load only. This is merely a suggestion as to how extra strength might be obtained if it is needed.

Other methods of increasing the strength are to attach angles or structural shapes to the web or flanges. The possibilities are numerous and shall be left to the ingenuity of the designer. However, the designer shall remember to pay strict attention to practicality as well as strength and fatigue requirements. Consult the Office of Structural Engineering for recommendations or to review unusual details with the Office of Structural Engineering before proceeding.

When retrofitting or repairing truss members, the designer shall remember to provide for
temporary support where needed. Many truss members are non-redundant, and their removal could result in the collapse of the structure.

402.5 TRIMMING BEAM ENDS

Trimming of beam ends is sometimes necessary due to tilting of the abutment and closure of the end dam. A detail (plan and elevation view) showing where the beam is to be cut is required. Also provide pertinent notes and include the work with Item 513 “Trimming of Beam Ends” for payment. Pay attention to the clearance to the end cross frames and detail their removal and replacement if necessary.

In lieu of trimming the beam ends, consider modifying the backwall if backwall removal and replacement is being performed as part of the work. Modifying the backwall would be a viable option if it were necessary to remove and replace the end cross frames as a result of trimming the beam ends. Another option to consider is converting the existing abutment into a semi-integral abutment as discussed in Section 406.

402.6 HEAT STRAIGHTENING

Beams or girders that have been struck by trucks or bent by other causes can often be repaired by heat straightening only, or in combination with field welding to install new sections for the damaged steel member portions. The District Bridge Engineer or other ODOT representative with experience in heat straightening should assess the practicality of this type of repair before proceeding. If heat straightening is deemed to be practical, a proposal note is available which describes and controls the operation. Plan requirements are to provide a pay quantity and a detail showing the location of the repair.

402.7 HINGE ASSEMBLIES

Consideration should be given to removing hinges and making the members continuous.

If the hinge cannot be removed, consideration should be given to the need of providing redundancy in the event of a hinge failure. Figures 402 and 403 show a method for consideration.

If a pin and hanger assembly is to be rehabilitated, consult the Office of Structural Engineering for recommendations, including suggested lubrication and nondestructive testing requirements.

402.8 BOLTS

Bolts should conform in general to Section 300 of this Manual. However, oversize or slotted holes, designed in accordance with AASHTO Section 10.24, are permitted in repair or
rehabilitation work. Oversize or slotted holes may be desirable in some remedial applications especially where the fit of repair or replacement members or parts becomes tedious. These connections shall still be designed as slip critical.

The plans should state the specific requirements for the holes and necessary washers since all CMS requirements are for standard size holes.

The use of breakaway fasteners, such as Huck bolts, is acceptable when clearance problems arise. Remember that two or more bolt manufacturers shall be specified in order to satisfy FHWA requirements. Also, bear in mind that the installation specifications in the CMS deal strictly with the installation and testing of normal high strength bolts. If Huck or other breakaway fasteners are used, the installation specifications shall be modified by plan note.

403  CONCRETE REPAIR/RESTORATION (OTHER THAN DECK REPAIR)

403.1  GENERAL

Repairing concrete that is more than superficially damaged is expensive and problematic. Since many members can be completely replaced for less than the cost of extensive repair, aggressive replacement of deteriorated members should be pursued. Salvaging concrete containing corroding reinforcing steel or critically saturated aggregate does not often result in a long lasting component since the substrate concrete repaired is only marginally better than the unsound concrete removed. Any time there are major and extensive repairs being proposed to concrete structures, in depth and thorough investigation of the condition of the concrete will be required. This investigation shall include, but is not limited to, hand investigation with a chipping hammer, drilling into unsound concrete to determine the depth of deterioration, and concrete cores. In the past, the extent of concrete deterioration actually encountered in the field has far exceeded the amount anticipated in the design stage on certain projects.

403.2  PATCHING

It is the designer's responsibility to evaluate the repair areas and determine the most suitable repair method.

To serve as a guide to the designer, the following criteria have been established to help in the patching selection evaluation.

Item 519, Patching Concrete Structures, As Per Plan, should be used where the repair depth is 3 inches [75 mm] or greater and the surface can be readily formed and concrete placed. This type of patch is the most durable due to its depth and the utilization of reinforcing bars to tie it together. Where extensive curb repair is encountered, the patching should be paid for on a lineal foot [lineal meter] basis. This will require a pay item for: Item Special, Patching Concrete Structure, misc. A plan note will be required describing the work and tying it to CMS Item 519.
Item 520, Pneumatically Placed Mortar, should generally be used where the repair surface cannot be readily formed and concrete placed, where the depth of repair is between 1 and 6 inches [25 and 150 mm], and where at least 150 square feet [15 m²] of repair area is involved.

The detail plans shall show and detail the locations of the areas that require patching repairs. Additionally, Item 519 needs a plan note requiring the surfaces to be patched and the exposed reinforcing steel to be abrasively cleaned within 24 hours of application of patching material (or erection of forms if the forms would render the area inaccessible to blasting). See the note in Section 600 of this Manual.

Trowelable mortar should generally be specified when the repair depth is less than 1½ inches [40 mm] deep, and the repair area is less than 150 square feet [15 m²]. Trowelable mortar should also be specified in lieu of pneumatically placed mortar for the case where the depth of patch is equal to or less than 3 inches [75 mm] and the quantity is less than 150 square feet [15 m²]. 3 inches [75 mm] is the maximum depth of patch that should be attempted with this type of mortar.

A pay item, Item 843, Patching Concrete Structures with Trowelable Mortar, should be used and reference should be made to a Supplemental Specification 843.

The designer shall outline the areas to be repaired on the structure and also show where these areas are on details in the plans.

403.3 CRACK REPAIR

Cracks can be repaired by epoxy injection, C&MS 512.07. The location of the cracks shall be shown in the plans and marked in the field.

404 BRIDGE DECK REPAIR

404.1 OVERLAYS ON AN OVERLAY

In no case should a new asphalt or concrete overlay be placed over an already present overlay on a bridge deck. Removal of any existing overlay is required before a new overlay is placed.

404.2 OVERLAYS

The following types of overlays may be used in the repair of an existing reinforced concrete deck:

A. 1¼ inches [32 mm] minimum micro-silica modified concrete (MSC) per either Supplemental Specification 847 or 848. Micro-silica is state of the art and is recommended because it provides greater permeability resistance than the same thickness of other types of overlay materials.
B. 1¼ inches [32 mm] minimum latex modified concrete (LMC) per Supplemental Specification 847 or 848.

C. 1¾ inches [45 mm] minimum superplasticized dense concrete (SDC) per Supplemental Specification 847 or 848.

D. ¼ inch [6 mm] Epoxy Waterproofing Overlay for Bridge Decks are not normally recommended except for in the case where a concrete overlay would sufficiently lower the bridge’s load rating. A proposal note is available.

Be aware that the minimum overlay thicknesses indicated provide the maximum protection against chloride penetration. Increased thicknesses do not proportionally increase protection. Minimum thicknesses should be used if at all possible. The maximum thickness should be limited to 2½ inches [65 mm].

Overlays are not intended to be used for grade adjustments.

Overlays shall not be used on new decks.

A deck condition survey shall be performed in accordance with Section 412 of this Manual. This survey is required for all overlay projects in order to determine reasonably accurate variable thickness quantities.

Hydrodemolition is a recommended option for projects where uniform removal depth across an entire bridge deck is required. It is recommended that hydrodemolition be specified for any bridge overlay project where the total square yardage [square meters] of the bridge decks to be overlayed is 500 square yards [400 square meters] or greater. The normal depth of uniform removal of the original deck concrete called for shall be 1 inch [25 mm]. Plan removal depths should not be set up to go below the top mat of deck reinforcing.

404.3 UNDER DECK REPAIR

For under deck spalls up to 1 inch [25 mm] deep, use trowelable mortar (Supplemental Specification 843). For more severe underside deterioration, full depth repairs or Item 519 will be necessary. No spalls over traffic or other safety sensitive areas should be patched because potential debonding of the patch creates a hazard to the public. In these areas, remove loose concrete and provide a concrete sealer.

Low-pressure epoxy injection has also been tried as a remedy for delaminations detected in the bottom portion of the deck. However, there are no indications of how well this method of repair works and its usage should be scrutinized.
405 Bridge Deck Replacement

Notes similar to those found in Section 600 of this Manual should be provided for deck removal in order to prevent damage to existing steel stringers. Refer also to the applicable portions of Section 300 of this Manual.

Superelevated deck sections (existing and new) may need temporary modifications to the slope of the deck and/or shoulder in order to accommodate the traffic from the phase construction. The designer is to make this determination during the Structure Type Study and add additional details and/or notes as necessary. Structural members may require additional structural analysis to insure their adequacy and that no damage to the member will occur.

On all deck replacement projects, the elevations of the bottom of the beam shall be field determined so that when the deck is built to the new plan profile grade, it will be possible to obtain the required minimum deck thickness. Elevations shall be taken at the beam seats and in the interior portions of the spans. This is a design consideration and is not something that should be left for the contractor to deal with after a contract has been awarded.

405.1 Elimination of Longitudinal Deck Joint

For bridges up to 90 feet [27 meters] in width, consideration should be given to eliminating the longitudinal deck joint if one exists. However if the existing bearings are rockers and bolsters, they may need to be replaced with elastomeric bearings since the transverse movement due to temperature changes will be increased. Rockers and bolsters were designed to move in a longitudinal direction only.

An alternate to the cost of replacing all bearings in a structure is to increase the fit-up clearance or lateral play between the head of the rocker and its cap. One method (option) to consider is to provide a new rocker cap that has an increased transverse width that is able to accommodate the new increased anticipated transverse movements. This revision of the standard rocker bearing allows some additional lateral movement before the rocker head contacts the cap's welded side plate.

405.2 Deck Haunch

If possible, a 2 inch [50 mm] haunch depth should be provided over the stringers unless this haunch would cause undue problems with the profile grade off the bridge.

It is sometimes necessary to raise the profile grade of a structure. One way to accomplish this change when replacing the deck is by using deep haunches. The maximum recommended haunch depth is 12 inches [300 mm]. Provide reinforcing steel in any haunch greater than 5 inches [125 mm]. A deep haunch (5 inches [125 mm] or more) shall be made by providing a haunch similar to the one illustrated in the Figures portion of Section 300 with the horizontal haunch width...
limited to 9 inches [225 mm] on either side of the flange.

405.3 CLOSURE POUR

A closure pour may not be necessary for replacement of a deck on existing stringers even when using stage construction since differential deflections will be resisted by the existing cross frames.

If a deck replacement project also includes an integral or semi-integral retrofit at the abutments a closure pour may be required. New concrete abutment diaphragms without a closure pour at the stage line, will not allow the unloaded existing beams to freely deflect during the deck replacement pour.

When stage construction is used the single longitudinal construction joint shall be sealed with High Molecular Weight Methacrylate (HMWM) resin.

For additional information and requirements regarding closure pours, regardless of superstructure type, refer to Section 409.1.

405.4 CONCRETE PLACEMENT SEQUENCE

405.4.1 STANDARD BRIDGES

Placement sequences are not generally detailed for standard steel beam or girder bridges but are left to the contractor. However, the designer should recognize the need for a pour sequence is not limited to long structures with intermediate expansion devices. Other possible structure types are bridges with end spans less than 70 percent of internal spans and two span structures where uplift is a concern, structures whose size eliminates one continuous pour, etc.

405.4.2 STRUCTURES WITH INTERMEDIATE HINGES

Long multiple span steel beam and girder bridges have, in the past, been subdivided into units by means of intermediate expansion joints, located at points of contraflexure, in order to keep expansion and contraction within the capacities of bearing devices and expansion joints. The hinged structure is more sensitive to placement of deck slab concrete than a fully continuous structure. This sensitivity requires that the sequence of deck concrete placement be carefully planned because (1) the configuration of most intermediate joints makes them susceptible to damage if the deck placement sequence results in large angle changes between the articulated elements of the joints, and (2) the development of composite action in previously placed spans may cause deflections to vary from design deflections and result in a rough profile.

Plans should show the placement sequence, but should allow the contractor the option of a different sequence, subject to the approval of the Director. Generally, concrete should be placed
on the long cantilever before concrete is placed on the short cantilever, particularly before placement in the span contiguous with the short cantilever.

The most unsatisfactory sequence is to first place concrete in the span contiguous with the short cantilever, especially if concrete is first placed in half of the span immediately adjoining the short cantilever. This sequence produces the maximum angle change between the joint elements.

Refer also to Figure 404 for additional information.

Where controlled deck placement sequence alone will not provide adequate protection against damage to the joint, provision should be made for attaching part or all of the joint to the main structural elements after the major portion of the concrete is placed. Another alternative is to have a separate deck pour of approximately 36 inches [915 mm], or of a width necessary to accommodate the joint and its proper placement/installation and alignment, at the joint’s location to allow for installation of the joint after the rest of the deck has been placed.

406 EXPANSION JOINT RETROFIT

While it is desirable to seal the expansion joint of bridges, it is not desirable to demolish a functional expansion joint and possibly a backwall simply for the purpose of installing a seal. As long as a severe corrosion problem does not exist, additional coating will preserve the components exposed to the expansion joint discharge until the deck is replaced. However, it shall in fact be established that a severe problem does not exist if coating is the chosen course of action.

On overlay projects, when practical, a retrofit similar to that shown on Figures 405 and 406 can be used. The purpose of the steel bars in Figures 405 and 406 is to eliminate thin layers of concrete over the existing steel. These thin concrete layers would not adhere well to the steel and would break off in a short period of time. There have been some reports of problems with the field installation of the strip seals and with the physical operation of the seal. The designer will need to ensure that there are sufficient clearances for the installation and the proper operation of the seal. In lieu of using all field assembly and field welding, the designer has the option of using a combination of partial shop fabrication with the remainder of the expansion joint being assembled and welded in the field. An alternative is the use of the Polymer Modified Asphalt Expansion Joint System as discussed in Section 300 of this Manual. This joint is limited in movement to 1½ inches [40 mm].

Note that the designer will have to investigate the existing joint on the particular structure(s) and develop details for carrying the retrofit past the gutter line and into the sidewalk or parapet. Details shall show any existing concrete to be removed, how to attach new steel armor to existing steel, how to attach new steel to concrete, how to attach retainers, dimensions, any new concrete, reinforcing steel requirements, material requirements and coating requirements. In general views from the centerline of the joint looking toward the deck and the backwall, a plan view and section views are required. If the roadway width is being increased due to removal of a
safety curb and upgrading to the deflector shape, a detail of the horizontal extension of the end
dam steel shall be provided. Make sure that all items of work are described and included
somewhere for payment.

Many designers consider the detailing of these joints to be of secondary importance and merely a
nuisance. However improper detailing of these joints has frequently caused project delays and
caused numerous problems. The joints are important to the longevity of the structure or they
would not be included in the work. Designers shall take care to ensure that they are designed and
detailed in a professional manner.

On projects involving stage construction, joints in the seal armor shall be located and shown in
the plans. A complete penetration butt weld should be provided at the armor joints and a partial
penetration butt weld should be provided around the outer periphery of the abutting surfaces of
the retainer (not in the area in contact with the gland). The gland should be continuous and
installed in one piece. Consideration should be given to the means of performing this one-piece
installation.

On more extensive projects, where the deck is being replaced, consider using the semi-integral
design shown on Figures 407 and 408. There are many variations to this solution and Figures
407 and 408 are presented only as a general guide. This type of design can be used for bridges
whose foundations are stable and fixed (for example on two rows of piles). It is not to be used
when the foundation consists of a single row of piles. The semi-integral design is appropriate for
bridge expansion lengths up to 250 feet [80 meters] (400 feet [125 meters] total length assuming
2/3 movement in one direction). Additional considerations are that the geometry and layout of
the approach slab, wingwalls, curbs, sidewalks, utilities and transition parapets shall be
compatible with (not restrain) the anticipated longitudinal movement. For example approach
slabs would have to move independently of turned back wings since the superstructure and
approach slab move together. If the approach slab were connected to turned back wings in any
manner, then movement of the entire superstructure would be restricted. Also refer to Section
200 of this Manual for further discussion.

Type A pressure relief joints shall be specified when the approach roadway pavement is rigid
concrete and shall be placed at the end of the approach slab. See Section 200 of this Manual for
further discussion.

407 RAISING AND JACKING BRIDGES

Thought shall be given to any required jacking procedure and constraints. The bridge shall be
raised uniformly in a transverse direction in order to avoid inducing stresses into the super-
structure. Differential movement between stringers shall be limited to ¼ inch [6 mm]. Similarly,
consideration shall be given to the stresses induced into the structure by raising the bridge at one
substructure unit with respect to another. Limitations on the differential raising between units
may be necessary if stresses are found to be excessive.

4-12
When raising a structure, the adjustment in beam seat elevations shall be accomplished by steel shims if the amount raised is 4 inches [100 mm] or less. If the structure is raised more than 4 inches [100 mm], the bridge seat should be raised for its entire length by adding a reinforced concrete cap.

**BRIDGE DRAINAGE**

Much damage has occurred on bridges as a result of poorly designed drainage. The principles stated in Section 200 and 300, which cover drainage design for new structures, apply to rehab work also. Proper drainage is extremely important to the longevity of the structure. All dysfunctional drainage systems should be retrofitted. Consequently, the designer shall give adequate attention to the development and presentation of correct details for this important function.

If it is found that existing scuppers are not necessary, and the deck is not being replaced, they should be plugged. If the scuppers are plugged, the additional drainage directed off the bridge shall be collected.

If the deck is being replaced, the scuppers should be removed and the welds ground smooth.

Existing functional scuppers may need to be extended so that they are 8 inches [200 mm] below the bottom flange. Check to see if the bottoms are rusted through before preparing the scupper extension detail.

**WIDENING**

**409.1 CLOSURE POUR**

No single rule is applicable for closure pours on a widening project. The flexibility of each member, the overall theoretical deflection and use of integral or semi-integral abutments will cause each project to be unique. The purpose of the closure pour is to accommodate the differences in deflection that can occur between the new and the old during construction.

For widenings where the existing deck is removed and a new or wider, deck is being placed, with no superstructure members added, no closure pour is necessary. See Figure 409-A.

For widenings (2 beams or more) where either the existing deck is to remain or the phase line of a new deck will be between the existing and new superstructure, a closure pour should be provided. Cross frames (designated B1 in Figure 409-B) in the bay between the new and existing superstructure should not be welded until after the phase 1 and 2 new deck portions have been placed. After the cross frames have been welded, the closure section, phase 3, can be completed. Rebar splices should occur within the closure section. The width of the closure section should be at least 30 inches [800 mm]. See Figure 409-B.
For widenings (2 beams or more) where the deck’s phase line is not between the new and existing superstructure members, Figure 409-C, a closure pour will still be required. Existing cross frames (designated C1 in Figure 409-C) under the closure pour location need to be released before the phase 1 deck removal begins. Cross frames between new and existing members should be installed before the phase 2 pour. Re-install the released crossframes after the phase 2 pour but before the phase 3 pour. Rebar splices should occur within the closure section. The width of the closure section should be at least 30 inches [800 mm].

For widenings (2 beams or more) where a new deck is being constructed but the phase line is between existing superstructure members and at least 3 bays away from new member locations, Figure 409-D, a closure pour is still required. The procedure for crossframe release should be the same as defined in the paragraph above. The closure pour may be eliminated for this condition if the designer can show that the outside existing member, now being attached to the new member, is not restrained from returning to its original unloaded position by the new cross frames (designated D1 in Figure 409-D).

Closure pours may be eliminated if the differential deflection is expected to be less than ¼ inch [6 mm], regardless of superstructure type.

In special cases, the minimum closure section width may be reduced by the use of mechanical connectors as discussed in Sections 200 and 300 of this Manual. The designer should not blindly apply this exception since the use of lap splices is preferred and recommended.

Longitudinal construction joints should be treated with a high molecular weight methacrylate (HMWM) sealer. See the discussion in Section 300 for additional information.

Falsework for the new slab should be independent and not be tied to the original superstructure. This would not apply to falsework for the closure section. See Section 405.3 for closure pour requirements for deck replacements. The release of falsework for reinforced concrete slab superstructures may need to be coordinated (i.e. specified in the plans) between phases in certain situations.

Closure pours on bridge structures with integral or semi-integral abutments shall include the abutment’s diaphragm concrete. Any concrete pier diaphragm shall also be included in the closure pour.

409.2 SUPERSTRUCTURE DEFLECTIONS

The widened section should be designed so that superstructure deflections for the new and old portions are similar.
409.3 FOUNDATIONS

409.3.1 WIDENED STRUCTURES

Differential foundation settlements shall be considered. For example, if it is required to widen a bridge adjacent to an existing spread footing, it is possible that the existing foundation has settled as much as it is going to. However, if the widened portion is placed on a new spread footing, then that portion will settle with respect to the original and distress to the structure will result. Consequently, the new portion should be placed on piling or drilled shafts in an attempt to limit differential settlement.

409.3.2 SCOUR CONSIDERATIONS

Substructure foundations need to be investigated for scour. The investigation consists of determining what the substructures are founded on; how deep the foundation is; and a decision on whether potential scour will endanger the substructure’s integrity. Local scour and stream meander need to be considered.

409.4 CONCRETE SLAB BRIDGES

For single span slab bridges where stage construction is provided, all bridges should be screened to determine whether the main reinforcement is parallel to the centerline of the roadway or is perpendicular to the abutment.

Early standard drawings called for the main reinforcement to be placed perpendicular to the abutments when the skew angle became larger than a certain value. This angle was revised over the years as new standard drawings were introduced.

Concrete slab bridges should be screened according to the following criteria:

A. Prior to 1931 the slab bridge standard drawing required the main reinforcement to be placed perpendicular to the abutments when the skew angle was equal to or greater than 20 degrees. This angle was revised to 25 degrees in 1931, 30 degrees in 1933 and finally 35 degrees in 1946. The standard drawing in 1973 required the main reinforcement to be parallel with the centerline of roadway regardless of the skew angle.

B. If the skew angle of the bridge is equal to or greater than the angles listed above for the year built, a temporary longitudinal bent will have to be designed to support the slab where it is cut. For example a bridge built in 1938 with a 25 degree skew does not require a bent, however a bridge built in 1928 with a 25 degree skew does require a bent to be designed.

C. The deck should be inspected in the field to make a visual verification of the reinforcing steel direction.
409.5 PIER COLUMNS

New pier columns added for the purpose of widening shall be tied into the existing pier if the pier type is cap and column. Individual freestanding columns are not permitted.

When the existing piers are either T-type or wall-type piers, the designer should evaluate whether the new individual column should be tied back into the existing substructure unit or remain free standing. Two or more adjacent freestanding individual columns without a cap are not permitted.

410 RAILING

Railing not meeting current standards will require upgrading when that structure is included in a construction project as defined in Section 300 of this Manual.

The following sections are suggested methods for upgrading non-crash tested railing.

410.1 FACING

This method works when the existing parapet is in relatively good condition. The existing parapet and safety curb can be partially removed and a facing section placed on top as shown in Figure 410. Dowels should be at least 6 inches [150 mm] deep and should be spaced at no more than 15 inches [400 mm] c/c. Grout should be epoxy grout per CMS 705.20. It will be necessary to call for epoxy grout as other materials are also covered in these specifications. Details showing removal of existing concrete, dimensions for placement of new concrete, treatment of the parapet at the expansion joint (coordinate with details required and described under Expansion Joint Retrofit), parapet transition details, typical sections, joint spacing, reinforcing steel, limits for purpose of measurement and payment, and what pay item the work is to be included with are also required.

A typical note can be found in Section 600 of this Manual. Be aware that this note is general and shall be modified as needed for specific applications.

410.2 REMOVAL FLUSH WITH THE TOP OF THE DECK

If the outside of the existing parapet is in bad enough condition, the parapet and curb can be sawn off and a new parapet installed. Dowels should be at least 6 inches [150 mm] deep and should be spaced at no more than 15 inches [400 mm] c/c. The basis for this depth and spacing is research report FHWA-CA-TL-79-16 prepared by CalTrans in June of 1979 where they performed crash testing of various railing sections with shallow rebar anchorage. Grout should be epoxy per CMS 705.20. It will be necessary to call for epoxy grout as other materials are also covered in these specifications.
410.3 **THRIE BEAM RETROFIT**

If it is determined that upgrading of the parapet to the deflector shape is not prudent, there is a standard bridge drawing available that can be applied.

411 **BEARINGS**

Notes may be provided to rehabilitate the bearings if that is the chosen course of action. It is customary to split the jacking of the superstructure into one pay item (see Section 407 of this Manual) and the actual bearing restoration work into another. The contractor should be given the option of totally replacing the bearings with like bearings in lieu of rehabilitating the existing.

If elastomeric bearings are used at abutments where the existing expansion joint is built according to Standard Bridge Drawing SD-1-69, the joint shall be modified. The retired Standard Bridge Drawing SD-1-69 end dam consists of an angle, attached to the superstructure, which angle overlaps the abutment backwall. If compressible type bearings are used with this arrangement, the reaction will be transferred to the end dam angle causing distress to the end of the deck.

All bearings at an individual substructure unit shall be the same type. The bearings at any one substructure unit shall be compatible with all the bearings at the other substructure units.

When rehabilitation of existing bearings is being considered, the designer should make a cost comparison between rehabilitating the existing bearings and replacing them with all new bearings and any additional costs associated with modifying the existing structure to accommodate the new bearings. For new bearings, preference should be given to using elastomeric bearings. See Section 300 of this Manual for additional guidance. The cost comparison is to be submitted (included) as part of the Structure Type Study.

412 **CONCRETE BRIDGE DECK REPAIR QUANTITY ESTIMATING**

A deck condition survey shall be conducted and a report prepared for each existing concrete deck. The survey shall be performed as near as practicable to the plan preparation stage and shall be completed before beginning detail design work for the deck rehabilitation since it is to be used as a design tool toward that end. If the survey will be two winters or more old at the scheduled time of sale, a new survey shall be performed. The new survey will include recoring of the deck as deemed necessary.

The top surface of bare concrete decks shall be both visually inspected and sounded for obvious signs of deterioration. The top surface of decks with an asphalt overlay shall be visually inspected for signs of obvious and suspected deterioration.

The underside of all decks shall be inspected. Where there are indications of delamination, water
intrusion, discoloration, spalls, efflorescence or other signs of distress, the underside shall be sounded. The decks shall then be cored in suspicious areas to verify and further define areas of unsoundness. If it is suspected that full depth repair may be required, cores shall be taken full depth or at least to the bottom mat of reinforcing steel in those areas. A description of the core results shall accompany the deck condition survey report.

See Figure 411 for an example of the survey report form.

A sketched plan of the deck area, both top surface and underside, shall be included with the bridge deck condition survey. The unsound areas should be plotted on the sketch indicating the approximate dimensions that were used to estimate the percentage of total unsound deck area.

The minimum number of cores to be taken for a bare concrete deck shall be determined by the following criteria:

A. A minimum of two (2) per bridge for bridges with a deck area less than 2500 square feet [225 square meters].

B. A minimum of three (3) per bridge for bridges with a deck area between 2500 to 5000 square feet [225 to 450 square meters].

C. A minimum of four (4) per bridge for bridges with a deck area between 5000 to 10,000 square feet [450 to 900 square meters] with one additional core for each additional 10,000 square feet [900 square meters] or part thereof.

For bridge decks with an asphalt overlay the minimum number of cores listed above is required but it is further recommended that additional cores be taken due to the variability of unknowns hidden under the overlay.

Core locations shall be determined from conditions detected primarily from the bottom side of the deck; however, the top surface may also indicate areas to be cored. At least one core shall be taken from an apparently sound area and the others from questionable areas for comparison. The cores shall be submitted to the District Bridge Engineer with proper identification. They shall be retained for a minimum period of six months following the award of the actual construction contract.

Cores shall be inspected for:

A. Obvious crumbling
B. Stratification or delamination zones
C. Soundness of aggregate
D. Depth and condition of reinforcing steel

A description of the core results shall accompany the deck condition survey report.
An estimate of the unsound deck area as a percentage of total deck area shall be made from all of the information gathered from the survey and testing.

### 412.1 SPECIAL REQUIREMENTS FOR QUANTITY ESTIMATING FOR BRIDGES 500 FEET [150 m] OR GREATER IN LENGTH

In addition to the requirements of Section 412 above, additional requirements are added for structures with a length of greater than 500 feet [150 meters].

An electrical potential survey shall be performed. The area of active corrosion shall be compared with the delaminated area to determine a more accurate repair area.

Consideration should be given to engaging a company or agency specializing in bridge deck condition surveys that include thermographic acoustic and radar techniques, electromagnetic sounding and nuclear magnetic resonance.

Deck cores shall be analyzed for chloride content.

It should be noted that active corrosion is assumed to be taking place if a chloride ion content greater than 2.0 lbs. per cubic yard [1.2 kg/m³] is present and/or if there is an observed rebar electrical potential reading of greater than -0.35 volts compared to a copper-copper sulfate reference half cell.

It is not the intent to remove chloride contaminated or electrically active concrete but rather the results of the chloride ion content tests are to be used as a support tool for determining the type and extent of rehabilitation to be recommended.

### 412.2 ACTUAL QUANTITIES, ESTIMATING FACTORS

The following table gives estimating factors. The estimating factors are related on a sliding scale in order to project the quantities based upon measured areas to plan quantities 6 to 9 months beyond the actual date of the deck condition survey, including one winter.
The plan quantity shall be increased by a factor of 15% when the survey is one winter old.

Life cycle cost comparisons indicate that the benefits derived from replacement versus rehabilitation are approximately equal when the amount of unsound and delaminated concrete area is 50% to 60% of the total deck area. Therefore, unless there are overriding circumstances, the decks shall be replaced rather than rehabilitated when this area equals or exceeds 60% of the total deck area.

Do not include a pay item for full depth repair when such work is not indicated. The unit price established by this practice is worthless.

For any overlay project establishing accurate quantities are difficult. The difficulty is only increased if the bridge has an existing asphaltic or rigid concrete overlay. As asphaltic concrete overlays do not allow conventional sounding methods, additional coring and/or evaluation methods listed in Section 412.1 are recommended.

Required removal thicknesses of existing overlays should be established by coring of the deck to establish the true thickness of the existing overlay. Do not use the original design plans specified overlay thickness.

Variable thickness quantities should be established based on unsound areas of deck and assuming a depth to the bottom of the top layer of reinforcing steel + ¾ inch [19 mm]. Coring should be used to verify delamination depth.
Hand chipping bid items for overlay projects requiring hydrodemolition removal are associated with variable thickness quantities. Using 10% of the variable thickness surface area for quantities is one alternative, but other methods may be acceptable. Take note that this percentage is based on the variable thickness surface area and not the entire deck surface area. Another method would be to get local experience from the District Maintenance and Construction personnel as to what percentage would be best to use. A separate hand chipping bid item is not required if the method of removal is mechanical scarification as this is included in the overlay variable thickness quantity.

Accurate records of actual quantities shall be maintained for each bridge.

It is recommended that the Districts review any criteria for selecting rehabilitation and replacement projects with the Offices of Maintenance Administration and Structural Engineering to help assure statewide consistency on rehabilitation or replacement deck projects.

It should be noted that in all cases, maintaining the structural integrity of the structure is of prime importance. The effects of exposing large areas of the top mat of reinforcing in areas such as cantilevered parapets, negative moment reinforcing over beams on stringer bridges and over piers on continuous slab bridges and in other areas of a critical nature shall be clearly understood from a design standpoint.

**413 REFERENCES**

A. FHWA-RD-78-133, “Extending the Service Life of Existing Bridges by Increasing Their Load Carrying Capacity,” 1978


G. NCHRP Report 293, “Methods of Strengthening Existing Highway Bridges,” 1987

H. NCHRP Report 333, “Guidelines for Evaluating Corrosion Effects in Existing Steel...
Bridges,” 1990

I. Park, Sung H., “Bridge Rehabilitation and Replacement (Bridge Repair Practice),” S. H. Park, P.O. Box 7474, Trenton, N.J., 08628-0474, 1984
Procedure:
1. Drill 1/2" diameter hole through web and backing bar
2. Remove shaded area by grinding to radius of original hole.
   Final surfaces shall be smooth.
3. Perform magnetic particle and/or dye penetrant tests of the remaining metal in the presence of the Engineer
**Figure 403**

**Detail A**
- Proposed Expansion Windlock

- Existing Anchor Span
- Existing Suspended Span
- Angles
- Jacking Beam
- Shim Plate as Required to Provide Proper Clearance See Details This Sheet
- Plate as Required to Provide Proper Clearance

**Detail B**
- Proposed Fixed Windlock

- Existing Pin Plate
- High Strength Bolt (Typ.)
- Grind Corner of Plate to Clear Flange Weld
- Plate (Typ.) Grind to Bear on Flange

- Finding Stiffener
- Slotted Hole

- Plate
- Pin

- Windlock Pin
- Windlock Anchor Block
- Existing Expansion Windlock
- Existing Fixed Windlock

**Typical Bearing Installation**
- Existing Girders
- Uniform Cap
- Elastomeric Bearing
- High Strength Bolt
- Beveled Shim Plate as Required for Clearance
- Jacking Beam
- Grade 60 Elastomer
- ASTM A572 Steel Shim (Typ.)
- Deep Tapped Hole for High Strength Bolt

**Bearing Notes:**
1. Elastomeric Pad to be Attached to Bottom Plate with a Vulcanized Bond
TERMS:

EXAMPLE CONCRETE DECK POUR SEQUENCES:

ACCEPTABLE

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

UNACCEPTABLE

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

DECK CONCRETE, SEQUENCE OF PLACING
VERTICAL EXTENSION OF STRUCTURAL EXPANSION JOINTS INCLUDING ELASTOMERIC STRIP SEALS
THE OFFICE SHOULD BE CONTACTED FOR MORE CONTEMPORARY RECOMMENDATIONS FOR SEMI-INTEGRAL REHABILITATION OF TURNBACK WINGWALLS.

REHABILITATION EXAMPLE SHOWS EXISTING ROCKER TYPE BEARINGS CAN BE INCORPORATED IN A SEMI-INTEGRAL ABUTMENT DESIGN.
REHABILITATION EXAMPLE SHOWS EXISTING ROCKER TYPE BEARINGS CAN BE INCORPORATED IN A SEMI-INTEGRAL ABUTMENT DESIGN.

THE OFFICE SHOULD BE CONTACTED FOR MORE CONTEMPORARY RECOMMENDATIONS FOR SEMI-INTEGRAL REHABILITATION OF TURNBACK WINGWALLS.

---

**Figure 408**

- **SOUTHEAST**
  - Drill hole as required to match vertical reinforcement.
  - PVC waterstop (type) see detail.
  - Existing wingwall.

- **SOUTHWEST**
  - PVC waterstop (type) see detail.

- **NORTHWEST**
  - PVC waterstop (type) see detail.
  - Existing wingwall.

- **NORTHEAST**
  - PVC waterstop (type) see detail.
  - Existing wingwall.
  - Drill hole as required to match vertical reinforcement.
DESIGNER:
CONTACT THE DISTRICT TO DETERMINE THE DESIRED DISPOSAL OF THE EXISTING RAILING

REMOVAL PLAN

CUT OFF EXISTING CURB PLATES AT 1 INCH ABOVE CURB DECK AT CURB PLATE HOLE LOCATIONS UNLESS SPECIFIED. RAILING FACED AS PER PLAN.

MODIFIED CURB PLATES

REMOVAL PLAN

EXISTING CURB PLATES

AS EXAMPLE ONLY, NOT FOR DETAIL.

RAILING FACED, AS PER PLAN, TYPE A

RAILING FACED, AS PER PLAN, TYPE B

ESTIMATING INFORMATION

ALL ITEMS INCLUDED UNLESS SHOWN OR SPECIFIED AS PER PLAN

COMMON NOTES:
1. EXISTING MEDIAN LOCATIONS ARE NOT KNOWN
2. ALL REINFORCING STEEL SHALL BE SPACED AS SHOWN, VERTICAL STEEL SHALL BE SHOWN AT CONTROL JOINTS OR 30 LINEAL FT.
3. ALL LITTORAL PLATE STEEL BE CONTINUOUS WITH MIN. 1/2 INCH SPACING
4. CONCRETE COVER SHALL BE 4" TYPICAL
5. REINFORCING STEEL - 6/4" @ 20 LBP; 4 5/8" @ 15 LBP; 4 3/4" @ 15 LBP
6. FOR ADDITIONAL NOTES AND INFORMATION SEE DRAWING AND GENERAL NOTES
7. THE INFORMATION DETAIL APPLIES TO BOTH THE TRAFFIC ENGINEERING AND MINERAL RAILINGS
8. CURB SHALL BE AS PER DRAWING, METHOD #6

INDICATES REMOVAL AREA AVAILABLE FOR FUTURE STEEL PASS METH #8.

SHOWN AS LAYING CURB PLATE FACING ESP. PLEDGING RAILING FACED AS PER PLAN
<table>
<thead>
<tr>
<th>BRIDGE NO.</th>
<th>TYPE</th>
<th>DECK AREA</th>
<th>DECK AREA SURVEYED</th>
<th>TOP SURFACE OF DECK</th>
<th>SEPARATE WEARING SURFACE THICKNESS</th>
<th>CURRENT DEPTH TO REBAR</th>
<th>UNDER DECK CONDITION</th>
<th>DATE</th>
<th>SURVEY METHOD</th>
<th>UNSOUND AREA MEASURED</th>
<th>PERCENT OF TOTAL DECK AREA</th>
<th>AVERAGE DEPTH</th>
<th>EST. FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>50. YDS.</td>
<td>50. YDS.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

1. IT IS THE INTENT TO SURVEY THE ENTIRE DECK.

2. CORE ALL DECKS A MINIMUM OF 12 CORES FOR (2,500 SQ. FT., 1 FOR 2,500 TO 5,000 SQ. FT., 4 FOR 5,000 TO 10,000 SQ. FT., 1 ADDITIONAL CORE FOR EACH 10,000 SQ. FT.).

3. USE CORES/PACHOMETER TO DETERMINE DEPTH TO REBARS. USE CORES TO DETERMINE THICKNESS OF SEPARATE WEARING SURFACE.

4. FOR UNDERDECK CONDITION, DESCRIBE AND GIVE +, E.G. 5% WET, UNSOUND, CRACKED, LEACHING OR ANY COMBINATION.

5. SURVEY METHOD MAY BE ROD, CHAIN, DRAG, ELECTROMAGNETIC SOUNDING, NUCLEAR MAGNETIC RESONANCE, ACOUSTIC TECHNIQUE, HALF CELL OR PENETRATING RADAR.

6. ESTIMATED FACTOR: THE NUMBER BY WHICH THE CURRENT UNSOUND SURFACE IS MULTIPLIED TO OBTAIN THE UNSOUND SURFACE 6 TO 9 MONTHS LATER (INCLUDING ONE WINTER). IN LIEU OF A MORE RELIABLE FACTOR USE 1.15. IF 2 WINTERS PASS BEFORE OVERLAY IS TO BE PLACED, UPDATE SURVEY.

7. ESTIMATED QUANTITIES: UNSOUND SQ. YDS. X ESTIMATING FACTOR = PATCHING SQ. YDS.

8. FOR MEASURED UNSOUND AREA > 40% OF TOTAL DECK AREA, A UNIFORM DEPTH OF CONCRETE OVER THE ENTIRE DECK AREA SHOULD BE SET UP TO BE REMOVED USING A HYDRODEMOLITION OPTION.

9. FOR DECKS 500 FT. OR LONGER, RESULTS OF CL- CONTENT AND REBAR ELECTRICAL POTENTIALS SHOULD BE ATTACHED TO THE DECK CONDITION SURVEY REPORT.

10. CORE DESCRIPTIONS FOR ALL BRIDGES SHOULD BE INCLUDED AS REMARKS WITH THIS REPORT FORM.

SUBMITTED BY: ___________________________ DATE: ___________________________
DEEP HAUNCH DETAIL

DETERMINE THE LENGTH OF THE SHEAR CONNECTORS ACCORDING TO AASHTO 10.38.2.2 ASSUMING THE TOP OF THE TOP FLANGE TO BE THE BOTTOM OF THE SLAB.
Procedure:
1. Drill 38 mm diameter hole through web and backing bar
2. Remove shaded area by grinding to radius of original hole. Final surfaces shall be smooth.
3. Perform magnetic particle and/or dye penetrant tests of the remaining metal in the presence of the Engineer

$h = \text{ht. of groove weld} + 3 \ mm$
TERMS:

LONG CANTILEVER  SHORT CANTILEVER  SPAN CONTIGUOUS WITH SHORT CANTILEVER

EXAMPLE CONCRETE DECK POUR SEQUENCES:

ACCEPTABLE

EXPANSION JOINT

2nd

1st

1st

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

UNACCEPTABLE

EXPANSION JOINT

2nd

2nd

1st

1st

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

DECK CONCRETE, SEQUENCE OF PLACING

Figure 404M
VERTICAL EXTENSION OF STRUCTURAL EXPANSION JOINTS INCLUDING ELASTOMERIC STRIP SEALS
THE OFFICE SHOULD BE CONTACTED FOR MORE CONTEMPORARY RECOMMENDATIONS FOR SEMI-INTEGRAL REHABILITATION OF TURNBACK WINGWALLS.

REHABILITATION EXAMPLE SHOWS EXISTING ROCKER TYPE BEARINGS CAN BE INCORPORATED IN A SEMI-INTEGRAL ABUTMENT DESIGN.

PLAN

ELEV. E *

ELEV. F *

TOP OF PROPOSED BACKWALL

BRIDGE SEATS

ABUTMENT ELEVATION

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRIDGE 135'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BRIDGE 150'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BRIDGE 135'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BRIDGE 150'</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ELEVATIONS ARE SHOWN AT FRONT CORNER OF PROPOSED BACKWALL ** VIEW MAY BE OPPOSITE DEPENDING ON BRIDGE AND ABUTMENT

TABLE OF ELEVATIONS

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>ELEV.</th>
<th>ELEV.</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRIDGE 135'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BRIDGE 150'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NORTH ABT.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SOUTH ABT.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
REHABILITATION EXAMPLE SHOWS EXISTING ROCKER TYPE BEARINGS CAN BE INCORPORATED IN A SEMI-INTEGRAL ABUTMENT DESIGN.

THE OFFICE SHOULD BE CONTACTED FOR MORE CONTEMPORARY RECOMMENDATIONS FOR SEMI-INTEGRAL REHABILITATION OF TURNBACK WINGWALLS.
<table>
<thead>
<tr>
<th>BRIDGE NO.</th>
<th>TYPE</th>
<th>DECK AREA</th>
<th>DECK AREA SURVEYED</th>
<th>TOP SURFACE OF DECK</th>
<th>SEPARATE WEARING SURFACE</th>
<th>CURRENT DEPTH TO REBAR</th>
<th>UNDER DECK CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. IN THE INTENT TO SURVEY THE ENTIRE DECK.
2. CORE ALL DECKS A MINIMUM OF 12 CORES FOR (225 50 m: 3 FOR 225 TO 450 50 m: 4 FOR 450 TO 900 50 m: 1 ADDITIONAL CORE FOR EACH 900 50 m).
3. USE CORES/PACHOMETER TO DETERMINE DEPTH TO REBARS. USE CORES TO DETERMINE THICKNESS OF SEPARATE WEARING SURFACE.
4. FOR UNDERDECK CONDITION, DESCRIBE AND GIVE 1. E.G. 5% WET, UNSOUND, CRACKED, LEACHING OR ANY COMBINATION.
5. SURVEY METHOD MAY BE MOB, CHAIN, DRAG, ELECTROMAGNETIC SOUNGING, NUCLEAR MAGNETIC RESONANCE, ACOUSTIC TECHNIQUE, HALF CELL OR PENETRATING RADAR.
6. ESTIMATED FACTOR: THE NUMBER BY WHICH THE CURRENT UNSOUND SURFACE IS MULTIPLIED TO OBTAIN THE UNSOUND SURFACE 6 TO 9 MONTHS LATER (INCLUDING ONE WINTER). IN LIEU OF A MORE RELIABLE FACTOR USE 1.15. IF 2 WINTERS PASS BEFORE OVERLAY IS TO BE PLACED, UPDATE SURVEY.
7. ESTIMATED QUANTITIES: UNSOUND m² X ESTIMATING FACTOR = PATCHING m².
8. FOR MEASURED UNSOUND AREA > 40% OF TOTAL DECK AREA, A UNIFORM DEPTH OF CONCRETE OVER THE ENTIRE DECK AREA SHOULD BE SET UP TO BE REMOVED USING A HYDRODEMOLITION OPTION.
9. FOR DECKS 150m OR LONGER, RESULTS OF CL- CONTENT AND REBAR ELECTRICAL POTENTIALS SHOULD BE ATTACHED TO THE DECK CONDITION SURVEY REPORT.
10. CORE DESCRIPTIONS FOR ALL BRIDGES SHOULD BE INCLUDED AS REMARKS WITH THIS REPORT FORM.

SUBMITTED BY: __________________________ DATE: ____________
DEEP HAUNCH DETAIL

DETERMINE THE LENGTH OF THE SHEAR CONNECTORS ACCORDING TO AASHTO 10.38.2.2 ASSUMING THE TOP OF THE TOP FLANGE TO BE THE BOTTOM OF THE SLAB.

Figure 412M
SECTION 500 – TEMPORARY STRUCTURES

501 GENERAL

This section is a supplement to CMS 502, Structures For Maintaining Traffic. All design guidelines of CMS 502 apply.

502 PRELIMINARY DESIGN

For the Structure Type Study, the Designer shall show the grade, alignment, approximate location and width of the temporary structure on the Preliminary Structure Site Plan.

For the Preliminary Design Report, the Designer shall show the grade and the alignment of the temporary structure on the Site Plan. The Designer shall also determine the roadway width, hydraulic design, clearance requirements, and all other design parameters in conjunction with the development of the preliminary design. When the temporary structure can adequately be shown on the Site Plan for the permanent bridge, a Site Plan for the temporary structure is not required. The required Site Plan information shall be as detailed in Section 200. The Designer shall submit the preliminary design of the temporary structure concurrently with the Preliminary Design Report at the Stage 1 Detailed Design Review Submission for the permanent structure.

502.1 HYDRAULICS

The design year and other hydraulic requirements for temporary structures are defined in CMS 502.02. In addition to those requirements, scour depths for the design year discharge shall be calculated and accounted for in the design of the temporary bridge and its foundation.

With the owner’s approval, the design year may be reduced for low volume roads with an ADT less than 200.

The designer shall show the water surface elevation (“high water”) and velocity of the design year discharge on the temporary structure plans. The lowest portion of the superstructure of the temporary bridge shall clear the design year discharge.

Culvert pipes may be used in lieu of a bridge structure provided controls specified in Section 1006 of the ODOT Location and Design Manual are not exceeded for the design year discharge.

503 DETAIL DESIGN

The temporary structure detail plans shall be complete and independent of the permanent structure plans. The temporary structure detail plans shall include general plan and elevation...
views, general notes, a table of estimated quantities, a reinforcing steel bar list and all necessary
detail plan sheets. The Designer should clearly indicate that the temporary structure will be paid
for under one Lump Sum bid item - Item 502, Structure for Maintaining Traffic, and the table
provided for estimated quantities is “For Estimating Purposes Only”.

Temporary bridge structures shall be designed to withstand the same loads as the permanent
structure as defined in the current AASHTO Standard Specifications for Highway Bridges and
this Manual. These loads and forces shall include, but not be limited to, dead load, live load with
impact, wind, thermal effects, centrifugal effect, earth pressure, buoyancy, ice pressure, stream
current, longitudinal and transverse forces and erection stresses.

For ice pressure loads, the thickness of ice shall be assumed to be 6 inches [150 mm], with a 200
psi [1.4 MPa] effective ice strength. The force shall be assumed to act at the level of the design
year highwater elevation.

The temporary bridge plans shall show all the yield and/or allowable stresses used in the design.
An increase of the allowable stresses over AASHTO requirements or this Manual is not
permitted.

The bridge railing for the temporary structures shall meet the requirements of Section 304 of this
Manual. If the Designer elects to use standard Type 5 or 5A guardrail or standard portable
concrete barrier, the Designer should account for the deflection characteristics of the barrier.

Generally a temporary structure should be designed to be easily constructed and removed with
minimal cost. The following items should be considered when designing a temporary bridge:

A. Timber decks, H pile bents, and simple spans are commonly used.

B. Locally available lumber should be specified. The allowable design unit stresses of the
   lumber used in the design shall be specified in the plans. State whether timber sizes are full
   sawn or standard dressed sizes.

C. The nominal thickness of wood plank or strip floor shall be 3 inches [75 mm] minimum.

D. Timber floors shall be securely fastened to the stringers and stringers shall be securely
   fastened to the pier and abutment caps.

E. When circumstances permit, all or part of the existing bridge may be used for the run-around.

F. Field welded connections shall require nondestructive testing as per 513. Bolted connections
   are preferred and generally are more economical.

G. Designs that minimize debris accumulation should be considered.

H. Shop drawings are not required. Adequate plan details need to be provided.
I. The road surface on the temporary structure shall have antiskid characteristics, crown, drainage and superelevation in accordance with all ODOT and AASHTO publications.

504 GENERAL NOTES

The designer should provide plan note(s) with the Temporary Structure plans similar to the following:

A. The Contractor may substitute used or alternate members for the members shown on the Temporary Structure Plans, provided that the strength of the substitute or alternate member is equal to or greater than the original member. Maintain waterway opening size and required clearances. Submit calculations for the substitute or alternate member according to 502. Use only new bolts.

B. Structural steel need not be painted.

The following instructions are provided to assist in developing the necessary general notes.

When 513 Structural Steel is specified in the plans, only the following CMS descriptions shall apply:

A. Straightening.....................................................................................................................513.11

B. Holes for High Strength and Bearing Bolts ......................................................................513.19

C. High Strength Steel Bolts, Nuts and Washers ..................................................................513.20

D. Welding.............................................................................................................................513.21

E. Nondestructive Testing .....................................................................................................513.25

F. Shipping, Storage and Erection.........................................................................................513.26

When 511 Class "C" is specified in the plans, 511.18 shall be waived.

The following notes shall be included in the Structure General Notes. In the roadway plans the pay item description “614 Maintenance of Traffic” shall include an “as per Plan.” Coordination with the roadway plans for this item is required.

A. MAINTENANCE: Maintain all portions of the temporary structure in good condition with regard to strength, safety and ridability. The Department will consider this maintenance to be incidental to Item 614, Maintaining Traffic. Maintain the waterway opening shown on the
plans at all times. If debris accumulates within the waterway opening or on any part of the structure promptly remove the debris. The Department will compensate for debris removal according to 109.05.

B. CLOSING OF THE TEMPORARY STRUCTURE: If for any reason or at any time the temporary structure’s ability to safely carry traffic is in question, immediately take the actions necessary to protect traffic, repair and reopen the temporary structure. When closing a temporary structure for this purpose, immediately notify the Engineer and the appropriate law enforcement agency. Water elevations exceeding the design (5) year highwater elevation or an excessive accumulation of debris within the waterway opening shall be sufficient reasons to close the temporary structure. Mark the design (5) year highwater elevation with fluorescent paint on the temporary structure, at a visible location. The Department will consider the costs associated with closing the temporary structure to be incidental to Item 614, Maintaining Traffic.
SECTION 600 – TYPICAL GENERAL NOTES

601 DESIGN REFERENCES

601.1 GENERAL

This section contains various typical general notes. The designer needs to assure that the typical notes are complete and apply to the specific project. These notes may need to be revised or specific notes must be written to conform to the actual conditions that exist on each individual project.

601.2 STANDARD DRAWINGS AND SUPPLEMENTAL SPECIFICATIONS

The designer shall list all Standard Bridge Drawings and Supplemental Specifications that apply, giving date of approval or latest revision date, if revised.

The Designer shall also ensure the listed Standard Bridge Drawings and Supplemental Specifications are transferred to the project plans title sheet and match the information on the title sheet.

[1] REFER to the following Standard Bridge Drawing(s):

____________Dated (revised) ___________

____________Dated (revised) ___________

____________Dated (revised) ___________

and to the following Supplemental Specification(s):

____________Dated ____________

____________Dated ____________

____________Dated ____________

601.3 DESIGN SPECIFICATIONS

The designer shall include the following note specifying the design specifications used on the structure. If the note is not correct, then the note should be revised with the correct criterion that describes the design specifications for the structure.

* - Designer should fill-in current edition and latest interims.

The use of note [2] stipulates the use of standard Live Load Distribution and designs based on AASHTO’s standard specifications, assumptions and standard beam theory design. For the vast majority of ODOT bridges this criterion is not only adequate but also advantageously conservative. There are structure types which, due to either AASHTO’s own limitations or the type of structure, require specific live load distribution factors or other analysis methods other than classical beam theory to analyze the structure. Some examples may include a highly skewed slab bridge, a curved steel girder bridge, cable stayed bridges, etc.. If the structure’s analysis required the use of 2D or 3D models including grillage, finite element, finite strip, classical plate solutions the following note should be added.

[2a] SPECIAL DESIGN SPECIFICATIONS: This bridge required the use of a ____ (# two or three) dimensional model using the _____ (grillage, finite element, finite strip, classical plate theory, etc) design method to analyze the structure. The computer program used for structural analysis was _______. The bridge components designed by this method and the live load distribution factors used were:

Dead Load Distribution: (The designer is to explain the assumptions used in how the dead load was applied and distributed)

Live Load distribution Factors:
Exterior Members - ___ for wheel (or axle) load & ___ for lane load moments.  
- ___ for wheel (or axle) load & ___ for lane load shears
Interior Members - ___ for wheel (or axle) load & ___ for lane load moments.  
- ___ for wheel (or axle) load & ___ for lane load shears

NOTE TO DESIGNER: Modify the wording of the note as necessary. Also amend the Design Specifications note [2] with the wording “excepted as noted elsewhere in the plans”

602 DESIGN DATA

The designer shall include the following pertinent design information with the Structure General Notes for all bridge plans:

602.1 DESIGN LOADING

For bridges designed for highway loads, the design loading shall be:

Future Wearing Surface (FWS) of _60_ Lbs/ft².

[3M] DESIGN LOADING: MS-22.5 (#), Case(I or II)** and the Alternate Military Loading.

Future Wearing Surface (FWS) of _2.87_ kPa.

* Designer to modify load if appropriate.
** The statement "Case (I or II)" applies only to steel bridges.
# Replace vehicle type with HS-20, or metric equal, if the design is for a rehabilitation of an existing structure

For bikeway/pedestrian bridges that will not accommodate vehicular traffic the design loading shall be:

[4] DESIGN LOADING: ___ Lb/ft²

[4M] DESIGN LOADING: ___ kN/m²

* As defined for the specific structure in accordance with the AASHTO Guide Specifications for Design of Pedestrian Bridges

For bikeway/pedestrian bridges subject to vehicular traffic the design loading shall be:

[5] DESIGN LOADING: ___ Lb/ft² and H15-44 vehicle

[5M] DESIGN LOADING: ___ kN/m² and M13.5 vehicle

* As defined for the specific structure in accordance with the AASHTO Guide Specifications for Design of Pedestrian Bridges

602.2 DESIGN STRESSES

A. General Design Data For Load Factor design:

[6] DESIGN DATA:

Concrete ___* - compressive strength 4500 psi (superstructure)

Concrete ___* - compressive strength 4000 psi (substructure)
# Concrete S Modified - compressive strength 4000 psi (drilled shaft)

* Class S or Class HP Concrete for superstructure
  Class C or Class HP Concrete for substructure

# Only included if drilled shafts are being constructed

Reinforcing steel - ASTM A615 or A996
Grade 60 minimum yield strength = 60,000 psi
Spiral reinforcement may be plain bars, ASTM A82 or A615

If spiral reinforcing bars are not used, omit the portion of the note beginning with “spiral”.

** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 50,000 psi
A709 Grade 36 - yield strength 36,000 psi

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

[6M] DESIGN DATA:

Concrete ___*___ - compressive strength 31.0 MPa (superstructure)

Concrete ___*___ - compressive strength 27.5 MPa (substructure)

# Concrete S Modified - compressive strength 27.5 MPa (drilled shaft)

* Class S or Class HP Concrete for superstructure
  Class C or Class HP Concrete for substructure

# Only included if drilled shafts are being constructed

Reinforcing steel - A615M or A996M
Grade 420 minimum yield strength = 420 MPa
Spiral reinforcement may be plain bars, ASTM A82 or A615M

If spiral reinforcing bars are not used, omit the portion of the note beginning with “spiral”.

** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 350 MPa
A709 Grade 36 - yield strength 250 MPa

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

B. General Design Data For Service Load Design (Working Stress Design):

[7] DESIGN DATA:

Concrete ** - unit stress 1500 psi (superstructure)

Concrete ** - unit stress 1300 psi (substructure)

# Concrete S Modified - unit stress 1300 psi (drilled shaft)

* Class S or Class HP Concrete for superstructure
  Class C or Class HP Concrete for substructure

# Only included if drilled shafts are being constructed

Reinforcing Steel - ASTM A615 or A996
Grade 60 - unit stress 24,000 psi
Spiral reinforcement may be plain bars, ASTM A82 or A615

If spiral reinforcing bars are not used, omit the portion of the note beginning with “spiral”.

** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 27,000 psi
A709 Grade 36 - yield strength 20,000 psi

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

[7M] DESIGN DATA:

Concrete ** - unit stress 10.3 MPa (superstructure)

Concrete ** - unit stress 9.2 MPa (substructure)

# Concrete S Modified - unit stress 9.2 MPa (drilled shaft)

* Class S or Class HP Concrete for superstructure
  Class C or Class HP Concrete for substructure
Only included if drilled shafts are being constructed

Reinforcing Steel - ASTM A615M or A996M
Grade 420 - unit stress 165 MPa
Spiral reinforcement may be plain bars, ASTM A82 or A615M

If spiral reinforcing bars are not used, omit the portion of the note beginning with “spiral”.

** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 186 MPa
A709 Grade 36 - yield strength 138 MPa

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

C. Additional Design Data for Prestressed Concrete Members:

Provide the following note in addition to either note [6] or [7].

[8] DESIGN DATA:

Concrete for prestressed beams:
Compressive Strength (final) - 5500 psi **
Compressive Strength (release) - 4000 psi **

Prestressing strand:
Area = 0.153 in² **
Ultimate Strength = 270 ksi
Initial stress = 202.5 ksi (Low relaxation strands)

** Revise the prestressed concrete strength values for final strength and strength at release if a design strength is different than the values listed above. Also, modify the diameter and area of the strands as required.

[8M] DESIGN DATA:

Concrete for prestressed beams:
Compressive Strength (final) - 38 Mpa **
Compressive Strength (release) - 27.5 Mpa **

Prestressing strand:
Area = 99 mm² **
Ultimate Strength = 1860 MPa  
Initial stress = 1395 MPa (Low relaxation strands)

** Revise the prestressed concrete strength values for final strength and strength at release if a design strength is different than the values listed above. Also, modify the diameter and area of the strands as required.

### 602.3 FOR RAILWAY PROJECTS

For structures carrying railroad traffic, provide the following notes on the project plans:

[9] **DESIGN SPECIFICATIONS:** This structure conforms to the requirements of the "Manual for Railway Engineering" by the American Railway Engineering and Maintenance-of-way Association, XXXX * Edition.

**CONSTRUCTION AND MATERIAL SPECIFICATIONS:** State of Ohio, Department of Transportation, dated January 1, XXXX. *

* Designer should fill-in current edition and latest interims.

**NOTE TO DESIGNER:** Note [2A] may be required if special criteria or distributions have been used for the design of this rail structure. See [2A] and determine if a modified note is required for inclusion.

Use the following note, modified as necessary to meet AREMA and/or a specific railroad criterion, with all railroad structures.

[10] **DESIGN DATA :**

Design Loading - Cooper E-80 with diesel impact (or specific railroad criteria)

Concrete * - unit stress 1500 psi (superstructure)
Concrete ** - unit stress 1333 psi (substructure)

# Concrete S Modified - unit stress 1300 psi (drilled shaft)

* Class S or Class HP Concrete for superstructure  
Class C or Class HP Concrete for substructure

# Only included if drilled shafts are being constructed
** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 27,000 psi
A709 Grade 36 - yield strength 20,000 psi

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

Reinforcing steel - ASTM A615-unit stress 24,000 psi

[10M] DESIGN DATA:

Design Loading - Cooper E-80 with diesel impact (or specific railroad criteria)

Concrete ** - unit stress 10.3 MPa (superstructure)
Concrete *** - unit stress 9.2 MPa (substructure)

# Concrete S Modified - unit stress 1300 psi (drilled shaft)

* Class S or Class HP Concrete for superstructure
  Class C or Class HP Concrete for substructure

# Only included if drilled shafts are being constructed

** Structural Steel

ASTM A709 Grade 50W or A709 Grade 50 - yield strength 186 MPa
A709 Grade 36 - yield strength 138 MPa

** If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

Reinforcing steel - ASTM A615M-unit stress 167 Mpa

602.4 DECK PROTECTION METHOD

If any of the following deck protection method(s) have been specified in the plans, include the following note in the Design Data section of the Structure General Notes (modify as necessary for the specific structure):

[11] DECK PROTECTION METHOD:

Epoxy coated reinforcing steel
2½” [65 mm] concrete cover

Superplasticized dense, Micro-silica, Epoxy, or Latex modified concrete overlay (Only applicable for existing decks)

Waterproofing and asphalt concrete overlay

Steel drip strip

Other (Specify)

602.5 MONOLITHIC WEARING SURFACE

Furnish the following note for concrete bridge decks.

[12] MONOLITHIC WEARING SURFACE is assumed, for design purposes, to be 1 inch [25 mm] thick.

602.6 SEALING OF CONCRETE SURFACES

Use the following notes when permanent anti-graffiti coatings are required:

[13a] ITEM 512 SEALING OF CONCRETE SURFACES, AS PER PLAN, (PERMANENT GRAFFITI PROTECTION):
Apply a permanent graffiti coating qualified according to Supplement 1083 that is compatible with the concrete sealer over which it is applied. Apply the graffiti coating in accordance with the manufacturer’s printed instructions.

Use the following notes when sacrificial anti-graffiti coatings are required:

[13b] ITEM 512 SEALING OF CONCRETE SURFACES, AS PER PLAN, (SACRIFICIAL GRAFFITI PROTECTION):
Apply a permanent graffiti coating qualified according to Supplement 1083 that is compatible with the concrete sealer over which it is applied. Apply the graffiti coating in accordance with the manufacturer’s printed instructions.

603 EXISTING STRUCTURE REMOVAL NOTES

The following sample notes will serve as a guide in composing the note(s) for the removal of the existing structure. Modify the notes as required to fit the conditions. Use the following note if it is the desire of the owner to salvage any portion of the bridge.

[14] REMOVAL OF EXISTING STRUCTURE: Carefully dismantle the ________ and
store along the right-of-way for disposal by the State’s forces.

Describe the degree of care to be exercised in the removal in sufficient detail to allow accurate bidding. For example, for a truss bridge where the stringers and floor beams are to be salvaged for reuse but it is permissible to flame cut the truss members, state that clearly along with any other restrictions or allowances. If this option is used, the pay item shall be “as per plan”.

Use the following note when removal of structure to 1 foot [300 mm] below ground line as specified in CMS 202 will not fill the specific requirements of the project.

**ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN:** Remove abutments to Elev.____. Remove piers to Elev.____.

Use the following note when special protection of an existing structure to be incorporated into a
new structure is required.

[16] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN: This item shall include the elements indicated in the plans and general notes and that are not separately listed for payment, except for wearing course removal. Items to be removed include all existing materials being replaced by new construction and miscellaneous items that are not shown to be incorporated into the final construction and are directed to be removed by the Engineer. The use of explosives, headache balls and/or hoe-rams will not be permitted. The method of removal and the weight of hammer shall be approved by the Engineer. Perform all work in a manner that will not cut, elongate or damage the existing reinforcing steel to be preserved. Chipping hammers shall not be heavier than the nominal 90-pound [41 kilogram] class. Pneumatic hammers shall not be placed in direct contact with reinforcing steel that is to be retained in the rebuilt structure. Submit construction plans according to CMS 501.05.

[17] Note Retired – see Appendix

603.1 CONCRETE DECK REMOVAL PROJECTS

Use the following removal note for concrete deck removal projects, where the existing superstructure is to remain. Delete the portions in the note that are not appropriate for the specific project.

[18] ITEM 202, PORTIONS OF STRUCTURE REMOVED, AS PER PLAN DESCRIPTION: This work consists of the removal of concrete decks including sidewalks, parapets, railings, deck joints and other appurtenances from steel supporting systems (beams, girders, cross frames, etc.). The provisions of Item 202 apply except as specified by the following notes. Perform work carefully during deck removals to protect portions of such systems that are to be salvaged and incorporated into the proposed structure. The use of explosives, headache balls and/or hoe ram type of equipment is prohibited. Submit construction plans according to CMS 501.05.

PROTECTION OF STEEL SUPPORT SYSTEMS: Before deck slab cutting is permitted, draw the outline of primary steel members in contact with the bottom of the deck on the surface of deck. Drill small diameter pilot holes 2 inches [50 mm] outside these lines to confirm the location of flange edges. Deck cuts over or within 2 inches [50 mm] of flange edges shall not extend lower than the bottom layer of deck slab reinforcing steel. Cuts made outside 2 inches [50 mm] of flange edges may extend the full depth of the deck. Perform work carefully during cutting of the deck slab to avoid damaging steel members that are to be incorporated into the proposed structure. Replace or repair steel members damaged by the deck slab cutting operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Director. Obtain the Director’s approval before performing repair.
PROTECTION OF PRESTRESSED CONCRETE SUPPORT SYSTEMS: Before deck slab cutting is permitted, draw the outline of primary prestressed concrete members in contact with the bottom of the deck on the surface of deck. Drill small diameter pilot holes 2 inches [50 mm] outside these lines to confirm the location of the edges of those members. Deck cuts over or within 2 inches [50 mm] of flange edges shall not extend lower than the bottom layer of deck slab reinforcing steel. Cuts made outside 2 inches [50 mm] of flange edges may extend the full depth of the deck. Perform work carefully during cutting of the deck slab to avoid damaging prestressed concrete members that are to be incorporated into the proposed structure. Replace or repair prestressed concrete members damaged by the deck slab cutting operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Director. Obtain the Director’s approval before performing repair.

REMOVAL METHODS: The Contractor may remove concrete by cutting and by means of hand operated pneumatic hammers employing pointed or blunted chisel type tools. For removals over structural members (prestressed box beam, I-beam, steel beam steel girder, etc), the Contractor may use a hammer heavier than 35 pounds [16 kilograms] but not to exceed 90 pounds [41 kilograms] unless approved by the Engineer. Removal methods over structural members shall ensure adequate depth control and prevent nicking or gouging the primary structural members.

Due to the possible presence of attachments (e.g., finishing machine, scupper and form supports, etc.) to existing structural members, perform work carefully during deck removal to avoid damaging structural members that are to remain. Replace or repair structural members damaged by the removal operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Director. Obtain the Director’s approval before performing repair.

DECK REMOVALS - COMPOSITE DECK DESIGNS - STEEL SUPERSTRUCTURES: Due to the presence of welded studs to the existing structural steel, submit a detailed procedure of the deck removal to the Engineer at least 7 days before construction begins. Department acceptance is not required. The procedure shall include all details, equipment and methods to be used for removal of the concrete over the flanges and around the studs. Replace or repair main steel and studs damaged by the removal operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Director. Obtain the Director’s approval before performing repair.

DECK REMOVALS - COMPOSITE DECK DESIGNS - PRESTRESSED SUPERSTRUCTURES: Due to the presence of composite reinforcing steel between the deck and the prestressed beam flanges, submit a detailed procedure of the deck removal to the Engineer at least 7 days before construction begins. Department acceptance is not required. The procedure shall include all details, equipment and methods of removal
over the prestressed beams and around the composite reinforcing steel. Replace or repair prestressed members and composite reinforcing damaged by the removal operations at no cost to the project. At least 7 days before performing repair work, submit a proposed repair plan, developed by an Ohio registered professional engineer to the Director. Obtain the Director’s approval before performing repair.

EXISTING WELDED ATTACHMENTS: Remove existing welded attachments (e.g., finishing machine and form supports; and supports for scuppers and bulb angles which are to be removed) located in the designated tension portions of the top flanges of existing steel members and grind the flange surfaces smooth. Carefully grind parallel to the flanges.

MEASUREMENT & PAYMENT: The Department will measure the quantity of removals on a lump sum basis. The Department will pay for the accepted quantities of removals at the contract price for Item 202, Portions of Structure Removed, As Per Plan.

For modifications to or extensions of existing concrete substructure members where aesthetics is a concern, include the following notes in an Item 202, as per plan note.

[19] CUT LINE CONSTRUCTION JOINT PREPARATION: Saw cut boundaries of proposed concrete removals 1 inch [25 mm] deep. Remove concrete to a rough surface. Leave the existing reinforcing steel, if required in the plans, in place. Install dowel bars if specified. Prior to concrete placement abrasively clean joint surfaces and existing exposed reinforcement to remove loose and disintegrated concrete and loose rust. Thoroughly clean the joint surface and exposed reinforcement of all dirt, dust, rust or other foreign material by the use of water, air under pressure, or other methods that produce satisfactory results. Existing reinforcing steel does not have to have a bright steel finish, but remove all pack and loose rust. Thoroughly drench existing concrete surfaces with clean water and allow to dry to a damp condition before placing concrete.
[20]  SUBSTRUCTURE CONCRETE REMOVAL: Remove concrete by means of approved pneumatic hammers employing pointed and blunt chisel tools. Hydraulic hoe-ram type hammers will not be permitted. The weight of the hammer shall not be more than 35 pounds [16 kilograms] for removal within 18 inches [450 mm] of portions to be preserved. Outside the 18 inch [450 mm] limit, the contractor may use hammers not exceeding 90 pounds [41 kilograms] upon the approval of the Engineer. Do not place pneumatic hammers in direct contact with reinforcing steel that is to be retained in the rebuilt structure.

604  TEMPORARY STRUCTURE CONSTRUCTION

Include the applicable portions of the following temporary structure note on the plans if the bridge roadway width is other than 23 feet [7 meters], or if the use of the existing structure is part of the temporary road. See Section 500 for additional information.

[21]  TEMPORARY STRUCTURE roadway width shall be ______ feet [meters]. The existing structure may be moved and used for the temporary structure without strengthening.

605  EMBANKMENT CONSTRUCTION

For all substructures units where embankment construction is involved, provide appropriate embankment construction notes in the Structure General Notes. Consult the Office of Structural Engineering for the recommended notes to use at a specific project site.

605.1  FOUNDATIONS ON PILES IN NEW EMBANKMENTS

The following construction method helps to eliminate any lateral forces on the piles and abutment due to the construction of the embankment, settlement of the subgrade under the embankment and poor construction of the embankment if the piles were driven before the embankment is placed.

For structures with abutments on piles placed in new embankments use the following note:

[22]  PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of * behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment piles until after the above required embankment has been constructed.

*  Generally 200 feet [60 meters]. May be defined by station-to-station dimensions.

For structures with abutments and piers on piles placed in new embankments use the following note:
[22a] PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of * behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment and pier piles, for pier(s)**, until after the above required embankment has been constructed.

* Generally 200 feet [60 meters]. May be defined by station-to-station dimensions.

** Identify specific piers.

For structures with wall type abutments on piles placed in new embankment use the following note:

[23] PILE DRIVING CONSTRAINTS: Prior to driving piles at the abutments, construct the bridge approach embankment behind the abutments up at a 1:1 slope from the top of the heel of the footing* to the subgrade elevation and for a minimum distance of 250 feet [75 meters] behind the abutments. Do not begin the installation of the abutment piles until after the above required embankment has been constructed. After the footing and the breastwall have been constructed, construct the embankment immediately behind the abutments up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation prior to setting the beams on the abutments.

* In some cases the bottom of the heel may be used.

For MSE wall supported abutments with driven piles use the following note:

[23a] PILE DRIVING CONSTRAINTS: Prior to driving abutment piles to the Ultimate Bearing Value (UBV) or to refusal on bedrock, construct the MSE wall and the bridge approach embankment behind the abutment up to the bottom of the footing for a minimum distance of * behind each abutment. The Contractor may pre-drive abutment piles before constructing MSE walls. Pre-driving consists of installing the abutment piles into the soil only as far as necessary so that the pile will remain vertical during MSE wall construction. If pre-driving piles, install pile sleeves around piles before constructing the MSE wall. At least three feet of pile must extend above the top of the pile sleeve to meet the requirements of CMS 507.09 regarding splices. Do not drive abutment piles to the UBV or to refusal on bedrock until after the above required MSE wall and embankment have been constructed and a ** calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. After the specified waiting period has elapsed, drive abutment piles to the UBV or to refusal on bedrock. In order to remove any negative skin friction that has developed during the waiting period, drive each abutment pile a distance of at least 0.5 inch.

If not pre-driving abutment piles, install the abutment piles through pile sleeves after the above required MSE wall and embankment have been constructed and the specified waiting period has elapsed.
* Generally 200 feet [60 meters]. May be defined by station-to-station dimensions.
** Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur.

605.2 FOUNDATIONS ON SPREAD FOOTINGS IN NEW EMBANKMENTS

The following construction method helps to eliminate any lateral forces on the foundation due to the construction of the embankment and/or settlement of the subgrade under the embankment. For stub abutments on spread footings being constructed in new embankments provide note [26] or [27] and the following note:

[24] CONSTRUCTION CONSTRAINTS: Prior to constructing the spread footing foundations, construct the bridge approach embankments behind the abutment up at a 1:1 slope from the bottom of the heel of the footing to the subgrade elevation and for a minimum distance of 250 feet [75 meters] behind the abutments. After the abutment footing and breastwall are completed and prior to setting superstructure members, construct the embankment immediately behind the abutment up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation, with Type B granular material conforming to 703.16.C.

NOTE TO DESIGNER: Modify the note, as appropriate, for piers constructed on a spread footing foundation.
For wall type abutments on spread footings with no new embankment provide note [26] or [27] and the following note:

[25] CONSTRUCTION CONSTRAINTS: Fill the void created by excavating for the abutment footings with Type B granular material, 703.16.C. After the footing and the breastwall have been constructed, fill the void behind each abutment up to the beam seat elevation and from the beam seat up on a 1:1 slope to the subgrade elevation prior to constructing the backwall and setting the beams on the abutment.

605.3 EMBANKMENT CONSTRUCTION NOTE

In an attempt to reduce settlements of the roadway approaches, specify the placement of embankment materials in 6 inch [150 mm] lifts. Include one of the following plan notes in the Project General Notes and make reference to the work defined below at the appropriate locations within the plans.

Note that Item 203 is a roadway quantity and coordination with the roadway plans is necessary.

To define the limits of measured pay quantities for bridges with wall-type abutments, provide excavation, backfill, and embankment diagrams (or a composite diagram, where suitable), using schematic abutment cross-sections, showing the boundaries between structure and roadway excavation, and between structure backfill and roadway embankment.

[26] ITEM 203 EMBANKMENT, AS PER PLAN: Place and compact embankment material in 6 inch [150 mm] lifts for the construction of the approach embankment between stations ** to **.

** The approximate limits should be 100 feet behind each abutment

[27] ITEM 203 EMBANKMENT, AS PER PLAN: Place and compact embankment material in 6 inch [150 mm] lifts for the construction of the approach embankment.

605.4 UNCLASSIFIED EXCAVATION

Compute and use pay items for Item 503 as follows:

When an excavation includes 10 yd³ [m³] or more of rock (or shale), itemize the quantity of rock excavation separately under:

Item 503 - Rock (or Shale) Excavation
When the rock (or shale) excavation is under 10 yd$^3$ [m$^3$], do not itemize the rock (or shale) excavation separately. Provide the following pay item:

**Item 503 - Unclassified excavation, including rock (and/or shale)**

When excavation includes no rock (or shale), provide the following pay item:

**Item 503 - Unclassified excavation**

In computing the quantity of Item 503 excavation, the designer should confirm that all removals under items 201, 202 or 203 have been excluded, according to CMS 503.01. Generally, the basis of payment for Item 503 should be yd$^3$ [m$^3$]. Lump sum quantities may be used if authorized by the District and with the understanding that cost may be higher than when specific quantities are used.

### 605.5 PROPRIETARY RETAINING WALLS

For projects with proprietary retaining wall systems supporting bridge abutments on spread footings, provide the following note and table:

**[98] PROPRIETARY RETAINING WALL DATA:**
The proprietary wall supplier shall design the internal stability of a mechanically stabilized earth (MSE) wall in accordance with the SS840 to support the abutment loads provided in the table below. All loads in the table represent unfactored service loads applied to the reinforced soil mass at the base of the concrete footing. DL represents a vertical spread footing strip load that includes the dead load of the approach slab; the dead load of the abutment; and the dead load from the superstructure. LL represents a vertical spread footing strip load that includes only the live load from the superstructure. H represents a horizontal strip load from the superstructure applied perpendicular to the face of wall. Ecc. represents the distance between the geometrical center of the strip footing and the resultant of all loads applied to the footing.

<table>
<thead>
<tr>
<th>Wall Location</th>
<th>DL (k/ft)</th>
<th>LL (k/ft)</th>
<th>H (k/ft)</th>
<th>Ecc. (ft)</th>
<th>Bearing Pressure (k/ft$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For projects with proprietary retaining wall systems supporting bridge abutments on pile foundations, provide the following note:

**[99] PROPRIETARY RETAINING WALL DATA:**
The proprietary wall supplier shall design the internal stability of a mechanically stabilized earth (MSE) wall in accordance with the SS840 to support the
abutment. The design for internal stability shall include an unfactored horizontal strip load from the superstructure of _____ k/ft [kN/m] applied perpendicular to the face of wall at the base of the concrete footing.

606 FOUNDATIONS

606.1 PILES DRIVEN TO BEDROCK

The following note generally will apply where steel-H piles are to be driven to bedrock:

[29] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating soft bedrock for several inches to a minimum resistance of 20 blows per inch [25 mm] or by contacting hard bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve the Ultimate Bearing Value given below for the piles. Payment for dynamic load testing performed at the Contractor’s option is included in the unit price pay item for piles driven.

The Ultimate Bearing Value is ___ tons [kN] per pile for the ____ abutment piles. The Ultimate Bearing Value is ___ tons [kN] per pile for the ____ pier piles.

Abutment piles:
___ piles ____ feet [meters] long, order length

Pier piles:
___ piles ____ feet [meters] long, order length
Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP 250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

606.2  FRICITION TYPE PILES

The following notes, modified to fit the specific conditions for the foundation required, will apply in all cases except where the piles are to be driven to bedrock. Provide the actual calculated Ultimate Bearing Value as shown below:

[30]  PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is ___ tons [kN] per pile for the ___ abutment piles. The Ultimate Bearing Value is ___ tons [kN] per pile for the ___ pier piles.

Abutment piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items

Pier piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour depth is significant.

[30a]  PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is ___ tons [kN] per pile for the ___ abutment piles. The Ultimate Bearing Value is ___ tons [kN] per pile for the ___ pier piles. The pier piles include an additional ___ tons [kN] per pile of Ultimate Bearing Value due to the possibility of losing frictional resistance due to scour.

Abutment piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items
Pier piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items

# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

The following note, modified to fit the conditions, will apply where piles are driven through new embankment and the embankment settlement is expected, by design, to cause calculated downdrag forces.

[30b] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is ___#___ tons [kN] per pile for the ____ abutment piles. The Ultimate Bearing Value is ___#___ tons [kN] per pile for the ____ pier piles. The addition of ___#___ tons [kN] of Ultimate Bearing Value per abutment pile and ___#___ tons [kN] per pier pile is due to the possibility of downdrag forces induced by embankment settlement.

Abutment piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items

Pier piles:
___ piles ___ feet [meter] long, order length
___ Dynamic load testing items

# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

Provide the following note when Static Load Testing is required according to Section 303.4.2.5. Modify the note as necessary to fit the specific condition.

[30c] STATIC LOAD TEST: Perform dynamic testing on the first two production piles to determine the required blow count for the specified Ultimate Bearing Value. Perform the static load test on either pile. Do not over-drive the selected pile. Drive the third and fourth production piles to 75% and 85% of the determined blow count, respectively and perform dynamic testing on them. The test piles and the reduced capacity piles shall not be battered. After installation of the first four production piles, cease all driving operations on piling represented by the static load testing for a minimum of 7 days. After the waiting period, perform pile restrikes on the four piles (two restrike test items). The Engineer will review the results of the pile restrikes and establish the driving criteria.
for the remaining piling represented by the testing. Submit all test results to the Office of Structural Engineering.

For subsequent static load tests, upon completion of a 10,000 ft [3000 m] increment of driven length, repeat the above procedure for the initial static load test. If necessary, the Engineer will revise the driving criteria for the remaining piling accordingly.

When performing the restrike, if the pile has not reached the blow count determined for the plan specified Ultimate Bearing Value, continue driving the pile until this capacity is achieved.

Provide the following note when battered piles are specified.

[B30d] BATTERED PILES: The blow count for battered piles shall be the blow count determined for vertical piles of the same Ultimate Bearing Value divided an efficiency factor (D). Compute the efficiency factor (D) as follows:

\[
D = \frac{1-UG}{\sqrt{(1+G^2)}}
\]

\[U = \text{Coefficient of friction, which is estimated at 0.05 for double-acting air operated or diesel hammers; 0.1 for single-acting air operated or diesel hammers; and 0.2 for drop hammers.}\]

\[G = \text{Rate of batter (1/3, 1/4, etc.)}\]

606.3 STEEL PILE POINTS

Use the following note where steel points are required, and see Section 202.2.3.2.a.

[B31] ITEM 507, STEEL POINTS, AS PER PLAN: Use steel pile points to protect the tips of the proposed steel “H” piling. Furnish steel points from the following manufacturers/suppliers: Associated Pile and Fitting Corporation, 262 Rutherford Blvd., Clifton, New Jersey 07014, phone: (973)773-8400, (800)526-9047, fax: (973)773-8442; International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, North Carolina 28015, phone: (704)821-8200, (888)423-8721, fax: (704)821-8201; Dougherty Foundation Products, Inc., P.O. Box 688, Franklin Lakes, New Jersey 07417, phone: (201)337-5748, fax: (201)337-9022; Versa Steel Inc., 1618 N.E. First Ave., Portland, Oregon 97232, phone: (503)287-9822, (800)678-0814, fax: (503)287-7483; Versabite Piling Accessories, 1704 Tower Industrial Dr., Monroe, North Carolina 28110, phone: (800)280-9950, (704)225-1566, fax: (704)225-1567; or by a manufacturer that can furnish a steel point that is acceptable to Director. The material used for the manufacturing of pile points shall conform to ASTM A27/A27M 65/35 [450/240] – Class 2 – Heat Treated or AASHTO M103/M103M 65/35 [450/240] – Heat Treated. Weld the
pile points to the pile in accordance with AWS D1.5 or the manufacturer’s written welding procedure supplied to the engineer before the welding is performed. Submit a notarized copy of the mill test report to the Engineer.

606.4 PILE SPLICES

Provide the following note when H-piles are specified.

[100] PILE SPLICES: In lieu of using the full penetration butt welds specified in CMS 507.09 to splice steel H-piles, the Contractor may use a manufactured H-pile splicer. Furnish splicers from the following manufacturer:

Associated Pile and Fitting Corporation
8 Wood Hollow Rd. Plaza 1
Parsippany, New Jersey 07054

Install and weld the splicer to the pile sections in accordance with the manufacturer’s written assembly procedure supplied to the Engineer before the welding is performed.

606.5 MINIMUM HAMMER SIZE

[33] Note retired - see appendix

606.6 PILE ENCASEMENT

The following note shall be used where capped pile piers and steel "H" piles are being used for a bridge structure crossing a waterway. The exposed steel piling corrodes at the waterline, or near there. The note should not be used if the capped pile pier standard drawing is being used as standard drawing already specifies pile encasement methods.

[34] ITEM SPECIAL - PILE ENCASEMENT

Encase all steel H-piles for the capped pile piers in Class C concrete. Provide a concrete slump between 6 to 8 inches with the use of a superplasticizer. Place the concrete within a form that consists of polyethylene pipe (707.33), or PVC pipe (707.42). The encasement shall extend from 3 feet [1 meter] below the finished ground surface up to the concrete pier cap. Position pipe so that at least 3 inches [75 mm] of concrete cover is provided around the exterior of the pile.

In lieu of encasing the pile in concrete, galvanize the piles according to 711.02. The galvanizing shall be continuous from a minimum of 3 feet below the finish ground surface up to the concrete pier cap. The galvanized coating thickness shall be a minimum of 4 mils [100 μm]. Repair all gouges, scrapes, scratches or other surface imperfections caused by the handling or the driving of the pile to the satisfaction of the Engineer.

The Department will measure pile encasement by the number of feet. The Department will determine the sum as the length measured along the axis of each pile from the
bottom of the encasement to the bottom of the pier cap. The Department will not pay for
galvanizing provided beyond the project requirements. The Department will pay for
accepted quantities at the contract price for Item – Special, Pile Encasement.

606.7 FOUNDATION BEARING PRESSURE

Provide the following note, with the blanks filled in as appropriate for each individual project, if
there are abutments or piers which are supported by spread footings. Show the actual calculated
maximum bearing pressure under the footing.

[35] FOUNDATION BEARING PRESSURE: _____ footings, as designed, produce a
maximum bearing pressure of _____ tons per square foot [Mpa]. The allowable bearing
pressure is ____ tons per square foot [MPa].

606.7.1 SPREAD FOOTINGS NOT ON BEDROCK

When abutments or piers are supported by spread footings, include the following note to require that reference monuments be constructed in each footing. The purpose of the reference monuments is to document the performance of the spread footings, both short and long term.

[35A] ITEM 511, CLASS *** CONCRETE, **** , AS PER PLAN : * In addition to the requirements of Item 511*, install a reference monument at each end of each spread footing. The reference monument shall consist of a #8, or larger, epoxy coated rebar embedded at least 6" [150 mm] into the footing and extended vertically 4 to 6 inches [100 to 150 mm] above the top of the footing. Install a six inch [150 mm] diameter, schedule 40, plastic pipe around the reference monument. Center the pipe on the reference monument and place the pipe vertical with its top at the finished grade. The pipe shall have a removable, schedule 40, plastic cap. Permanently attach the bottom of the pipe to the top of the footing.

Establish a benchmark to determine the elevations of the reference monuments at various monitoring periods throughout the length of the construction project. The benchmark shall be the same throughout the project and shall be independent of all structures.

Record the elevation of each reference monument at each monitoring period shown in the table below.

The original completed tables will become part of the District’s project plan records. Send a copy of the completed tables to the Office of Structural Engineering.

<table>
<thead>
<tr>
<th>Project Number:</th>
<th>Maximum Bearing Pressure: *</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Number: *</td>
<td>Structure File Number: *</td>
</tr>
<tr>
<td>Benchmark Location:</td>
<td></td>
</tr>
<tr>
<td>Footing Location: *</td>
<td></td>
</tr>
<tr>
<td>Monitoring Period</td>
<td>Left monument</td>
</tr>
<tr>
<td>After footing concrete is placed</td>
<td></td>
</tr>
<tr>
<td>Before placement of superstructure members</td>
<td></td>
</tr>
<tr>
<td>Before deck placement</td>
<td></td>
</tr>
<tr>
<td>After deck placement</td>
<td></td>
</tr>
<tr>
<td>Project completion</td>
<td></td>
</tr>
</tbody>
</table>

* The Designer shall modify items marked with an asterisk to describe the class of concrete, pier and/or abutment location, bridge number, SFN, maximum bearing
pressure and to correctly describe the “As Per Plan” bid item.

606.8 FOOTINGS

Provide the following note if the footing excavation is mainly bedrock and the footings are to be at an elevation no higher than plan elevation:

[36] FOOTINGS: Place footings in bedrock at the elevation shown.

Provide the following note where footings are to be founded in bedrock at an elevation no higher than plan elevation.

[37] FOOTINGS shall extend a minimum of 3 inches [75 mm]* into bedrock or to the elevation shown, whichever is lower.

Provide the following note where footings are to be founded in bedrock, and where the encountering of bedrock at an elevation considerably above plan elevation may make it desirable to raise the footing to an elevation not above the specified maximum in order to effect an appreciable saving:

[38] FOOTINGS shall extend a minimum of 3 inches [75 mm]* into bedrock. If necessary due to poor bedrock material, the footings should be lowered. If the low point of the bedrock surface occurs 2 feet [0.6 meters] or more above plan elevation, the final footing elevations may be raised, upon approval by the Director, but to an elevation not higher than ** feet [meters]. Stepping of individual footings will not be permitted unless shown on the plans.

* Shall be greater than 3 inches [75 mm] if required by design considerations.

** The maximum elevation allowed should assure that minimum soil cover over the footing is obtained; clearance from the superstructure to the finished ground elevation meets standards; quality of bedrock material at that elevation is adequate; and minimum embedment into the bedrock material will not be adversely affected.

606.9 DRILLED SHAFTS

Use the following drilled shaft notes when applicable for the specific project. Revise the note for the project conditions and the different drilled shaft designs, if any, on the project.

[38a] DRILLED SHAFTS
The design load to be supported by each drilled shaft is ______*____ tons [kN] at the abutments and ______*____ tons [kN] at the piers. This load is resisted by shaft adhesion within a portion of the bedrock socket and also by shaft end bearing. The allowable
bedrock socket adhesion is ___ ** tons [kN], assumed to act along the bottom ___ feet [meters] of the bedrock socket for the abutments and ___ feet [meters] of the bedrock socket for the piers. The allowable end bearing pressure is ___ tons per square foot [Mpa]. The reinforcing steel shall be epoxy coated according to 709.00.

* Complete the loads and dimensions in this note. Abutment and Pier sections of the note should be removed or revised as required.

## 607 MAINTENANCE OF TRAFFIC

Notes concerning maintenance of traffic often are required for bridge work, especially in phased construction projects. The designer is responsible for any bridge maintenance of traffic notes being coordinated with the project’s overall maintenance of traffic plans. Any phased construction lane widths, temporary or construction vertical and horizontal clearances, or construction access requirements must match requirements in project’s maintenance of traffic plans.

## 608 RAILROAD GRADE SEPARATION PROJECTS

### 608.1 CONSTRUCTION CLEARANCE

Obtain the actual dimensions used in the text of this note from the "Agreement" (a legal document signed by the Director and Railroad). To help limit project construction problems, validate those dimensions with the district railroad coordinator before the note is considered complete. Revise the note to define the agreed upon restraints, including items such as short term clearances, if different than the construction clearances; maximum period of time for restricted clearances; or other project specific controls.

[39] **CONSTRUCTION CLEARANCE:** Maintain a construction clearance of ____ feet [meters] horizontally from the center of tracks and ____ feet [meters] vertically from a point level with the top of the higher rail, and ____ feet [meters] from the center of tracks, at all times.

* The Designer shall fill in the dimensions.

### 608.2 RAILROAD AERIAL LINES

Modify the note below to match the specific requirements of the "Agreement" (a legal document signed by the Director and Railroad). Contact the District railroad coordinator to confirm whether the railroad will move, maintain, or re-construction their lines or other cable systems attached to the bridge or whether the note must specify this scope of work as part of the project.

[40] **RAILROAD AERIAL LINES** will be relocated by the Railroad. Use all precautions...
necessary to see that the lines are not disturbed during the construction stage and cooperate with the Railroad in the relocation of these lines. The cost of the relocation will be included in the railroad force account work.

608.3 RAILROAD STRUCTURAL STEEL

[40A] Note Retired - see appendix

609 UTILITY LINES

The District Utility Coordinator shall coordinate utilities. The District utilities coordinator shall be contacted for required notes. If existing utilities contain asbestos, or other hazardous materials, plan notes will be required to define what material, what utility line and where located. Designer shall assure any plan notes added are coordinated with the bid item descriptions to assure the Contractor properly bids the item. The following note should be used only if utilities are involved and as modified by the district utilities coordinator.

[41] UTILITY LINES: The Utility(ies) shall bore all expense involved in relocating (installing) the affected utility lines. The Contractor and Utility(ies) are to cooperate by arranging their work in such a manner that inconvenience to either will be held to a minimum.

610 REHABILITATION OF EXISTING STRUCTURES

610.1 EXISTING STRUCTURE VERIFICATION

Provide the following note on plans for altering, widening or repairing existing structures in order to qualify the plans and to ensure that the Contractor's obligation is clearly stated. The inclusion of this note in the plans does not relieve the designer of the requirements stated in Sections 401 and 405:

[42] EXISTING STRUCTURE VERIFICATION: Details and dimensions shown on these plans pertaining to the existing structure have been obtained from plans of the existing structure and from field observations and measurements. Consequently, they are indicative of the existing structure and the proposed work but they shall be considered tentative and approximate. The Contractor is referred to CMS Sections 102.05, 105.02 and *513.04.

Base contract bid prices upon a recognition of the uncertainties described above and upon a prebid examination of the existing structure. However, the Department will pay for all project work based upon actual details and dimensions that have been verified in the field.
600.2 REINFORCING STEEL REPLACEMENT

Place the following note in the plans where the preserved existing reinforcing steel which projects from the existing structure after partial removal is to be lapped with new reinforcing steel.

[43] ITEM 509 REINFORCING STEEL, REPLACEMENT OF EXISTING REINFORCING STEEL, AS PER PLAN: Replace all existing reinforcing bars deemed by the Engineer to be unusable because of corrosion. The Department will measure the replacement reinforcing steel by the number of pounds accepted in place.

Replace all existing reinforcing steel bars which are to be incorporated into the new work and are deemed by the Engineer to be made unusable by concrete removal operations with new epoxy coated reinforcing steel of the same size at no cost to the Department.

NOTE TO DESIGNER: Include a bid item as defined above with a specific weight of reinforcing steel.

On rehabilitation plans where new reinforcing steel may require field bending and cutting, use the following note. Clearly designate in the plans the bar marks to which this note applies.

[43a] ITEM 509 - EPOXY COATED REINFORCING STEEL, AS PER PLAN: In addition to the provisions of item 509, field bend and/or field cut the reinforcing steel designated in the plans, as necessary, in order to maintain the required clearances and bar spacings. Repair all damage to the epoxy coating, as a result of this work, according to 709.00.

600.3 REHABILITATION – STRUCTURAL STEEL

Use the following note on bridge rehabilitation projects where repair or replacement of members not designed to carry tension live loads (i.e. cross frames, bearing plates, etc.) consist of materials readily available from a structural warehouse (i.e. angles, channels, bars, etc.) and must be field fabricated to dimensions obtained in the field after contract letting. The recommended bid item quantity for rehabilitation work is in pounds [kilograms] rather than Lump Sum. The Designer should adequately define all steel members to be included in this pay item.

[44] ITEM 513 - STRUCTURAL STEEL MEMBERS, LEVEL UF, AS PER PLAN: All requirements of 513 apply to shop fabricated members. Perform work for field-fabricated members according to Item 513, except as modified herein. The Department will not require the contractor performing field fabrication to be pre-qualified as specified in Supplement 1078. Submit a written letter of material acceptance, 501.06, to the Engineer. Provide shop drawings according to 513.04 or supply the Engineer with “as
built” drawings meeting 513.04 after completion of field fabrication. The Engineer will review the submitted drawings for concurrence with the final as-built condition. If necessary, the Engineer may contact the Office of Structural Engineering for technical assistance. If the Engineer is satisfied with the “as-built” drawings and the delivered materials, supply a copy of the drawings, stamped and dated, along with microfilm, to the Structural, Welding and Metals Section of the Office of Material Management for record purposes.

The following members are included in this item: _____, _____ and _____.

[44a] Note retired - see appendix

610.4 REFURBISHED BEARINGS

When the following note is used, a separate plan note and pay quantity for jacking or temporary support of the superstructure is required. Revise this note, as appropriate, to describe the work for the type of bearing being refurbished.

[45] ITEM 516 - REFURBISHING BEARING DEVICES, AS PER PLAN: This item shall include all work necessary to properly align bridge bearings as well as their cleaning and painting. Included shall be the disassembly of the bearings, hand tool cleaning (grinding if necessary), painting according to Item 514, replacement of any damaged sheet lead with preformed bearing pads (711.21), installation of any necessary steel shims of the same size as the bearings to provide a snug fit, realignment of the upper bearing plate by removing existing welds and rewelding so that the bearings are vertically aligned at 60° [15EC], lubricating sliding surfaces, and reassembly of the bearings. Assure all bearings are shimmed adequately and that no beams and/or bearing devices are “floating”. At no additional cost to the State, the Contractor may install new bearings of the same type as the existing in place of refurbishing the bearings. All work shall be to the satisfaction of the Engineer. Payment for all of the above described labor and materials will be made at the contract price bid for Item 516 - Refurbish Bearing Devices, As Per Plan.

610.5 JACKING BRIDGE SUPERSTRUCTURES

Use the following note, modified as necessary, where jacking and/or temporary support of the existing superstructure is required. Modifications to this note are often not being performed by the designers. Use of this note without a review of the project may add unnecessary requirements to the jacking process or, in reverse, not be restrictive enough. Designers are again cautioned to appropriately review this note before incorporation into a set of plans.

[46] ITEM 516, JACKING AND TEMPORARY SUPPORT OF SUPERSTRUCTURE, AS PER PLAN:

This work consists of raising or re-positioning existing structures to the
dimensions and requirements defined in the project plans.

Submit construction plans in accordance with CMS 501.05.

If, during the jacking operations, cracking of the concrete superstructure, separation of the concrete deck from the steel stringers, or other damage to the structure is visually observed, immediately cease the jacking operation and install supports to the satisfaction of the Engineer. Analyze the damage and submit a method of correction to the Engineer for approval. Epoxy inject all beams that separate from the deck for the distance of the separation in accordance with CMS 512.07. The Department will not pay for the cost of this epoxy injection or other required repairs. The bridge bearings shall be fully seated at all contact areas. If full seating is not attained, submit a repair plan to the Engineer. The Department will not pay for the repair costs to ensure full seating on bearings.

The Department will measure this work on a lump sum basis.

The Department will pay for the accepted quantities at the contract price for Item 516, Jacking and Temporary Support of Superstructure, As Per Plan.

610.6 FATIGUE MEMBER INSPECTION

When re-decking a continuous beam bridge containing top flange fillet-welded cover plates and/or field butt-welded beams, provide the following note to facilitate the Engineer's inspection of the welded connections.

INSPECTION OF EXISTING STRUCTURAL STEEL: The Engineer will visually inspect all existing butt-welded splices and/or top flange cover plate fillet welds to ensure the welds, plates and beams or girders are free of defects and cracks. If necessary, remove all deck slab haunch forms immediately adjacent to such welds that may interfere with the Engineer’s inspection. The inspection will not take place until the top flanges are cleaned according to 511.10, but it will be done before the deck slab reinforcement is installed. The Department will pay for the cost associated with this inspection with Item 511, Superstructure Concrete. The Engineer will report all cracks found to the Office of Construction Administration, Bridge Construction Specialist, along with specific information on location of the cracks, length, and depth so an evaluation and repair or replacement recommendation can be made.
610.7 RAILING

Use the following note where the existing parapet is to be refaced. Modify the note accordingly for each specific project.

[48] ITEM 517 - RAILING FACED, AS PER PLAN

DESCRIPTION: This work consists of facing curb style parapets, using cast in place concrete, to obtain the deflector shape as shown in the plans.

REMOVAL: Carefully remove the existing aluminum railing, posts, curb plates, existing concrete curb and bulb angle gutter. Remove all loose or unsound concrete. Remove sound concrete, as necessary, to obtain a minimum 4 inch [100 mm] thickness of new concrete.

NOTE TO DESIGNER: Modify the list of items in the above removal portion of this note as necessary to fit the actual conditions of your particular project.

DOWEL HOLES AND REINFORCING STEEL: Drill dowel holes where shown in the plans. Install reinforcing steel according to Item 510 using epoxy grout, 705.20. Prior to drilling dowel holes, locate all existing reinforcing steel bars in the area of the hole with the aid of a reinforcing steel bar locator (pachometer). If an existing bar is encountered at the same location as a proposed dowel hole, move the dowel hole to either side of the existing bar. The Department will pay for all reinforcing steel, dowel holes and grouting with Item 517.

SURFACE PREPARATION: Thoroughly clean the parapet surface in contact with the refacing with detergent to remove surface contaminants. After detergent cleaning and within 24 hours of placing concrete, blast clean and air broom or power sweep all surfaces in contact with the refacing to remove all spalls, laitance, curing compounds, concrete sealers and other contaminants detrimental to the achievement of an adequate bond. Acceptable blast cleaning methods are high-pressure water blasting with or without abrasives in water, abrasive blasting with containment or vacuum abrasive blasting. Use hand tools as necessary to remove scale from any exposed reinforcing steel. Materials: Concrete shall be Class *(S or HP) with a compressive strength of 4500 psi [31 MPa]. Furnish reinforcing steel according to 709.00, grade 60 [420], with a minimum yield strength of 60,000 psi [420 MPa].

CONTROL JOINTS: Sawcut 1 1/4 inch [32 mm] deep control joints along the perimeter of the parapet as soon as the saw can be operated without damaging the concrete. Place the joint saw cuts at the same location as the existing deflection joints. Use an edge guide, fence or jig to ensure that the cut joint is straight, true and aligned on all faces of the parapet. The joint width shall be the width of the saw blade, a nominal width of 1/4 inch [6 mm]. Seal the perimeter of the control joint to a minimum depth of one inch [25
mm] with a polyurethane or polymeric material conforming to ASTM C920, Type S. Leave the bottom one-half inch [12 mm] of both the inside and outside faces of the parapet unsealed to allow any water which may enter the joint to escape.

METHOD OF MEASUREMENT: The Department will measure this item in feet by the actual length of railing faced between the ends of the existing concrete parapet.

BASIS OF PAYMENT: Payment for this item includes all costs of removal, dowel holes, reinforcing steel, concrete, shrinkage control joints, epoxy injection and inspection platforms. The Department will pay for accepted quantities at the contract price for Item 517, Railing Faced, As Per Plan.

NOTE TO DESIGNER: Include the reinforcing steel in the bar list with appropriate bending diagrams, as necessary, even though the reinforcing steel is included with item 517 for payment. Modify the method of measurement and items of work included in this pay item as necessary to fit your specific project.

611 MISCELLANEOUS GENERAL NOTES

611.1 DOWEL HOLES

[49] Note Retired - See appendix

611.2 APPROACH SLABS

[50] Note retired - see appendix

[50A] Note Retired - See appendix

Item 526, Reinforced Concrete Approach Slabs was developed such that the concrete used in the superstructure would also be used for the approach slabs. The new supplemental specification for QC/QA concrete is not included in Item 526.

Provide both of the following notes on projects that specify SS898, QC/QA Concrete for Structures:

[93A] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (APPROACH SLAB), AS PER PLAN

Furnish approach slabs conforming to CMS 526 except concrete shall be in accordance with Supplemental Specification 898, QC/QA Concrete, Class QSC2. The accepted quantities shall include: concrete, curbs, reinforcing steel, joint fillers, joint sealers, joint seals, and waterproofing. The Department will measure approach slabs by the number of square yards. The Department will initially pay the full bid price to the Contractor upon
completing the work. The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.

[93B] ITEM 898 - QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (DECK), AS PER PLAN
The Department will calculate the final adjusted payment according to 898.17 and include approach slab concrete and deck concrete in the same lot to determine final pay factors.
INTEGRAL AND SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEALS

A neoprene sheet is required for waterproofing of the backside of the joint between the integral backwall and the bridge seat. Include the following note, which contains criteria for the installation of this seal, for all integral and semi-integral abutments. Plan details will be required to show location and dimensional position for installation.

[51] ITEM 516 SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEAL, AS PER PLAN: Install a 3 foot [1 meter] wide neoprene sheet at locations shown in the plans. Secure the neoprene sheeting to the concrete with 1 1/4" [32 mm] x #10 gage [3 mm] (length x shank diameter) galvanized button head spikes through a 1 inch [25 mm] outside diameter, #10 gage [3 mm] galvanized washer. Maximum fastener spacing is 9 inches [225 mm]. Use of other similar galvanized devices, which will not damage either the neoprene or the concrete, will be subject to the approval of the Engineer.

Center the neoprene strips on all joints. For horizontal joints, secure the horizontal neoprene strip by using a single line of fasteners, starting at 6 inches [150 mm], +/-, from the top of the neoprene strip. For the vertical joints secure the vertical neoprene strip by using a single vertical line of fasteners, starting at 6 inches [150 mm], +/-, from the vertical edge of the neoprene strip nearest to the centerline of roadway. For vertical joints, install 2 additional fasteners at 6 inches [150 mm], center to center, across the top of the neoprene strip on the same side of the vertical joint as the single vertical row of fasteners is located.

The vertical neoprene strips shall completely overlap the horizontal strips. Lap lengths of the horizontal strips that are not vulcanized or adhesive bonded, shall be at least 1 foot [0.3 meter] in length, or 6 inches [150 mm] in length if the lap is vulcanized or adhesive bonded. No laps are acceptable in vertically installed neoprene strips.

The neoprene sheeting shall be 3/32" [2.5 mm] thick general purpose, heavy-duty neoprene sheet with nylon fabric reinforcement. The sheeting shall be “Fairprene Number NN-0003”, by E. I. Dupont De Nemours and Company, Inc., “Wingprene” by the Goodyear Tire and Rubber Company, or an approved alternate. The neoprene sheeting shall conform to the following:

<table>
<thead>
<tr>
<th>Description of Test</th>
<th>ASTM Method</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, inches [mm]</td>
<td>D751</td>
<td>0.094 ± 0.01</td>
</tr>
<tr>
<td>Breaking Strength, Grab, lbs [N], minimum (Long. X Trans.)</td>
<td>D751</td>
<td>700 x 700</td>
</tr>
<tr>
<td>Adhesive Strip, 1&quot; [25 mm] wide x 2&quot; [50mm] long, lbs [N] minimum</td>
<td>D751</td>
<td>9 [27]</td>
</tr>
<tr>
<td>Burst Strength, psi [Mpa] minimum</td>
<td>D751</td>
<td>1400 [9.65]</td>
</tr>
</tbody>
</table>
Heat Aging, 70 Hr, 212 °F [100 °C], 180° D2136 No cracking of bend without cracking coating
Low temp. brittleness, 1 Hr, ! 40 °F D2136 No cracking of ![40^oC] coating mandrel

METHOD OF MEASUREMENT: The Department will measure the total length of joint to be sealed by the number of feet.

BASIS OF PAYMENT: The Department will pay for accepted quantities at the contract price for Item 516, Semi-Integral Abutment Expansion Joint Seal, As Per Plan.

NOTE TO DESIGNER: Change “semi-integral” to “integral” as appropriate.

611.4 BACKWALL DRAINAGE

[52] Note retired - see appendix

[53] Note retired - see appendix

611.5 CONCRETE PARAPET SAWCUT JOINTS

Include the following note in the Structural General Notes when a concrete parapet or railing is used and standard drawings do not cover the below requirements.

[54] CONCRETE PARAPETS: As soon as a concrete saw can be operated without damaging the freshly placed concrete, sawcut 1 1/4" [32 mm] deep control joints into the perimeter of the concrete parapet starting and ending at the elevation of the concrete deck. Place the sawcuts at a minimum of 6 feet [2 meter] and a maximum of 10 feet [3 meter] centers. Use an edge guide, fence, or jig to ensure that the cut joint is straight, true, and aligned on all faces of the parapet. The joint width shall be the width of the saw blade, a nominal width of 1/4 inch [32 mm]. Seal the perimeter of the deflection control joint to a minimum depth of 1 inch [25 mm] with a polyurethane or polymeric material conforming to ASTM C920, Type S. Leave the bottom ½ inch [13 mm] of the inside and outside face unsealed to allow water to escape.

611.6 BEARING PAD SHIMS, PRESTRESSED

Add the following note to ensure proper seating of prestressed concrete box beams for skewed bridges.

[55] BEARING PAD SHIMS: Place 1/8" [3 mm] thick preformed bearing pad shims, plan area ___ inches by ___ inches, under the elastomeric bearing pads where required for
proper bearing. Furnish two shims per beam. The Department will measure this item by the total number supplied. The Department will pay for accepted quantities at the contract price for Item 516 - 1/8" [3 mm] Preformed Bearing Pads. Any unused shims will become the property of the State.

NOTE TO DESIGNER: The plan area of the shim pad shall be the same as the elastomeric bearing.

611.7 CLEANING STEEL IN PATCHES

Use this note with all concrete patching bid items that refer to the cleaning requirements specified in 519.04

[55a] ITEM 519 - PATCHING CONCRETE STRUCTURES, AS PER PLAN: Prior to the surface cleaning specified in 519.04 and within 24 hours of placing patching material, blast clean all surfaces to be patched including the exposed reinforcing steel. Acceptable methods include high-pressure water blasting with or without abrasives in the water, abrasive blasting with containment, or vacuum abrasive blasting.

611.8 CONVERSION OF STANDARD BRIDGE DRAWINGS

The Department's Standard Bridge Drawings are available in English units only. If the project scope has established that Contract Documents will be prepared in metric units, the Designer has two options:

A. Convert the English drawings to metric units. This means removing the standard title blocks and using the converted drawings as plan insert sheets directly in the project plans. In doing so, the Designer assumes responsibility for the accuracy to the converted drawings. CADD files may be downloaded from the ODOT, Office of Structural Engineering web site.

B. Specify the English Standard Bridge Drawings in the Plans and use the following note:

[55b] CONVERSION OF STANDARD BRIDGE DRAWINGS: The Standard Bridge Drawings referenced in this plan are in English units. Any conversion of dimensions required to construct the items shown on the standards is the responsibility of the Contractor. Refer to 109.02 for a listing of Conversion Factors. Conversions shall be appropriately precise and shall reflect standard industry metric values where suitable.

611.9 COFFERDAMS AND EXCAVATION BRACING

Use this note when the plans include detail designs for temporary shoring.
ITEM 503, COFFERDAMS AND EXCAVATION BRACING, AS PER PLAN:
The design shown on the plans for temporary support of excavation is one representative
design that may be used to construct the project. The Contractor may construct the
design shown on the plans or prepare an alternate design to support the sides of
excavations. If constructing an alternate design for temporary support of excavation,
prepare and provide plans in accordance with C&MS 501.05. The Department will pay
for the temporary support of excavation at the contract lump sum price for Cofferdams
and Excavation Bracing. No additional payment will be made for providing an alternate
design.

611.10 DECK PLACEMENT NOTES

611.10.1 FALSEWORK AND FORMS

Use the following note when web depths greater than 84 in. are specified.

ITEM 511, CLASS HP CONCRETE, SUPERSTRUCTURE, AS PER PLAN *

Locate the lower contact point of the overhang falsework at least ** inches ± 2
in. above the top of the girder’s bottom flange. The bracket contact point location
requirements of C&MS 508 do not apply.

NOTE TO DESIGNER:
* Modify the pay item description to fit the specific project requirements.
** The minimum dimension for the location for the lower point of contact should be 76 in.
   below the bottom of the top flange. Designers should verify the acceptability of the design
   within the range of tolerance specified.

611.10.2 DECK PLACEMENT DESIGN ASSUMPTIONS

Use the following note on all projects requiring mechanized finishing machines to place deck
concrete.

DECK PLACEMENT DESIGN ASSUMPTIONS:
The following assumptions of construction means and methods were made for the
analysis and design of the superstructure. The Contractor is responsible for the
design of the falsework support system within these parameters and will assume
responsibility for superstructure analysis for deviation from these design
assumptions.

An eight wheel finishing machine with a maximum wheel load of _____ kips for
a total machine load of _____ kips.

A minimum out-to-out wheel spacing at each end of the machine of 103”.
A maximum spacing of overhang falsework brackets of 48 in.
A maximum distance from the centerline of the fascia girder to the face of the safety handrail of 65”.

NOTE TO DESIGNER:
Refer to BDM Section 302.2.7.2.c for design information regarding finishing machine loads.
SECTION 700 – TYPICAL DETAIL NOTES

701 SUBSTRUCTURE DETAILS

701.1 STEEL SHEET PILING

Place the following note on the substructure or retaining wall sheet with the details of steel sheet piling that is to be left in place.

[56] STEEL SHEET PILING left in place shall have a minimum section modulus of ________ in$^3$ per foot of wall.

[56M] STEEL SHEET PILING left in place shall have a minimum section modulus of ________ mm$^3$ per meter of wall.

701.2 POROUS BACKFILL

Provide the following porous backfill note on the appropriate detail sheets.

[57] POROUS BACKFILL WITH FILTER FABRIC, 2 feet [0.6 meter] thick shall extend up to the plane of the subgrade, to 1 foot [0.3 meter] below the embankment surface, and laterally to the ends of the wingwalls.

For use when weep holes are specified:

[58] POROUS BACKFILL WITH FILTER FABRIC, 2 feet [0.6 meter] thick shall extend up to the plane of the subgrade, to 1 foot [0.3 meter] below the embankment surface, and laterally to the ends of the wingwalls. Place two cubic feet [0.06 cubic meter] of bagged No. 3 aggregate at each weephole. The Department will include bagged aggregate with porous backfill for payment.

701.3 BRIDGE SEAT REINFORCING

For structures that contain bearing anchors, place one of the two following notes on an appropriate abutment or pier detail sheet near the "Bearing Anchor Plan". Where the Contractor is allowed the option of presetting bearing anchors (or formed holes), or of drilling bearing anchor holes, provide the first note. Where drilling of anchors into the bridge seat is required, provide the second note. (Formed holes are not practical for prestressed concrete box beam bridges.)

[59] BRIDGE SEAT REINFORCING, SETTING ANCHORS: Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the drilling of bearing anchor holes.
holes or the pre-setting of bearing anchors.

[60] **BRIDGE SEAT REINFORCING, SETTING ANCHORS:** Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the drilling of anchor bar holes.

### 701.4 BRIDGE SEAT ELEVATIONS FOR ELASTOMERIC BEARINGS

Where bridge seats have been adjusted to compensate for the vertical deformation of elastomeric bearings, place the following note with the necessary modifications on the appropriate substructure detail sheet.

[61] **BRIDGE SEAT ELEVATIONS** have been adjusted upward _____ inches [mm] at abutments and _____ inches [mm] at piers to compensate for the vertical deformation of the bearings.

### 701.5 PROPER SEATING OF STEEL BEAMS AT ABUTMENTS

For a structure with concrete backwalls, deck joints and concrete decks supported on beams or girders, show an optional backwall construction joint at the level of the approach slab seat and provide the following note either on the appropriate abutment detail sheet or in the General Notes.

[62] **BACKWALL CONCRETE:** In addition to 511.10, do not place backwall concrete above the optional construction joint at the approach slab seat until after the deck concrete in the span adjacent to the abutment has been placed.

For a steel beam bridge with concrete backwalls and sealed deck joints employing superstructure support or armor steel of considerable stiffness where there is a possibility of individual beams being lifted off of their bearings in a clamping operation, a note similar to the following shall be provided:

[63] **INSTALLATION OF SEAL:** During installation of the support/armor for the superstructure side of the expansion joint seal, observe the seating of beams on bearings to assure that positive bearing is maintained.

### 701.6 BACKWALL CONCRETE PLACEMENT FOR PRESTRESSED BOX BEAMS

For prestressed concrete box beam bridges where the placement of the wingwall concrete above the bridge seat needs to occur after the beams have been erected to allow for the tolerances of the beam fit-up and for beam erection clearances, provide the following note:
ABUTMENT CONCRETE: Do not place the abutment concrete above the bridge seat construction joint until the prestressed concrete box beams have been erected.

SECTION 701.7 SEALING OF BEAM SEATS

Provide the following note when elastomeric bearings are to be placed on substructures with beam seats sealed with an epoxy or non-epoxy sealer:

SEALING OF BEAM SEATS: If the beam seats are sealed with an epoxy or non-epoxy sealer prior to setting the bearings, do not apply sealer to the concrete surfaces under the proposed bearing locations. If these locations are sealed, remove the sealer to the satisfaction of the Engineer prior to setting the bearings. The Department will not pay for this removal.

SECTION 702 SUPERSTRUCTURE DETAILS

702.1 STEEL BEAM DEFLECTION AND CAMBER

For steel beam or built-up girder bridges provide a table similar to Figure 701 on a structural steel detail sheet. Tabulation is required regardless of the amount of deflection and is required for all beams or girders, if the deflection is different.

Show the deflection and camber data as described in Section 302.4.1.8. The table is to include bearing points, quarter points, center of span, splice points, and maximum 30 foot [10.0 meter] increments. Unique geometry may require an even closer spacing.

702.2 STEEL NOTCH TOUGHNESS REQUIREMENT (CHARPY V-NOTCH)

CVN material is a requirement to help assure fracture toughness of main material. Designers using this note should understand not only why CVN is specified but what is a main member. Section 302.4.1.10 helps with the definition of main members and specially highlights that crossframes of curved steel structures, because they are actual designed members carrying liveload forces, are also main members. Designers are reminded they must indicate specific pieces, members, shapes, etc. that are main members.

Place the following note on a structural steel detail sheet for bridges having main load-carrying members that must meet minimum notch toughness requirements:

CVN: Where a shape or plate is designated (CVN), furnish material that meets the minimum notch toughness requirements as specified in 711.01.
702.3  HIGH STRENGTH BOLTS

For all structural steel superstructures, place the following note on the structural detail sheet:

[66] HIGH STRENGTH BOLTS shall be ___________ diameter A325 unless otherwise noted.

[67] Note retired - see appendix

702.4  SCUPPERS

[68] Note retired - see appendix

702.5  ELASTOMERIC BEARING LOAD PLATE

Where the load plate of an elastomeric bearing is to be connected to the structure by welding, provide the following note with the pertinent bearing details:

[69] WELDING: Control welding so that the plate temperature at the elastomer bonded surface does not exceed 300°F [150°C] as determined by use of pyrometric sticks or other temperature monitoring devices.

702.6  BEARING REPOSITIONING

Where elastomeric bearing repositioning is required for a steel beam or girder superstructure, provide the following plan note.

[70] BEARING REPOSITIONING: If the steel is erected at an ambient temperature higher than 80°F [26°C] or lower than 40°F [4°C] and the bearing shear deflection exceeds 1/6 of the bearing height at 60°F (+/-) 10°F [15°C +/- 5°C], raise the beams or girders to allow the bearings to return to their undeformed shape at 60°F (+/-) 10°F [15°C +/- 5°C].

702.7  CONCRETE PLACEMENT SEQUENCE NOTES

Also see section 701.5 notes.

702.7.1  CONCRETE INTERMEDIATE DIAPHRAGM FOR PRESTRESSED CONCRETE I-BEAMS

If the design plans do not reference Standard Bridge Drawing PSID-1-99, provide the following note.
INTERMEDIATE DIAPHRAGMS: Do not place the deck concrete until all intermediate diaphragms have been properly installed. If concrete diaphragms are used, complete the installation of the intermediate diaphragms at least 48 hours before deck placement begins. Concrete shall be Class S.

702.7.2 SEMI-INTEGRAL OR INTEGRAL ABUTMENT CONCRETE PLACEMENT FOR DIAPHRAGMS

Hardened concrete end diaphragms restrain the movement and rotation of beam/girder ends that occur during deck placement. This restraint will increase stress in both the beam/girder and diaphragm. Factors that can contribute to detrimental stress increases include large structure skew and phased construction. When these factors exist, hardened diaphragms should be avoided during the deck placement. The following table provides guidelines for concrete diaphragm placement options.

<table>
<thead>
<tr>
<th>Diaphragm Placement (3)</th>
<th>Skew &lt; 30° (Steel) or &lt; 10° (1-beam)</th>
<th>Skew ≥ 30° (Steel) or ≥ 10° (1-beam)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase 1</td>
<td>Placed 48 hrs before deck placement</td>
<td>✔️ ✔️</td>
</tr>
<tr>
<td></td>
<td>Placed with deck placement</td>
<td>✔️ ✔️ (1)</td>
</tr>
<tr>
<td></td>
<td>Placed after deck placement</td>
<td>✔️ ✔️</td>
</tr>
<tr>
<td>Successive Phases</td>
<td>Placed 48 hrs before deck placement</td>
<td>✔️ ✔️</td>
</tr>
<tr>
<td></td>
<td>Placed with deck placement</td>
<td>✔️ ✔️ (1)</td>
</tr>
<tr>
<td></td>
<td>Placed after deck placement</td>
<td>✔️ ✔️</td>
</tr>
</tbody>
</table>

Notes:

 ✔️ = Diaphragm concrete placement option is acceptable
1. Requires submittal and Engineer approval
2. Place closure pour concrete in the diaphragm with or after deck placement.
3. For bridges built without phased construction, follow guidance for Phase 1 with No Closure Pour.
Designers should consider the absence of restraint at the diaphragm location and when calculating the unbraced length of beam/girder flanges. If necessary, temporary bracing details should be included in the plans. Temporary end bracing should be oriented perpendicular to beam/girder webs.

The following notes may be needed depending on whether the bridge superstructure is steel or prestressed concrete; requires phased construction; or is skewed a specific amount. Use the following note for either steel superstructures skewed less than 30 degrees or I-beam superstructures skewed less than 10 degrees without phased construction.

[71a] ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate the horizontal construction joint between the diaphragm and deck concrete at the approach slab seat.

Use the following note for either steel superstructures skewed 30 degrees or more or I-beam superstructures skewed less than 10 degrees without phased construction.

[71b] ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set.

Use the following note for either steel superstructures skewed less than 30 degrees or I-beam superstructures skewed less than 10 degrees with phased construction and closure pours required per BDM Section 302.2.9.

[71c] ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate the horizontal construction joint between the diaphragm and deck concrete at the approach slab seat. Place closure pour concrete in the diaphragm and deck concurrently.

NOTE TO DESIGNER:
For bridges with phased construction that do not require closure pours according to BDM Section 302.2.9, omit the last sentence of note [71c]

Use the following note for either steel superstructures skewed 30 degrees or more or I-beam superstructures skewed 10 degrees or more with phased construction and closure pours required per BDM Section 302.2.9.

[71d] ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase after the
deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set. Place closure pour concrete in the diaphragm and deck concurrently.

NOTE TO DESIGNER:
For bridges with phased construction that do not require closure pours according to BDM Section 302.2.9, omit the last sentence of note [71d]

702.8 CONCRETE DECK SLAB DEPTH AND PAY QUANTITIES

For all steel beam and girder bridges with a concrete deck, provide the following note that describes how the quantity of deck concrete was calculated.

DECK SLAB CONCRETE QUANTITY: The estimated quantity of deck slab concrete is based on the constant deck slab thickness, as shown, plus the quantity of concrete that forms each beam/girder haunch. The estimate assumes a constant haunch thickness of inches [mm] and a constant haunch width outside the edge of each beam/girder flange of 9 inches [230 mm]. Deviate from this haunch thickness as necessary to place the deck surface at the finished grade. The allowable tolerance for the haunch width outside the edge of each beam/girder flange is ± 3 inches [75 mm].

The haunch thickness was measured at the centerline of the beam/girder, from the surface of the deck to the bottom of the top flange minus the deck slab thickness. The area of all embedded steel plates has been deducted from the haunch quantity in accordance with 511.24.

NOTE TO DESIGNER: The note above applies to new structures with beams/girders placed parallel to the profile grade line. A constant depth haunch may not be practical for existing structures or new structures whose beams/girders are not placed parallel to the profile grade line. In these special cases, the note shall be modified to fit the exact conditions.

Note retired – see appendix
702.9  CONCRETE DECK HAUNCH WIDTHS

Note retired - see appendix

702.10  PRESTRESSED CONCRETE I-BEAM BRIDGES

For prestressed concrete I-beam bridges with concrete deck, compute the concrete topping depth over the top of the beams as follows:

A = Design slab thickness.
B = Anticipated total midspan camber due to the design prestressing force at time of release
C = Deflection at midspan due to the self weight of the beam
D = Deflection at midspan due to dead load of the slab, diaphragms and other non-composite loads.
E = Deflection due at midspan to dead load of railing, sidewalk and other composite dead loads not including future wearing surface
F = Adjustment for vertical curve. Positive for crest vertical curves
G = Sacrificial haunch depth (2" [50 mm])
H = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E - F + G. (If F > 1.8 B - 1.85C - (D+E) then H = A + G)
I = Total Topping Thickness at mid-span = A + G. If F > 1.8B - 1.85C - (D+E) then I = A - (1.8B - 1.85C) + D + E + F + G

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section.

Show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness including the design slab thickness and the haunch thickness at the centerline of spans and bearings. Also show screed elevation tables similar to 702.16.2. Provide the following note in the plans:

[97]  Calculated camber at the time of release is _____ inches [mm].

Calculated camber at the time of erection is _____ inches [mm].

Calculated long-term camber is _____ inches [mm].

NOTE TO DESIGNER: The camber at the time of release is (B-C), the camber at the time of erection is (1.8B – 1.85C), and the long-term camber is (2.45B- 2.40C).

[75]  DECK SLAB THICKNESS FOR CONCRETE QUANTITY : The Topping thicknesses shown from the top of the deck slab to the top of the top flange along the centerline of the I-beam are theoretical dimensions. The haunch depth is the Topping thickness minus the
design slab thickness. The Department will pay for superstructure concrete based on the
design slab thickness and the average of the theoretical haunch depths at mid-span and at
each beam bearing even though deviation from the dimensions shown may be necessary
to place the deck surface at the finished grade. Once all beams are set in their final
position, the actual camber for each member will be the top of beam elevation at mid-
span minus the average top of beam elevation at each bearing. The actual Topping
thickness at mid-span will be the theoretical dimension plus or minus the difference
between the actual and anticipated camber.
Use the following note when the length of the I-beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8" [10mm]:

[94] NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a *, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the shop drawings.

NOTE TO DESIGNER: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

702.11 PRESTRESSED CONCRETE BOX BEAM BRIDGE

For prestressed concrete box beam bridges, the asphalt or concrete topping depth over the top of the beams shall be computed as follows:

A = Minimum topping thickness
B = Anticipated total midspan camber due to the design prestressing force at time of release
C = Deflection due to the self weight of the beam (including diaphragms)
D = Deflection due to dead load of the topping and other non-composite loads
E = Deflection due to dead load of railing, sidewalk and other composite dead loads not including future wearing surface
F = Adjustment for vertical curve. Positive for crest vertical curves
G = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E - F. If F > 1.8B - 1.85C - (D + E) then G = A
H = Total Topping Thickness at mid-span = A
If F > 1.8B - 1.85C - (D + E) then H = A - (1.8B - 1.85C) + D + E + F

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section when designing a composite box beam otherwise use the non-composite section. Note that with the exception of when F > 1.8B - 1.85C - (D + E), the dead load deflection adjustment (D + E) is made by adjusting the beam seat elevations upward.

For non-composite prestressed concrete box beam bridges with an asphaltic concrete surface course provide a note similar to [76]. For composite prestressed concrete box beam bridges with a concrete surface course provide a note similar to [76a].

[76] Calculated camber at the time of release is ______ inches [mm].

Calculated camber at time of paving is ______ inches [mm].

Long term camber is ______ inches [mm].
Calculated deflection due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is ____ inches [mm].

The vertical curve adjustment to the topping thickness at midspan is ____ inches [mm] upward.

The vertical curve adjustment to the topping thickness at each bearing is ____ inches [mm] upward/downward.

(1) The thickness of the intermediate asphalt course shall be 1½ inches [38 mm]. No variation in thickness is required.

(2) The thickness of the intermediate asphalt course shall vary from 1½ inches [38 mm] at each centerline of beam bearing to ____ inches [mm] at midspan.

(3) The thickness of the intermediate asphalt course shall vary from ____ inches [mm] at each centerline of beam bearing to 1½ inches [38 mm] at midspan.

[76a] Calculated camber at the time of release is ______ inches [mm].

Calculated camber at time of paving is ______ inches [mm].

Long term camber is ______ inches [mm].

Calculated deflection due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is ____ inches [mm].

The vertical curve adjustment to the topping thickness at midspan is ____ inches [mm] upward.

The vertical curve adjustment to the topping thickness at each bearing is ____ inches [mm] upward/downward.

(1) The concrete thickness shall be 6 inches [150 mm]. No variation in thickness of concrete is required.

(2) The concrete thickness shall vary from 6 inches [150 mm] at each centerline of beam bearing to ____ inches [mm] at midspan.

(3) The concrete thickness shall vary from ____ inches [mm] at each centerline of beam bearing to 6 inches [150 mm] at midspan.

NOTE TO DESIGNER: The calculated camber at the time of release is (B – C), at the time of paving is (1.8B - 1.85C), and long term is (2.45B – 2.40C). The calculated deflection due to dead load applied after the beams are set is (D + E). The vertical curve adjustment at midspan is (F) when F > 1.8B - 1.85C - D - E. The vertical curve adjustment at each bearing is (F) when F
< 1.8B - 1.85C - D - E and may be upward for sag curves or downward for crest curves. Remove the reference to the vertical curve adjustment that does not apply.

Conclude note [76] with note (1), (2) or (3) as appropriate. Note (1) should be used when after placement of the topping, the top surface of the beam parallels the profile grade. Note (2) should be used when \( F > 1.8B - 1.85C - D - E \). Note (3) should be used for all other cases.

For non-composite designs, include in the bridge plans a diagram similar to Figure 702 showing the thickness of the Item 448 Intermediate course and the Item 448 surface course at each centerline of bearing and at midspan.

For composite design, show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness at each centerline of bearings and at midspan. Also show screed elevation tables similar to 701.1.

Use the following note when the length of the box beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8" [10mm]:

[95] NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a *, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the shop drawings.

NOTE TO DESIGNER: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

702.12 ASPHALT CONCRETE WEARING COURSE

Place note [77] on the plans for prestressed concrete box beam bridges having an asphalt concrete wearing course. If the nominal thickness of 448 varies from the 1½" [38 mm] shown, revise the note accordingly.

While this note specifies how to place only the two 448 bid items, the designer should recognize that two Item 407 tack coat items are also required. One tack coat is applied before the intermediate asphalt concrete course. The other tack coat is applied between the intermediate and surface course.

[77] ASPHALT CONCRETE WEARING COURSE shall consist of a variable thickness of 448 asphalt concrete intermediate course, Type 2, PG64-28 and a 1½" [38 mm] thickness of 448 asphalt concrete surface course, Type 1H. Place the 448 intermediate course in two operations. The first portion of the course shall be of 1½" [38 mm] uniform thickness. Feather the second portion of the course to place the surface parallel to and 1 ½" [38 mm] below final pavement surface elevation.
702.13 PAINTING OF A588/A709 GRADE 50 STEEL

[78] Note retired - see appendix

Provide the following note for bridge superstructures using unpainted A588/A709 Grade 50W steel and having deck expansion joints at the abutments. Modify the note accordingly for structures with intermediate expansion joints. Bridges with an integral or semi-integral type abutment will not require painting of the beam ends.

[79] PARTIAL PAINTING OF A709 GRADE 50W STEEL: Paint the last 10 ft [3 m] of each beam/girder end adjacent to the abutments including all cross frames and other steel within these limits. The prime coat shall be 708.01. The top coat color shall closely approach Federal Standard No. 595B - 20045 or 20059 (the color of weathering steel).

702.14 ERECTION BOLTS

Where erection bolts are specified for attaching crossframes on steel girder or rolled beam bridges, and the expected dead load differential deflection at each end of the crossframes is less than or equal to ½" [13 mm] provide the following note. (Do not use the note if standard drawing GSD-1-96 is being referenced.)

[80] ERECTION BOLTS: The hole diameter in the cross frames and girder stiffeners shall be 3/16" [4 mm] larger than the diameter of the erection bolts. Erection bolts shall be high strength bolts and shall remain in place. Supply two hardened washers with each high strength bolt. Fully torque the bolts or use a lock washer in addition to the two hardened washers. Furnish erection bolts as part of Item 513.

[81] Note Retired – See Appendix
702.15  WELDED ATTACHMENTS

Provide the following note on plans for steel beam or girder bridges:

[82]  WELD ATTACHMENT of supports for concrete deck finishing machine to areas of the fascia stringer flanges designated "Compression". Do not weld attachments to areas designated "Tension". Fillet welds to compression flanges shall be at least 1" [25 mm] from edge of flange, be no more than 2" [50 mm] long, and be at least 1/4" [6 mm] for thicknesses up to 3/4" [19 mm] or 5/16" [8 mm] for greater than 3/4" [19 mm] thick.

702.16  DECK ELEVATION TABLES

702.16.1  SCREED ELEVATION TABLES

Screed elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks. Screed elevations are not required for slab bridges. The general criteria for screed elevation tables are defined in BDM Section 302.2.3. Refer to Figure 703 for an example screed table for structural steel members and Figure 704 for an example screed table for composite box beams. In lieu of a table format, the designer may supply screed elevations through the use of a deck plan view showing elevations and stations of the points required in BDM Section 302.2.3.

In addition to the screed elevation table or diagram, provide a screed elevation note similar to the one below to define the elevations that are given. The screed elevation locations should be identified on the transverse section.

[104]  SCREED ELEVATIONS shown represent the theoretical deck surface location prior to deflections caused by deck placement and other anticipated dead loads.

702.16.2  TOP OF HAUNCH ELEVATION TABLES

Top of haunch elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam and other superstructure types requiring deck falsework. Top of haunch elevations are not required for slab bridges. The general criteria for top of haunch elevation tables are defined in BDM Section 302.2.3.

In addition to the top of haunch elevation table, provide a top of haunch elevation note similar to the one below to define the elevations that are given. The top of haunch elevation locations should be identified on the transverse section.

[105]  TOP OF HAUNCH ELEVATIONS shown represent the theoretical location of the
bottom of the deck above the beam/girder haunch prior to deflections caused by deck placement and other anticipated dead loads.

702.16.3  **FINAL DECK SURFACE ELEVATION TABLES**

Final deck surface elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks including slab bridges. The general criteria for final deck surface elevation tables are defined in BDM Section 302.2.3.

In addition to the final deck surface elevation table, provide a final deck surface elevation note similar to the one below to define the elevations that are given.

[F106] **FINAL DECK SURFACE ELEVATIONS** shown represent the deck surface location after all anticipated dead load deflections have occurred.
702.17  STEEL DRIP STRIP

[84]  Note retired - see appendix
[85]  Note retired - see appendix

702.18  REINFORCING STEEL FOR REHABILITATION

[86]  Note retired - see appendix

702.19  ELASTOMERIC BEARING MATERIAL REQUIREMENTS

[87]  ELASTOMERIC BEARINGS: The elastomer shall have a hardness of ___ durometer. The bearings were designed under Division I, Section 14.6.__ (Method ___ ) of the AASHTO Standard Specifications for Highway Bridges.

NOTE TO DESIGNER: Design Method A is Section 14.6.6 and Method B is Section 14.6.5.

702.20  BEARING SEAT ADJUSTMENTS FOR SPECIAL BEARINGS

Provide the following plan note in project plans that specify specialized bearings such as pot, spherical or disc. This note is intended to ensure that the contractor builds the bearing seats to the proper elevation in the event that the bearing manufacturer adjusts the height of the bearing from the height assumed in the design plans.

[88]  The pier and abutment beam seat elevations are based on bearing heights provided in the table below. If the Contractor’s selected bearing manufacturer has a design that does not conform to the heights provided in the table, adjust the bearing seat elevations at no additional cost to the state. Adjust the location of reinforcing steel horizontally as necessary to avoid interference with the bearing anchor bolts. Maintain the minimum concrete cover and minimum spacing required by the project plans. If the reinforcing steel cannot be moved to provide the required position for the anchor bolts, the Contractor’s bearing manufacturer shall re-design the bearings to accommodate an acceptable anchor bolt configuration.

<table>
<thead>
<tr>
<th></th>
<th>Rear Abutment</th>
<th>Pier No #</th>
<th>Forward Abutment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member Line 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Member Line 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Member Line 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Member Line 4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
702.21  HAUNCHED GIRDER FABRICATION NOTE

For steel haunched girders, add the following note on the design plan sheet that shows an elevation view of the typical haunched girder section defining web size, flange size, depth of member, CVN, etc.

[89]  HAUNCHED GIRDERS: Near the bearing, at the intersection of the horizontal bottom flange with the curved (haunched) portion of the bottom flange, the Contractor’s fabricator shall hot bend the flange in accordance with AASHTO Division II, Section 11.4.3.3.3 or provide a full penetration weld, with 100% radiographic inspection.

702.22  FRACTURE CRITICAL FABRICATION NOTE

For structures that contain fracture critical components and members, place the following note in the design plans.

[90]  FCM: All items designated FCM (, including _____ )* are Fracture Critical Members and Components and shall be furnished and fabricated according to the requirements of Section 12 of the AASHTO/AWS Bridge Welding Code D1.5.

* - Include this additional wording if there exists fracture critical components such as welds, attachments, etc. that are not easily or clearly identified in the plan details. Write descriptions of such components as specific as necessary to prevent any possible confusion during fabrication.

702.23  WELDED SHEAR CONNECTORS ON GALVANIZED STRUCTURES

For galvanized structures with welded shear connectors, place the following note on the same plan sheet as the shear connector spacing.

[91]  WELDED SHEAR CONNECTORS: Install shear connectors after the decking or other walking/working surface, has been installed. Remove the galvanic coating by grinding at each connector location prior to welding.

703  SITE PLAN REQUIREMENTS FOR SECTION 401 AND 404 OF THE CLEAN WATER ACT

For waterway crossing projects, include the following information on the Preliminary Structure Site Plan. Refer to Section 201.2.2 for additional information.
For this project, permits for Sections 401 and 404 of the Clean Water Act, are based on the limits of temporary construction fill placed in “Waters of the United States” as shown below. If either of the limits provided are exceeded, then a 404/401 permit modification will be required. If a permit modification is required, refer to Supplemental Specification 832.09 for the application requirements.

Plan Area of Temporary Fill Material = ____ acres [m²]
Total Volume of Temporary Fill Material = ____ yd³ [m³]
<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Bearing Pt</th>
<th>1/4 Pt</th>
<th>Pt (3)</th>
<th>Mid Span</th>
<th>Pt (3)</th>
<th>1/4 Pt</th>
<th>Splice Pt</th>
<th>Bearing Pt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection due to weight of steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection due to remaining deadload (4)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adjustment required for vertical curve</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adjustment required for horizontal curve</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adjustment required for heat curving (5)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Required Shop Camber</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) Table is only an example. For multiple span structures include each span and the bearing points in a single table.

(2) Define distance and position

(3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 30 feet. If the distance does exceed 30 feet locate the additional point midway between standard points.

(4) Do not include a separate wearing surface that is not installed during the project

(5) For horizontally curved girders the designer is responsible for establishing the required additional camber to be included in the girder per AASHTO 10.15.3. Include these values.
ASPHALT THICKNESS DIAGRAM
(Crest Vertical Curve)

$\frac{3}{8}$" Uniform Thickness 448 Surface

$\frac{3}{8}$" Minimum Uniform Thickness 448 Intermediate

Proposed Roadway Surface

Excessive Curve

3"

Box Beam Member

Q Span

Variable Thickness 448 Intermediate as required to match final elevation (Dimensions shall be detailed)

$\frac{3}{8}$" Uniform Thickness 448 surface

$\frac{3}{8}$" Minimum Uniform Thickness 448 Intermediate

Proposed Roadway Surface

3"

Box Beam Member

Q Span

ASPHALT THICKNESS DIAGRAM
(Straight Grade or Sag Vertical Curve)

Figure 702
### STRUCTURAL STEEL SCREED TABLE

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Station</th>
<th>Station</th>
<th>Station</th>
<th>Station</th>
<th>Station</th>
<th>Station</th>
<th>Station</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Pt</td>
<td>1/4 Pt</td>
<td>Pt (3)</td>
<td>Mid Span</td>
<td>Pt (3)</td>
<td>Splice Pt</td>
<td>1/4 Pt</td>
</tr>
<tr>
<td>Left Curb Line</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phased Const Line</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Curb Line</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Detail all spans.
2. Station all Points for screeds.
3. Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.
### COMPOSITE BOX BEAM SCREED TABLE

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>SPAN NUMBER (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Station</td>
</tr>
<tr>
<td></td>
<td>Bearing Pt</td>
</tr>
<tr>
<td>Left Curb line</td>
<td></td>
</tr>
<tr>
<td>Phased Const Line</td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td></td>
</tr>
<tr>
<td>Right Curb Line</td>
<td></td>
</tr>
</tbody>
</table>

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.
### STRUCTURAL STEEL DEFLECTION & CAMBER TABLE

#### DEFLECTION AND CAMBER

<table>
<thead>
<tr>
<th>SPAN NUMBER (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Pt</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Deflection due to weight of steel</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Deflection due to remaining deadload (4)</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Adjustment required for vertical curve</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Adjustment required for horizontal curve</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Adjustment required for heat curving (5)</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Required Shop Camber</th>
</tr>
</thead>
</table>

(1) Table is only an example. For multiple span structures include each span and the bearing points in a single table.

(2) Define distance and position

(3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 10 meters. If the distance does exceed 10 meters locate the additional point midway between standard points.

(4) Do not include a separate wearing surface that is not installed during the project.

(5) For horizontally curved girders the designer is responsible for establishing the required additional camber to be included in the girder per AASHTO 10.15.3. Include these values.
ASPHALT THICKNESS DIAGRAM
(Crest Vertical Curve)

38 mm Uniform Thickness 448 Surface

37 mm Minimum Uniform Thickness 448 Intermediate

Variable Thickness 448 Intermediate course as required to match final elevation (Dimensions shall be detailed)

Proposed Roadway Surface

Excessive Curve

75 mm

Box Beam Member

Q Span

ASPHALT THICKNESS DIAGRAM
(Straight Grade or Sag Vertical Curve)

Figure 702M
### STRUCTURAL STEEL SCREED TABLE

<table>
<thead>
<tr>
<th>Point (2)</th>
<th>SPAN NUMBER (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Station</td>
</tr>
<tr>
<td></td>
<td>Bearing Pt</td>
</tr>
<tr>
<td>Left Curb Line</td>
<td></td>
</tr>
<tr>
<td>Phased Const Line</td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td></td>
</tr>
<tr>
<td>Right Curb Line</td>
<td></td>
</tr>
</tbody>
</table>

1. Detail all spans.
2. Station all Points for screeds.
3. Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 10 meters. If the distance does exceed 10 meters, locate the additional point midway between standard points.
<table>
<thead>
<tr>
<th>Point (2)</th>
<th>Span Number (1)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Station</td>
</tr>
<tr>
<td>Bearing Pt</td>
<td>1/4 Pt</td>
</tr>
<tr>
<td>Left Curb line</td>
<td></td>
</tr>
<tr>
<td>Phased Const Line</td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td></td>
</tr>
<tr>
<td>Right Curb Line</td>
<td></td>
</tr>
</tbody>
</table>

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 10 meters. If the distance does exceed 10 meters, locate the additional point midway between standard points.
SECTION 800 – NOISE BARRIERS

Refer to the July 2007 ODOT Bridge Design Manual for all Noise Barrier requirements.
SECTION 900 – BRIDGE LOAD RATING

901 PURPOSE

The purpose of this Section is to provide consistency and uniformity in procedures, guidelines and policies for determining safe live load carrying capacity or load rating of the highway bridges in the State of Ohio.

902 SCOPE

The guidelines, policies and recommendations provided in this Section are meant to assist Bridge Owners and bridge raters by establishing evaluation practices that meet the Ohio Revised Code (ORC), the National Bridge Inspection Standards (NBIS), ODOT Bridge Design Manual (BDM) and American Association of State Highway Transportation Officials (AASHTO). The intent is to establish standardized load rating procedures to conform FHWA reporting requirements and posting of bridges throughout the State of Ohio.

903 APPLICABILITY

The provisions of this Section apply to all highway structures in Ohio, which qualify as bridges in accordance with the definition for a bridge set in this Section. These provisions may be applied to smaller structures which do not qualify bridges, as such.

904 QUALITY MEASURES

To maintain the accuracy and consistency of load rating, the bridge owners should implement appropriate quality assurance and quality control (QA/QC) measures. Typical quality control procedures include the use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than the originating individual and periodic field review of the inspection teams and their work.

Each load rating analysis shall be performed and reviewed by two different individuals. One of them shall be a Professional Engineer (PE) registered in the state of Ohio. The same PE shall then sign and stamp (seal) the final load rating report before submission to the bridge owner.

905 DEFINITIONS AND TERMINOLOGY

**ASR:** Allowable Stress Rating (also known as Working Stress Rating)

**ADTT:** Average Daily Truck Traffic volume in one direction

**Bridge:** A structure including supports over a depression or an obstruction such as water,
highway, or railway; having a roadway to carry vehicular traffic and having an opening measured along the centerline of the roadway of 10 ft. [3.048 m] or more between under-copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. It may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

**Bridge Management System (BMS):** A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges

**Bridge Owner:** A public or private entity that has jurisdiction over the bridge

**Buried structure:** A structure, including a flat slab, an arch, a frame, a box section, etc., that has a fill or pavement material of 2 ft. [600 mm] or more on top of it

**Collapse:** A major change in the geometry of the bridge rendering it unfit for its intentional use

**Condition Rating:** The result of the assessment of the functional capability and the physical condition of bridge components by considering the extent of deterioration and other defects. Generally, Condition Rating is evaluated on a scale “0” through “9” (where “9” is the best) and also referred to as General Appraisal

**Exemption List:** A list of structures exempt from the requirements of load rating given in this section

**Failure:** A condition where a limit state is reached or exceeded. This may or may not involve collapse or other catastrophic occurrences

**FHWA:** Federal Highway Administration – U.S. Department of Transportation

**Inventory Rating:** Load ratings based on the inventory level allow comparisons with the capacity for new structures and, therefore result in a live load, which can safely utilize a structure for an indefinite period of time

**Health Index:** An indicator of the structural health of an element, a bridge or a group of bridges expressed as a value (0 to 100), where 0 corresponds to the worst possible condition and 100 corresponds to best possible condition

**LFR:** Load Factor Rating

**Limit State:** A condition beyond which a bridge or a component ceases to satisfy the criteria for which it was designed.

**Load Effect:** The response (axial force, shear force, bending moment, torque, etc.) in a member or an element due to the loading

**Load Factor:** A load multiplier accounting for the variability of the loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads
**Load Rating**: The determination of the live-load carrying capacity of a bridge

**Long span bridge**: Any single or multi span bridge that has at least one span greater than 200 ft. [61 m]

**LRFD**: Load and Resistance Factor Design

**LRFR**: Load and Resistance Factor Rating

**NBI**: National Bridge Inventory, the aggregation of structure inventory and appraisal data collection to fulfill the requirements of National Bridge Inspection Standards (NBIS)

**NBIS**: National Bridge Inspection Standards, Federal regulations establishing requirements for inspection procedures, frequency of inspection, a bridge inspection organization, qualification of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS applies to all structures defined as NBIS bridges located on or over all public roads.

**NBIS Bridge**: A structure including supports over a depression or an obstruction such as water, highway, or railway; having a roadway to carry vehicular traffic and having an opening measured along the centerline of the roadway of more than 20 ft. [6.01 m] between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. It may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

**Nominal Resistance**: Resistance of a component or connection to load effect, based on its geometry, permissible stresses, or specific strength of materials

**Non-buried Structure**: A structure, including a flat slab, an arch, a frame, a box section, etc., that have a fill or pavement material of less than 2'-0" [600 mm] on top of it.

**ODOT**: Ohio Department of Transportation

**ODOT Bridge**: A bridge in which ODOT has jurisdiction

**Operating Rating**: Load ratings based on the operating rating level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge.

**OPI**: Organizational Performance Indices, A set of Indicators to measure the overall condition of bridges at the District or network level based on the several appraisal ratings

**ORC**: Ohio Revised Code (as amended and adopted)

**OSE**: ODOT Office of Structural Engineering

**Owner**: Agency having jurisdiction over the bridge
**Pavement of a roadway:** The pavement of a roadway includes all the paved or unpaved portions of a roadway including graded shoulders that may support vehicular traffic

**PDF:** Portable Document Format, a type of industry standard, electronic file format developed by the Adobe Corporation

**Posting:** Signing a bridge for load restriction

**Preliminary Design Date:** The date when Federal-aid funds are obligated for the studies or design activities related to identification of the type, size, and/or location of bridges. For ODOT projects following the Project Development Process (PDP), this date corresponds to the initiation of Step 1 for a Minimal Project, Step 3 for a Minor Project or Step 6 for a Major Project.

**Quality Assurance:** The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify and measure the quality level of the entire bridge inspection and load rating program

**Reliability Index:** A computed quantity defining the relative safety of a structural element or structure expressed as the number of standard deviations that the mean of the margin of safety falls on the safe side.

**Resistance Factor:** A resistance multiplier accounting for the variability of material properties, structural dimensions, workmanship and the uncertainty in the prediction of resistance

**RF:** Rating Factor, an indicator of live load carrying capacity of a member or a bridge

**Safe Load Capacity:** A live load that can safely utilize a bridge repeatedly over the duration of a specified inspection cycle

**Service Limit State:** Limit state related to stress, deformation and cracking

**Serviceability:** A term that denotes restrictions on stress, deformation, and crack opening under regular service conditions

**Serviceability Limit State:** Collective term for service and fatigue limit states

**Strength Limit State:** Safety limit state relating to strength and stability

**Superload:** In Ohio, a Superload is any highway vehicular load with the total gross load equal to or more than 120,000 pounds (60 tons) or [54,431 kg].

**Target Reliability:** A desired level of reliability in a proposed evaluation

906 **REFERENCES FROM OHIO REVISED CODE**

References from the ORC related to bridge load rating, posting are given below:
5577.071 Reduction of weight of vehicle or load or speed on deteriorated or vulnerable bridge.

(A) When deterioration renders any bridge or section of a bridge in a county insufficient to bear the traffic thereon, or when the bridge or section of a bridge would be damaged or destroyed by heavy traffic, the board of county commissioners may reduce the maximum weight of vehicle and load, or the maximum speed, or both, for motor vehicles, as prescribed by law, and prescribe whatever reduction the condition of the bridge or section of the bridge justifies. This section does not apply to bridges on state highways.

(B) A schedule of any reductions made pursuant to division (A) of this section shall be filed, for the information of the public, in the office of the board of county commissioners in each county in which the schedule is operative. A board of county commissioners that makes a reduction pursuant to division (A) of this section shall, at least one day before a reduction becomes effective, cause to be placed and retained on any bridge on which a reduction is made, at both ends of the bridge, during the period of a reduced limitation of weight, speed, or both, signs of substantial construction conspicuously indicating the limitations of weight or speed or both which are permitted on the bridge and the date on which these limitations go into effect. No person shall operate upon any such bridge a motor vehicle whose maximum weight or speed is in excess of the limitations prescribed. The cost of purchasing and erecting the signs provided for in this division shall be paid from any fund for the maintenance and repair of bridges and culverts.

(C) Except as otherwise provided in this division, no reduction shall be made pursuant to division (A) of this section on a joint bridge as provided in section 5591.25 of the Revised Code unless the board of county commissioners of every county sharing the joint bridge agrees to the reduction, the amount of the reduction, and how the cost of purchasing and erecting signs indicating the limitations of weight and speed is to be borne. A board of county commissioners may make a reduction pursuant to division (A) of this section on a section of a joint bridge, without the agreement [of] any other county sharing the bridge, if the section of the bridge on which the reduction is to be made is located solely in that county.

5591.42 Carrying capacity of bridges - warning notice.

The board of county commissioners together with the county engineer or an engineer to be selected by the board, or the director of transportation, may ascertain the safe carrying capacity of the bridges on roads or highways under their jurisdiction. Where the safe carrying capacity of any such bridge is ascertained and found to be less than the load limit prescribed by sections 5577.01 to 5577.12 of the Revised Code, warning notice shall be conspicuously posted near each end of the bridge. The notice shall caution all persons against driving on the bridge a loaded conveyance of greater weight than the bridge’s carrying capacity.

Effective Date: 11-02-1989
BRIDGE FILES (RECORDS)

Bridge owners should maintain a complete, accurate and current record of each bridge under their jurisdiction. Complete information, in good usable form, is vital to the effective management of bridges. Such information provides a record that may be important for repair, rehabilitation, replacement and future planning of the bridges.

Items that should be assembled as part of the bridge record are discussed below. Some or all of the information pertaining to a bridge may be stored in electronic format as part of a bridge management system.

CONSTRUCTION PLANS

Each bridge record should include one clear and readable set of all drawings used to construct, repair and/or rehabilitate the bridge. In lieu of hard copies, the construction plans may be stored in an electronic format in such a way that clear and readable paper copies can easily be reproduced from the electronic records.

CONSTRUCTION & MATERIAL SPECIFICATIONS

Each bridge record should include the reference to the construction and material specification used during the construction of the bridge. Where general technical specifications were used, only the special technical provisions need to be incorporated in the bridge record.

SHOP AND WORKING DRAWINGS

One set of all shop and working drawings approved for the construction or repair of a bridge should be saved or preserved as a part of the bridge record.

AS-BUILT DRAWINGS

Each bridge record should include one set of final drawings showing the “as-built” condition of the bridge, complete with signature of the individual responsible for recording the as-built conditions.

CORRESPONDENCE

Include all pertinent letters, memoranda, notice of project completion, telephone memo and other related information directly concerning the bridge in chronological order in the bridge record.
907.6  INVENTORY DATA

A complete inventory of a bridge in the ODOT BMS shall be done as soon as a bridge is open to traffic. FHWA mandates an ODOT bridge shall be inventoried within 90 days and a Non-ODOT bridge shall be inventoried within 180 days from the day the bridge was open to traffic. The same rule applies to modifications in the inventory record of replaced bridges or the bridges that have been reopened after the repairs are done. Initial inventory can be completed using the bridge plans. However, a history of dates of physical closing or opening of the traffic on the bridge should be maintained in the bridge record.

907.7  INSPECTION HISTORY

Each bridge record should include a chronological record of the date and the type of all inspections performed on the bridge. When available, scour, seismic, wind and fatigue evaluation studies; fracture critical information; deck evaluations; field load testing; and corrosion studies should be part of the bridge record.

907.8  PHOTOGRAPHS

Each bridge record should at least contain photographs of the bridges showing, top view, approach views and the elevation. Other photos necessary to show major defects, damages, or other important features, such as utilities on or under the bridge, should also be included.

907.9  RATING RECORDS

The bridge record should include a complete record of the determination of the bridge’s load-carrying capacity.

907.10  ACCIDENT DATA

Details of accidents or damage occurrences, including date, description of accident, member damage and repairs, supported by photographs and investigation reports should be included in the bridge record.

907.11  MAINTENANCE AND REPAIR HISTORY

Each bridge record should include a chronological record documenting the maintenance and repairs that have occurred since the initial construction of the bridge. Include details such as date, description of project, contractor cost and related data for in-house projects.
907.12  POSTING HISTORY

Each bridge record should include a summary of all posting actions taken for the bridge, including load capacity calculations, date of posting and description of signing used.

908  GENERAL

The provisions of BDM Section 900 apply to ODOT bridges. All provisions of BDM Section 900 may also be applied to Non-ODOT bridges at the discretion of the bridge owner.

For load rating of new bridges, BDM Sections 911 through 926 shall apply.

For load rating of existing bridges, BDM Sections 911 through 925 & 927 shall apply.

909  UNIT WEIGHTS & DENSITIES

The following assumptions should be made while performing the load rating analysis, unless more accurate site information is available:

A. Unit weight of asphalt .............................................................. 145 lb/ft³ [22.8 kN/m³]
B. Unit weight of concrete ............................................................ 150 lb/ft³ [23.6 kN/m³]
C. Unit weight of latex modified concrete ................................... 150 lb/ft³ [23.6 kN/m³]
D. Unit weight of soil ................................................................. 120 lb/ft³ [18.9 kN/m³]
E. Unit weight of steel ................................................................. 490 lb/ft³ [77.0 kN/m³]
F. Water density ................................................................. 62.4 lb/ft³ [9.8 kN/m³]

910  STRUCTURES EXEMPT FROM LOAD RATING

Following types of buried structures are exempt from load rating under the provisions of this Section.

A. Circular plastic pipes
B. Concrete pipes (circular, or elliptical)
C. Buried metal frames
D. Junction chambers
E. Manholes
F. Inlets and outlets
WHICH PORTION OF BRIDGES SHALL BE LOAD RATED

Any bridge or structural member of a bridge that would carry highway traffic shall be load rated. Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the owner or the rating engineer has reason to believe that their capacity may govern the capacity of the bridge.

PROCEDURE FOR RATING

A. The relative strength ratings for each bridge shall be determined in the following manner:
   1. A careful field inspection of the existing bridge shall be made by the District Bridge Engineer and/or other qualified structural engineer to determine its condition, and the percent of effectiveness of the various members for carrying load. All information shown in the Bridge Inventory and Inspection Records shall also be carefully checked and revised as necessary to show the current condition of the bridge.
   2. New (proposed) bridges shall be load rated at the design stage per BDM Section 926
   3. Using pertinent current information, the District Bridge Engineer/Bridge Owner shall determine the Inventory, Operating, and Ohio Legal Load Ratings.
   4. The yield stresses for the construction materials in older bridges, for which plan information is not available, can generally be determined using the date of construction.
   5. For a load rating analysis request on an ODOT bridge, the District Bridge Engineer shall submit to the OSE, Bridge Rating Engineer a complete inspection report (including comments), bridge photographs, field measurements and a copy of the previous rating calculation sheets or computer input data sheets.
   6. The Bridge Rating Engineer shall review the submitted material, analyze bridge and return a copy of the final calculations or computer output to the District Bridge Engineer, along with any recommendations concerning the proposed ratings.
   7. The District Bridge Engineer shall keep the final calculations or computer output along with any recommendations concerning the proposed ratings on file.

WHEN LOAD RATING SHALL BE REVISED

The load rating of a bridge should be revised when:

A. There is a physical change in the condition of a structural member of the bridge
B. Rusting or damage to a slab, beam, girder or other structural element that has resulted in section loss
C. There is structural damage to steel, like a hit by a vehicle, excessive deflection or elongation under temperature or highway loads
D. When the inspection general appraisal rating of the superstructure of a bridge drops below 5
E. There is an addition of a new beam or girder
F. A new deck is added or the existing deck width is changed
G. There is a change in the dead load on the superstructure, like addition or removal of wearing surfaces, addition or removal of sidewalks, parapets, railings, etc.

The load rating of a bridge does not need to be revised when:

A. The change in the thickness of external wearing surface is less than 1 inch [2.54 cm]
B. The change in the dead load on a beam member is not more than 10 pounds per ft.

914 ANALYSIS OF BRIDGES WITH SIDEWALKS

A pedestrian load of 75 pounds per square feet shall be applied to all sidewalks wider than 2.0 ft. and considered simultaneously with the live load in the vehicle lane.

When pedestrian load is present, the pedestrian load effect should be subtracted from the capacity of the member at the location being investigated while calculating the RF.

For bridges load rated according to the AASHTO Standard Specifications for Highway Bridges, AASHTO Table 3.22.1A applies. For bridges load rated according to the AASHTO LRFD Bridge Design Specifications, refer to BDM Section 925.2.

Pedestrian load shall not be considered in Special or Permit Load Analysis per BDM Section 916.

915 ANALYSIS OF MULTILANE LOADING

Traffic lanes to be used for rating purposes shall be in accordance with AASHTO.

AASHTO reduction factors for multiple lane loadings shall be applied where appropriate.

For rating analysis of floor beams, trusses, non-redundant girders or other non-redundant main structural members, position identical rating vehicles in one or more of the through traffic lanes on the bridge, spaced and shifted laterally on the deck, within the traffic lanes, so as to produce the maximum stress in the member under consideration.

916 ANALYSIS FOR SPECIAL LOAD OR SUPERLOAD

When a structure is required, in the Scope of Services, to be analyzed for special or Superload vehicle, a second analysis shall be performed for a single lane loading of the special or Superload vehicle condition. The special or Superload vehicle shall be placed laterally on the structure to produce maximum stresses in the critical member under consideration.

The analysis for special or Superload vehicle shall be performed at the operating level only.
917 LOAD RATING OF LONG SPAN BRIDGES

917.1 WHEN THE LOAD RATING SHALL BE DONE

Perform the load rating of long span bridges according to BDM Sections 926.3.3, 927.3.2, or 927.4.2.

917.2 HOW THE LOAD RATING SHALL BE DONE

917.2.1 INVENTORY & OPERATING LEVEL RATING USING HL-93 LOADING

The live load shall be applied as per AASHTO LRFD Design Specification.

917.2.2 INVENTORY & OPERATING LEVEL RATING USING HS20 TRUCK

The live load shall be applied as follows:

A. In the right-most lane, place a series of HS20 trucks to simulate a train of vehicles. The vehicle train shall consist of the HS20 trucks spaced with 30 ft. clear distance between the rear axle of the leading vehicle and the front axle of the trailing vehicle. The distance between the second axle and the rear axle shall be fixed at 14.0 ft. Place as many fixed-axle spacing HS20 trucks as necessary to produce the maximum load effect on the component to be rated. No partial HS20 trucks shall be used.

B. In all other lanes in the same direction, simultaneously place single, variable-axle spacing HS20 trucks positioned to produce the maximum load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.

C. For bridges with two-way traffic, apply the live load as described in A. and B. above to the lanes in the opposite direction.

917.2.3 OPERATING LEVEL RATING USING OHIO LEGAL LOADS

The provisions of BDM Sections 926 or 927 shall apply except the live load application shall be in accordance with BDM Section 917.2.3.1, 917.2.3.2 or 917.2.3.3.

917.2.3.1 BRIDGES WITH THREE OR MORE LANES

If no permit vehicle is present, apply the following live load:

A. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle
of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

B. In all other lanes in the same direction, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.

C. For bridges with two-way traffic, apply the live load for the opposing traffic in the same manner as the one-way traffic.

If a permit load vehicle is present, apply the following live load:

A. In the right-most lane, place one permit load vehicle positioned to produce the maximum live load effect on the component to be rated. In the adjacent lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

B. In all other lanes in the same direction, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.

C. For bridges with two-way traffic, place a series of Ohio 5C1 vehicles in the right-most lane. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used. In all remaining lanes, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.

917.2.3.2 BRIDGES WITH TWO LANES

If no permit vehicle is present, apply the following live load:

A. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

B. For bridges with one-way traffic, in the other lane simultaneously place a single 5C1 vehicle positioned to produce the maximum live load effect on the component to be rated.

C. For bridges with two-way traffic, in the other lane place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading
vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

If a permit load vehicle is present, apply the following live load:

A. In the right-most lane, place one permit load vehicle positioned to produce the maximum live load effect on the component to be rated.

B. In the other lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

917.2.3.3 BRIDGES WITH A SINGLE LANE

If no permit vehicle is present:

The live load shall be a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

If a permit vehicle is present:

The live load shall be the one permit vehicle positioned to produce the maximum live load effect on the component to be rated.

918 BRIDGE POSTING FOR REDUCED LOAD LIMITS

918.1 PURPOSE

The Procedure outlined in this section is to be followed for posting warnings of bridge strength deficiencies on ODOT bridges.

918.2 REFERENCE

Ohio Revised Code, Section 5591.42:

918.3 PROCEDURE FOR BRIDGE POSTING

A. When the Operating Rating of the bridge is determined to be less than 100% of the Ohio legal load and the bridge cannot be strengthened immediately to a rating of 100% or above, the following procedures shall be used:
1. The District Bridge Engineer shall:
   a. Establish a rating and submit to the OSE Bridge Rating Engineer, a written request for the bridge posting. (See BDM Section 918.6 for required content of request.)
   b. After the Director signs the posting request, the District Roadway Services Manager shall prepare, erect and maintain all necessary signs until the bridge is either strengthened or replaced.

2. The District Bridge Engineer shall update all Bridge Inventory and Inspection records to show the latest official posted capacity.

3. After the posting request is signed, the Bridge Rating Engineer shall send a copy to the:
   District Bridge Engineer; Manager, Hauling Permits Section of the Office of Highway Management; Superintendent of State Highway Patrol; Executive Director Ohio Trucking Association; the Board of County Commissioners; and the County Engineer where the bridge is located.

B. Special treatment shall be applied to legal load ratings of 95% or higher and also to legal load ratings of 15% or less as follows:
   1. Because of the use of some judgmental data in the rating computations, bridges with a calculated load reduction of 5% or less, after rounding, shall not be posted. These bridges shall be rated at 100% of legal load.
   2. For calculated load reductions of 85% or more, after rounding, the bridge must be considered for "closing" to all traffic until it can be rehabilitated or replaced.

C. Where posting of a bridge is determined necessary and no unusual or special circumstance at the bridge dictates otherwise, Ohio standard regulatory signs shall be placed in sufficient numbers and at the specific locations required below.
   1. Example of standard wording to be used on signs is given in Figure 905.
   2. Bridge Ahead signs shall be erected at intersecting state roads located just prior to the bridge to allow approaching vehicles to by-pass the bridge or turn around safely with a minimum of interference to other traffic.
   3. Bridge Weight Limit signs shall be erected at each end of the bridge.
Table 918.3-1: ODOT Bridge Posting Policy

<table>
<thead>
<tr>
<th>% Ohio Legal Value</th>
<th>Reported % Ohio Legal in BMS</th>
<th>Posting for Reduced Loads Needed</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥150%</td>
<td>150%</td>
<td>NO</td>
</tr>
<tr>
<td>≥100% and &lt;150%</td>
<td>Actual percentage rounded to the nearest 5 (i.e. 100, 105, 110, 115, etc.)</td>
<td>NO</td>
</tr>
<tr>
<td>≥92.5% and &lt;100%</td>
<td>100%</td>
<td>NO</td>
</tr>
<tr>
<td>&lt;92.5%</td>
<td>Actual percentage rounded to the nearest 5 (starting with 90%, 85%, 80%, etc.)</td>
<td>YES</td>
</tr>
</tbody>
</table>

918.4 PROCEDURE FOR RESCINDING POSTING

A. When a posted bridge has been strengthened or replaced and no longer needs posting, the District Bridge Engineer shall forward to the Bridge Rating Engineer a written request to rescind the existing signed posting. The request shall include a complete statement of the reason for the action as specified in BDM Section 918.6.

B. The Bridge Rating Engineer shall review the data submitted by the District Bridge Engineer and upon concurrence shall forward to the Director a request to rescind the posting.

C. The Bridge Rating Engineer shall distribute copies of the rescind notice as described in Section 918.3.A.3.

918.5 PROCEDURE FOR CHANGING POSTING

When the rated capacity of bridge changes, so as to require a revised posting level, the procedures in BDM Section 918.3 apply and in addition, the existing posting must be rescinded as set forth in BDM Section 918.4.
918.6 REQUIRED INFORMATION FOR POST, RESCIND AND CHANGE REQUESTS

The following minimum information is required on all post, rescind and change requests:

A. Posting Request (Reduction in Load Limits)
   1. County in which bridge is located
   2. Current Bridge Number
   3. Structure File Number
   4. Feature intersected (over or under bridge)
   5. Tonnage unit requested for the four typical legal vehicles.
   6. Existing rating of bridge expressed as a percent of legal load or tons.
   7. Explanation as to why posting is required
   8. Attach copies of all official documentation for any associated actions by involved agencies other than the state.

B. Rescinding Request (Removal of Existing Load Limits)
   1. County in which bridge is located
   2. Current Bridge Number
   3. Structure File Number
   4. Feature intersected (over or under bridge)
   5. Existing posting (% reduction or weight limit currently in effect)
   6. Date existing posting was effective
   7. Explanation as to why posting restrictions can now be removed (show contract project numbers or indicate force account or other work method used to correct problem)
   8. New load rating for the rehabilitated or new structure

C. Change Request (Revision of Existing Posted Limits)
   1. County in which bridge is located
   2. Current Bridge Number
   3. Structure File Number
   4. Feature intersected (over or under bridge)
   5. Existing posting (weight limit currently in effect)
   6. Revised posting request
   7. Date of existing posting
   8. Explanation as to why posting change is necessary (include project numbers etc.)
SOFTWARE TO BE USED FOR LOAD RATING

One of the following computer programs to be used for the load rating of bridges, as applicable.

A. **AASHTO Virtis**: Virtis is a load rating and analysis product developed and licensed by AASHTO. Virtis can rate the bridges by LRFR and LFR methods. It is one of ODOT’s preferred programs to do load rating. ([http://aashto.bakerprojects.com/virtis/](http://aashto.bakerprojects.com/virtis/)).

B. **Bentley LARS**: LARS is bridge analysis software maintained and licensed by the Bentley Systems. It can load rate bridges by LRFR and LFR methods. It is one of ODOT’s preferred programs to do load rating. ([http://www.bentley.com](http://www.bentley.com))

C. **AASHTO BARS-PC**: BARS-PC is the default bridge analysis and load rating program by LFR method for all bridges designed prior to October 1, 2010. BARS-PC program is available from ODOT for a nominal charge of material and shipping. It is one of ODOT’s preferred programs to do load rating.

D. **BRASS-Culvert**: BRASS-Culvert can load rate reinforced concrete flat-topped 3-sided frames and 4-sided boxes buried under the fill by LRFR and LFR methods. BRASS-Culvert software shall be used for the analysis of concrete box sections and three sided concrete frames. BRASS family of programs is developed, maintained and licensed by the Wyoming Department of Transportation. It is one of ODOT’s preferred programs to do load rating.

([http://www.dot.state.wy.us/wydot/engineering_technical_programs/bridge/brass](http://www.dot.state.wy.us/wydot/engineering_technical_programs/bridge/brass))

E. **LARSA 4D**: Finite element analysis programs by LARSA, Inc., 105 Maxess Road, Melville Corporate Center, Suite 115N, Melville, NY 11474 ([http://www.larsausa.com](http://www.larsausa.com)).

F. **DESCUS I**: DESCUS I can perform analysis of horizontally curved flanged steel sections which act compositely or non-compositely with a concrete deck. The program can be run using Load Factor or Load and Resistance Factor method.


G. **MDX Software**: MDX software can be used to design and load rate straight and curved steel bridges. The program can be run using Load Factor or Load and Resistance Factor method.


For the analysis of arches and other special structures that cannot be modeled using any of the programs A through D above, contact the OSE for pre-approval of the software before use.

Also, contact the OSE prior to using any computer program other than A through E above. The Department will not accept load rating performed using any software not pre-approved for that bridge.
LOAD RATING REPORT SUBMISSION

The load rating report shall be submitted to the project manager, District Bridge Engineer or the respective owner (in case of a non-ODOT bridge). The submission shall include two (2) printed copies and one electronic copy of the Load Rating Report and one copy of the electronic input data files. The Load Rating Reports shall be signed, sealed and dated by an Ohio Registered Engineer.

For an ODOT-bridge the District Bridge Engineer will send one printed copy, an electronic copy of the report, the electronic data files and a copy of the final bridge plans to the OSE for review.

The report must list final inventory and operating ratings of each main bridge member, overall ratings of each structure unit (mainline, ramps, etc.), and the final ratings of the entire bridge summarized in a tabular form. The ratings of each member and the overall ratings of the structure shall be presented for each Ohio Legal Load and either AASHTO HS20-44 or HL-93 live load.

An example of a Load Rating Report Summary is given as Figure 908.

For existing bridges, the report shall state how the material properties were determined. Any specific details about the current conditions and bridge geometry shall be listed.

All calculations related to the load rating should be a part of the load rating report.

Submit copies of the input & output computer files in electronic format. Input files must be error free and ready to be run. The rating engineer shall incorporate any changes in the input files as a result of ODOT review.

LOAD RATING USING AASHTO VIRTIS PROGRAM

GENERAL

Virtis is a load rating program licensed from AASHTO. Virtis runs on Microsoft Windows and can load rate a variety of bridges by LFR as well as LRFR methods.

Virtis Vehicle library can be customized to include ODOT Legal Loads. Alternatively Virtis library can be requested from OSE.

VIRTIS LOAD RATING REPORT SUBMISSION

The load rating report shall be submitted to the project manager, District Bridge Engineer or the respective owner (in case of a non-ODOT bridge). The submission shall include two (2) printed copies and one electronic copy of the Load Rating Report and one copy of the electronic input data files. The Load Rating Reports shall be signed, sealed and dated by an Ohio Registered Engineer.
For an ODOT-bridge the District Bridge Engineer or Project Manager shall send a printed copy & an electronic copy of the report, the electronic data files and a copy of the final bridge plans to the OSE for review.

The report must list final inventory and operating ratings of each main bridge member, overall ratings of each structure unit (mainline, ramps, etc.), and the final ratings of the entire bridge summarized in a tabular form.

An example of a Load Rating Summary Report is given as Figure 908.

For existing bridges, the report shall state how the material properties were determined. Any specific details about the current conditions and bridge geometry shall be listed.

All calculations related to the load rating should be included as a part of the load rating report.

921.3 VIRTIS COMPUTER INPUT AND OUTPUT FILES

Submit the error-free and working electronic copies of the input file exported as an “XML” file. To get the electronic copy of a bridge data file in Virtis, open the “Bridge Workspace,” Export the bridge input file into an XML file and submit it electronically for review.

In addition to the electronic input data file, the rating report shall also include copies of the computer rating summary and the rating summary report. The rating report can be submitted as a “PDF” file or a printed hard copy.

922 LOAD ANALYSIS USING LARS PROGRAM

922.1 GENERAL

LARS (Load Analysis and Rating System) is a family of bridge load analysis and rating programs maintained and licensed by Bentley, Systems.

LARS can run on any Microsoft Windows compatible machine.

LARS can import BARS-PC files.

LARS Vehicle library can be customized to include ODOT Legal Loads.

922.2 LARS CAPABILITIES

LARS program can analyze and rate single and multiple span bridges by Allowable Stress; Load Factor; and Load & Resistance Factor methods.
922.3 LARS COMPUTER INPUT AND OUTPUT FILES

Follow the Report submission guidelines given in BDM Section 920.

Also submit the error-free and working electronic copies of the input & output files.

In addition to the electronic input data file, the rater may submit hard (printed) copies of the computer input and output files.

923 LOAD ANALYSIS USING BRASS-CULVERT PROGRAM

923.1 GENERAL

BRASS (Bridge Rating and Analysis System) is a family of several structural analysis modules, such as BRASS-Culvert, BRASS-Girder, BRASS-Pole, etc. BRASS-Culvert program can be used to analyze reinforced concrete three-sided flat-topped frames and four-sided box sections.

If haunch dimensions are different, use the smallest dimension in the analysis.

BRASS can run on any Microsoft Windows compatible machine.

BRASS data files should use the same naming convention as the BARS-PC data files.

BRASS Vehicle library can be customized to include ODOT Legal Loads.

923.2 BRASS CAPABILITIES

BRASS program can analyze single-cell and multi-cell reinforced concrete box structures and frames.

Technical support on BRASS program is available to the BRASS licensed users from the Wyoming Department of Transportation.

923.3 BRASS COMPUTER INPUT AND OUTPUT FILES

Follow the Report submission guidelines given in BDM Section 920.

Also submit the error-free and working electronic copies of the input & output files with extensions “dat,” “cus” and “xml.”

In addition to the electronic input data file, the rater may submit hard (printed) copies of the computer input and output files.
924 LOAD RATING ANALYSIS USING BARS-PC

924.1 GENERAL

The BARS-PC is the PC version of the AASHTO BARS (Bridge Analysis and Rating System) program that can analyze and load rate structures based on the AASHTO Standard Specifications for Highway Bridges.

BARS-PC program installation CD is available for use on ODOT Projects, from the OSE at a nominal cost.

The OSE will provide limited technical support to install and execute the program.

BARS-PC program is not compatible Windows 7 or later operating systems. BARS-PC cannot perform rating based on the LRFR method.

The types of material, methods of construction, bridge member and types of section that can be handled by BARS-PC are provided in the BARS User’s Manual that can be downloaded from the OSE-Bridge Management website.

Figures 906 and 907 provide general information about ODOT Allowable Stresses in bending and shear and material strengths based on the year of construction. These material properties are different from those given in AASHTO BARS Manual 2, Appendix A. However, they are used as default values in the BARS-PC customization file prepared by ODOT, which is available from Structure Rating website. Any material stresses and specifications specified on the design plans shall supersede the values given in Figures 906 and 907.

The rater is cautioned to pay extra attention to the design plans and the year of construction, when determining material strengths for structures built during transition years of Figures 906 and 907 (e.g., for member type SS, years 1964-68, or 1988-93, etc.), as materials may have been substituted.

924.2 BARS-PC ANALYSIS – GENERAL GUIDELINES

All information in a BARS-PC input data file shall be entered in uppercase with “Caps Lock ON.”

The first six digits from left of the Structural File Number (SFN) of the bridge with prefix “R” and extension “dat” shall be used as the input data file name. The same first six digits of the SFN shall be used as Structure Group ID No. on all BARS-PC data input cards. For example, if the SFN of a bridge is 4729854, the input file name should be named as “R472985.DAT” and the Structure Group ID No. will be “472985.”

If a SFN has not been assigned to a new structure, contact Structure Inventory Section in the OSE to get a SFN for the structure.
All BARS-PC input files shall have the word “NEW” in columns 9-11 and the letter “X” in column 17 of card type AA.

All BARS-PC input files shall have the bridge rater’s initials and company/office abbreviations in the columns 15-22 of card type 01 and columns 9-16 of card type 02.

All structures rated by BARS-PC using LF method shall have letter “L” in column 65 of card type 05.

All structures rated by BARS-PC shall have letter “F” in column 66 of card type 05.

All structures rated using BARS program shall have a three-digit Structure Type Code in columns 41-46 of card type 02. The three-digit code shall be selected based on the material, type and the description of the main members according to Structure Type Codes of ODOT Bridge Inventory Coding Guide. For example, Concrete Slab Continuous shall be coded as “112” and Steel Beam Simple Span shall be coded as “321.”

The complete seven digit SFN shall be entered in columns 9-16 of card type 05.

The original method of construction and the loading used for the design of the bridge shall be explicitly stated on card type 06.

The assumptions made to model a structural member or unit for computer analysis shall also be stated on card type 06.

The live load distribution factors for the single lane loading shall be given on the card type 6.

If space on card type 06 (maximum of six cards of type 06) is not sufficient, additional information can be included with the load rating report for ODOT review.

BARS (mainframe) and BARS-PC programs do not recognize standard steel rolled beams, Prestressed I-girder or Prestressed box beam sections. Standard rolled beams shall be coded on card type 12 in terms of flange and web plates. Prestressed I-girders and box beams shall be coded on card type 15 with special attention given to the type, area and strength of the prestressing strands.

When using BARS-PC to load rate multi-span prestressed structures, each member shall be analyzed as a simply supported member.

**924.3 BARS-PC LOAD RATING REPORT SUBMISSION**

Follow the load rating report submission guidelines given in the BDM Section 920.
924.4  BARS-PC COMPUTER INPUT AND OUTPUT FILES

In addition to the electronic input data file, each copy of the rating report shall also include hard (printed) copies of the computer input and output files.

Some computer programs generate several output files during the process of analysis. Include those files that contain information. For example, the load rating analysis report of a steel beam bridge using BARS-PC shall contain printed copies of the following files:

A. lista.lis
B. rate2.lis
C. summary.lis
D. flex.lis

925  LOAD RATING OF BRIDGES USING LRFR SPECIFICATIONS

925.1  APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS

This Section is consistent with the current AASHTO LRFD Bridge Design Specifications. Where this Section is silent, the current AASHTO LRFD Bridge Design Specification shall govern.

925.2  GENERAL LOAD RATING EQUATION

The following general equation shall be used in determining the load rating of each component and connection subject to a single force effect (axial force, flexure or shear) [MBE 6A.4.2]:

\[
RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_{P})(P) - (\gamma_{PL})(PL)}{(\gamma_{LL})(LL)(1 + IM/100)}
\]

For Strength Limit States:
\[
C = \phi_c \cdot \phi_s \cdot \phi \cdot R_n
\]

Where the following lower limit shall apply:
\[
\phi_c \cdot \phi_s \geq 0.85
\]

For Service Limit States:
\[
C = f_R
\]

Where:
\[
C = \text{Capacity}
\]
\[
DC = \text{Dead load effect due to structural components and attachments}
\]
$DW =$ Dead load effect due to wearing surface and utilities
$fR =$ Allowable stress specified in LRFD Code
$IM =$ Dynamic load allowance expressed as percentage ($\%$)
$LL =$ Live Load effect
$P =$ Permanent loads other than dead loads, such earth pressure, shrinkage etc.
$PL =$ Pedestrian Load effect only to be applied when a sidewalk is present
$RF =$ Rating Factor
$R_n =$ Nominal member resistance
$\gamma_{DC} =$ Load factor for DC load
$\gamma_{DW} =$ Load factor for DW load
$\gamma_p =$ Load factor for $P$ load = 1.0
$\gamma_{LL} =$ Evaluation live load factor
$\gamma_{PL} =$ Load factor for Sidewalk load = 1.0
$\phi_c =$ Condition factor
$\phi_s =$ System factor
$\phi =$ LRFD Resistance factor

For Limit States and factors see BDM Section 925.3.

### 925.3 LIMIT STATES AND LOAD FACTORS FOR LOAD RATING

Strength is the primary limit state for load rating; service and fatigue limit states are selectively applied in accordance with the provisions of AASHTO Manual of Bridge Evaluation [MBE 6A.4.2]:

For Inventory and Operating Rating for AASHTO HL-93 Loading, use the following limit states and load factors:
### Table 925.3-1: LRFR Design Load Limit States and Load Factors

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Limit State</th>
<th>Dead Load</th>
<th>HL-93 Loading</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\gamma_{DC}$</td>
<td>$\gamma_{DW}$</td>
<td>Inventory</td>
</tr>
<tr>
<td>Steel</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>Service II</td>
<td>1.00</td>
<td>1.00</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>Fatigue</td>
<td>0.00</td>
<td>0.00</td>
<td>0.75</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>Service III</td>
<td>1.00</td>
<td>1.00</td>
<td>0.80</td>
</tr>
<tr>
<td>Wood</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>1.75</td>
</tr>
</tbody>
</table>

For Rating for Ohio Legal Loads, use the following limit states and load factors:

### Table 925.3-2: Legal Loads Limit States and Load Factors

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Limit State</th>
<th>Dead Load</th>
<th>Ohio Legal Loads</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\gamma_{DC}$</td>
<td>$\gamma_{DW}$</td>
<td>$\gamma_{LL}$</td>
</tr>
<tr>
<td>Steel</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td>Service II</td>
<td>1.00</td>
<td>1.00</td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td>Service III</td>
<td>1.00</td>
<td>1.00</td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td>Wood</td>
<td>Strength I</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
</tbody>
</table>
For Rating for Special and Permit Loads, use the following limit states and load factors:

Table 925.3-3: Permit Load Limit States and Load Factors

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Limit State</th>
<th>Dead Load $\gamma_{DC}$</th>
<th>Dead Load $\gamma_{DW}$</th>
<th>Permit or Special Loads $\gamma_{LL}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Strength II</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td></td>
<td>Service II</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Strength II</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td></td>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Strength II</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
<tr>
<td></td>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Wood</td>
<td>Strength II</td>
<td>1.25</td>
<td>1.50</td>
<td>Unknown ADTT -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\geq$ 5000 -- 1.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT = 1000 -- 1.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ADTT $\leq$ 100 -- 1.40</td>
</tr>
</tbody>
</table>

925.4 DYNAMIC LOAD ALLOWANCE (IM)

A. A dynamic load allowance of 33% shall be used for all non-buried bridges except for fatigue evaluation.

B. For fatigue evaluation a dynamic load allowance of 15% shall be used.

C. Dynamic load allowance shall only be applied to truck or tandem portion of HL-93 loading (dynamic load allowance shall not be provided to lane portion).

D. Dynamic load allowance needs not to be applied to wood components of a bridge.

E. Dynamic allowance may be ignored for slow moving (speed less than 10 mph) special or permit loads under controlled conditions.

F. For buried bridges, dynamic allowance (IM) shall be taken as:

$$IM = 33 \times (1.0 - 0.125 \times DE) \geq 0\% \quad \text{..................................................}$$ [AASHTO 3.6.2.2-1]

Where:

$$DE = \text{the minimum depth of cover above the structure (ft)}$$
925.5 CONDITION FACTOR ($\phi_c$)

A Condition Factor shall be applied to the calculated capacity of the structure, as follows:

Table 925.5-1: Condition Factors [MBE 6A.4.2.3]

<table>
<thead>
<tr>
<th>Structural Condition of a member</th>
<th>NBI General Appraisal</th>
<th>Condition Factor $\phi_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good or Satisfactory</td>
<td>6 or higher</td>
<td>1.00</td>
</tr>
<tr>
<td>Fair</td>
<td>5</td>
<td>0.95</td>
</tr>
<tr>
<td>Poor</td>
<td>4 or lower</td>
<td>0.85</td>
</tr>
</tbody>
</table>

925.6 SYSTEM FACTOR ($\phi_s$)

System factors are multiplied to the nominal resistance to reflect the level of redundancy of the complete superstructure [MBE 6A.4.2.4]. Bridges that are less redundant will have their factored member capacities reduced.

The following system factors may be used for Flexural and Axial Effects:

Table 925.6-1: System Factors [MBE 6A.4.2.4]

<table>
<thead>
<tr>
<th>Superstructure Type</th>
<th>$\phi_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded members in two-girder/truss/arch bridges</td>
<td>0.85</td>
</tr>
<tr>
<td>Riveted members in two-girder/truss/arch bridges</td>
<td>0.90</td>
</tr>
<tr>
<td>Multiple eyebar members in truss bridges</td>
<td>0.90</td>
</tr>
<tr>
<td>Three-girder bridges with girder spacing 6 ft.</td>
<td>0.85</td>
</tr>
<tr>
<td>Four-girder bridges with girder spacing $\leq$ 4 ft.</td>
<td>0.95</td>
</tr>
<tr>
<td>Floor beams with spacing $&gt;12.0$ ft. and non-continuous stringers</td>
<td>0.85</td>
</tr>
<tr>
<td>Redundant stringer subsystems between floor-beams</td>
<td>1.00</td>
</tr>
<tr>
<td>All other girder and slab bridges</td>
<td>1.00</td>
</tr>
</tbody>
</table>

925.7 RESISTANCE FACTOR ($\phi$)

Resistance factor ($\phi$) for the load rating has the same value as for a new design as given in AASHTO LRFD Specification. Also, $\phi = 1.0$ for all non-strength limit states [MBE C6A.4.2.1]. See appropriate section in the LRFD Specification for recommended values for resistance factors [LRFD 5.5.4.2, 6.5.4.2, 8.5.2, 12.5.5]

Some of the commonly used Resistance Factors are given here:
Table 925.7-1: Resistance Factors

<table>
<thead>
<tr>
<th>Type</th>
<th>$\varphi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension controlled reinforced concrete section</td>
<td>0.90</td>
</tr>
<tr>
<td>Tension controlled prestressed concrete section</td>
<td>1.00</td>
</tr>
<tr>
<td>Shear and torsion in normal weight concrete</td>
<td>0.90</td>
</tr>
<tr>
<td>Flexure in steel</td>
<td>1.00</td>
</tr>
<tr>
<td>Shear in steel</td>
<td>1.00</td>
</tr>
<tr>
<td>Axial Compression in steel only</td>
<td>0.90</td>
</tr>
<tr>
<td>Axial Compression in composite</td>
<td>0.90</td>
</tr>
<tr>
<td>Shear connectors, steel</td>
<td>0.85</td>
</tr>
<tr>
<td>Web crippling, steel</td>
<td>0.80</td>
</tr>
<tr>
<td>For block shear</td>
<td>0.80</td>
</tr>
<tr>
<td>For shear rupture in connection element</td>
<td>0.80</td>
</tr>
<tr>
<td>For weld metal in partial penetration and fillet weld</td>
<td>0.80</td>
</tr>
<tr>
<td>Flexure in wood</td>
<td>0.85</td>
</tr>
<tr>
<td>Shear in wood</td>
<td>0.75</td>
</tr>
<tr>
<td>Wood connections</td>
<td>0.65</td>
</tr>
<tr>
<td>RC cast-in-place buried box structures in flexure</td>
<td>0.90</td>
</tr>
<tr>
<td>RC cast-in-place buried box structures in shear</td>
<td>0.85</td>
</tr>
<tr>
<td>RC precast buried box structures in flexure</td>
<td>1.00</td>
</tr>
<tr>
<td>RC precast buried box structures in shear</td>
<td>0.90</td>
</tr>
<tr>
<td>RC precast buried 3-sided structures in flexure</td>
<td>0.95</td>
</tr>
<tr>
<td>RC precast buried 3-sided structures in shear</td>
<td>0.90</td>
</tr>
<tr>
<td>Structural steel pipe, minimum wall area &amp; buckling</td>
<td>1.00</td>
</tr>
<tr>
<td>Structural steel pipe, minimum longitudinal seam strength</td>
<td>0.67</td>
</tr>
</tbody>
</table>

925.8 EFFECT OF SKEW

Effect of skew on the distribution of live loads shall be considered according to AASHTO LRFD Specifications (LRFD 4.6.2.2.2 and 4.6.2.2.3).

926 LOAD RATING OF NEW BRIDGES

926.1 LOADS TO BE USED FOR LOAD RATING

All new and replacement bridges whose preliminary design is started after October 1, 2010, and requiring load rating, shall be load rated at inventory and operating levels by the AASHTO LRFR method to comply with FHWA requirements. The load to be used for inventory and operating rating based on LRFR method shall be AASHTO’s HL-93 loading (truck & lane or tandem & lane), according to Figure 902.
All bridges shall also be load rated for four Ohio Legal Loads (2F1, 3F1, 4F1, and 5C1) at operating level using the same method of analysis as used for inventory and operating ratings above. The four Ohio legal loads (2F1, 3F1, 4F1 and 5C1) are given in Figure 903.

All trucks used for analysis shall have transverse spacing, between centerline of wheels or wheel groups, of 6 ft. [1.830 m].

Long span bridges shall use the special load configurations given in BDM Section 917.

The inventory and operating ratings for the AASHTO HL-93 loading shall be expressed in terms of rating factors, rounded off to the nearest second decimal point. The operating ratings for each of the Ohio Legal Loads shall also be expressed in terms of rating factors of respective legal load rounded off to the nearest second decimal point. The summary rating for all of the Ohio Legal Loads shall be the smallest rating factor of the four vehicles expressed as a percentage rounded off to the nearest 5 (125%, 100%, 65%, etc.).

The owner may also require load rating to be done for special loads in addition to those specified above. The owner shall provide full configurations of the special load, including axle weights and spacing, number of tires on each axle, tire gauges and overall dimensions of the load. All special loads to be analyzed by the LRFR method of analysis at the operating level as per BDM Section 916 unless specified otherwise by the owner.

926.2 LOAD RATING OF NEW BURIED BRIDGES

926.2.1 CAST-IN-PLACE CONCRETE BOX & FRAME STRUCTURES

A. Cast-in-place bridges shall be load rated by the designer of the bridge.

B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, see BDM Section 923.

926.2.2 PRECAST CONCRETE BOXES

926.2.2.1 PRECAST CONCRETE BOXES OF SPAN GREATER THAN 12-FT

A. The load rating analysis will be performed by the OSE.

B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, see BDM Section 923.

926.2.2.2 PRECAST BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT

A. Manufacturer shall submit the actual information about the dimensions and reinforcing bars/cage to the OSE along with the shop drawings before the placement of structure.
B. The load rating analysis will be performed by the OSE.

**926.2.3 PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES**

A. The load rating analysis will be performed by the manufacturer.

B. Load rating report shall be submitted along with the shop drawings before the placement of the precast units.

C. Use the design software to load rate the bridge.

**926.2.4 ANALYSIS OF CONCRETE BOX SECTIONS & FRAMES**

Unless more accurate soil data exits, calculate the rating based on a lateral pressure as specified in AASHTO.

Apply live load surcharge according to AASHTO.

Effect of soil-structure interaction shall be taken into account according to AASHTO.

Assume hinged connections between the walls and the top and bottom slabs unless there is adequate reinforcing steel continuous between the slab and the walls at the joint. In that case continuity between the slab and the walls can be assumed.

**926.3 LOAD RATING OF NON-BURIED STRUCTURES**

**926.3.1 GENERAL**

All structures including flat slabs, arch structures, frames, box sections, etc., having a fill or pavement material of less than 2'-0" [600 mm] on top of the structures shall be load rated according to the provisions of this Section.

All main structural members of the superstructure affected by live load shall be analyzed.

**926.3.2 HOW THE LOAD RATING SHALL BE DONE**

The designer shall analyze and load rate all spans which are designed to carry vehicular traffic.

The load rating analysis shall be based on the final design plans. At the inventory level, the load rating shall be equal to or greater than the design loading.

All members shall have actual net section and current conditions incorporated into the member’s analysis.
Bridge members designed as non-composite with the deck slab should be analyzed as non-composite.

All dead loads are to be calculated based on the actual field conditions. Future dead loads shall not be applied, unless directed otherwise.

Total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.

Live load distribution factors, as defined in the current AASHTO LRFD, shall be used.

**926.3.3 WHEN THE BRIDGE LOAD RATING SHALL BE DONE**

The load rating of new bridges shall be done as per following:

**926.3.3.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS**

For bridges designed under the Major or Minor Plan Development Process (PDP), perform the load rating and include the load rating report in the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

**926.3.3.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS**

For bridges designed under the Minimal Plan Development Process (PDP), perform the load rating and include the load rating report in the Stage 3 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 3 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

**926.3.3.3 BRIDGES DESIGNED UNDER DESIGN-BUILD PROCESS**

Unless otherwise indicated in the project scope, include the load rating report for bridges designed as part of a Design Build project with the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.
926.3.3.4 **BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL**

For bridges re-designed under a Value Engineering Change Proposal (VECP), perform a load rating of the altered bridge design and submit the load rating report to the District Construction Engineer (DCE) with the Final VECP submission. The DCE will supply this information to the District Bridge Engineer.

927 **LOAD RATING OF EXISTING BRIDGES**

927.1 **APPLICABILITY OF AASHTO DESIGN SPECIFICATIONS**

This Section is consistent with the current AASHTO LRFD Bridge Design Specifications and Standard Specifications for Highway Bridges (14th edition). Where this Section is silent, the current AASHTO LRFD Bridge Design Specifications or Standard Specifications for Highway Bridges shall govern for LRFR and LFR methods respectively.

927.2 **LOADS TO BE USED FOR LOAD RATING**

Existing bridges, starting preliminary design, as defined in BDM Section 905, after October 1, 2010, may be load rated at inventory and operating rating by either LFR or LRFR method with the prior approval of the bridge owner. When LFR method is used the load for inventory and operating rating shall be AASHTO HS20-44 [MS 18]. When LRFR method is used the load for inventory and operating rating shall be AASHTO HL-93.

All bridges shall also be load rated for four Ohio Legal Loads (2F1, 3F1, 4F1, and 5C1) at operating level using the same method of analysis as used for inventory and operating ratings above. The four Ohio legal loads (2F1, 3F1, 4F1 and 5C1) are given in Figure 903.

All legal loads used for analysis shall have transverse spacing, between centerline of wheels or wheel groups, of 6 ft. [1.830 m].

For long span bridges as defined in BDM Section 905, use the special load configurations given in BDM Section 917.

The inventory and operating ratings for the HL-93 or HS20-44 loading shall be expressed in terms of rating factors, rounded off to the nearest two decimal points. The operating ratings for each of the Ohio Legal Loads shall also be expressed in terms of rating factors of respective legal load, rounded off to the nearest two decimal points. The summary rating for all of the Ohio Legal Loads shall be the smallest rating factor of the four legal loads expressed as a percentage rounded off to the nearest 5 (i.e. multiplied by 100).

The owner may also require load rating to be done for special loads in addition to those specified here. The owner shall provide full configurations of the special load, including axle weights and
spacing, number of tires on each axle, tire gauges, overall dimensions of the load and the desired method of load rating (LRFR or LFR). All special loads to be analyzed as per BDM Section 916, unless specified otherwise by the owner.

927.3 LOAD RATING OF BRIDGES TO BE REHABILITATED

927.3.1 HOW THE LOAD RATING SHALL BE DONE

The designer shall analyze and load rate all spans which are designed to carry vehicular traffic.

The load rating analysis shall be based on the final design plans. At the inventory level, the load rating shall be equal to or greater than the design loading.

All members shall have actual net section and current conditions incorporated into the member’s analysis. Any known section losses, defects or damage to the existing structural members shall be considered in the rating analysis.

Bridge members designed as non-composite with the deck slab should be analyzed as non-composite.

Structures to be rehabilitated shall be analyzed using the original design plans, the actual field conditions, and all major changes in the final rehabilitation plans.

A complete review of all the available inspection information as well as a thorough site inspection of the existing bridge must be performed to establish the current conditions prior to proceeding with the analysis.

Future wearing surface dead loads shall not be applied, unless directed otherwise.

Total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.

Live load distribution factors, in accordance with the governing AASHTO specifications, shall be used.

For existing bridges, the rater should review the original design plans as the first source of information for material strengths and stresses. If the material strengths are not explicitly stated on the design plans, ODOT Construction and Material Specifications (CMS) applicable at the time of the bridge construction shall be reviewed. This may require investigations into old ASTM or AASHTO Material Specifications active at the time of construction.

Ultimate or yield strengths of materials shall be as specified on the original design plans, unless it is required in the scope of services to conduct specific tests to determine the material strengths.

Figures 906 and 907 provide general information about Allowable Stresses in bending and shear
and material strengths based on the year of construction. Any material stresses and strengths specified on the design plans shall supersede the values given in Figures 906 and 907.

The rater is cautioned to pay extra attention to the design plans and the year of construction, when determining material strengths for structures built during transition years of Figures 906 and 907 (e.g., for member type SS, years 1964-68, or 1988-93, etc.), as materials may have been substituted.

927.3.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE

The load rating of bridges to be rehabilitated shall be done as per following:

927.3.2.1 BRIDGES DESIGNED UNDER MAJOR OR MINOR PLAN DEVELOPMENT PROCESS

For bridges designed under the Major or Minor Plan Development Process (PDP), perform the load rating and include the load rating report in the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

927.3.2.2 BRIDGES DESIGNED UNDER MINIMAL PLAN DEVELOPMENT PROCESS

For bridges designed under the Minimal Plan Development Process (PDP), perform the load rating and include the load rating report in the Stage 3 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 3 and prior to contract sale, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.

927.3.2.3 BRIDGES DESIGNED UNDER DESIGN-BUILD PROCESS

Unless otherwise indicated in the project scope, include the load rating report for bridges designed as part of a Design Build project with the Stage 2 Detail Design Submission. When design modifications that affect the previously submitted load rating analysis occur after Stage 2, revise and resubmit the load rating report to the District Project Manager. The District Project Manager will forward the final load rating report to the District Bridge Engineer.
927.3.2.4  **BRIDGES DESIGNED UNDER VALUE ENGINEERING CHANGE PROPOSAL**

For bridges re-designed under a Value Engineering Change Proposal (VECP), perform a load rating of the altered bridge design and submit the load rating report to the District Construction Engineer (DCE) with the Final VECP submission. The DCE will supply this information to the District Bridge Engineer.

927.3.3  **LOAD RATING OF BURIED BRIDGES**

927.3.3.1  **CAST-IN-PLACE STRUCTURES**

A. Cast-in-place bridges shall be load rated by the designer of the bridge.

B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, also see BDM Section 923.

927.3.3.2  **PRECAST BOXES OF SPAN GREATER THAN 12-FT.**

A. The load rating analysis will be performed by the OSE.

B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, see BDM Section 923.

927.3.3.3  **PRECAST BOXES OF SPAN EQUAL TO OR LESS THAN 12-FT**

The load rating analysis will be performed by the OSE.

927.3.3.4  **PRECAST FRAMES, ARCHES, AND CONSPANS & BEBO TYPE STRUCTURES**

A. The load rating analysis for any new or replacement precast sections will be performed by the manufacturer; otherwise the load rating analysis will be performed as per the scope of services.

B. Load rating report shall be submitted along with the shop drawings before the placement of the units.

C. Use the design software to load rate the bridge.

927.3.3.5  **ANALYSIS OF CONCRETE BOX SECTIONS & FRAMES**

Unless more accurate soil data exits, calculate the rating based on a lateral pressure as specified in AASHTO.
Apply live load surcharge according to AASHTO.

Effect of soil-structure interaction shall be taken into account according to AASHTO.

Assume hinged connections between the walls and the top and bottom slabs unless there is adequate reinforcing steel continuous between the slab and the walls at the joint. In that case continuity between the slab and the walls can be assumed.

**927.3.4 LOAD RATING OF NON-BURIED STRUCTURES**

All structures including flat slabs, arch structures, frames, box sections, etc., having a fill or pavement material of less than 2'-0" [600 mm] on top of the structures shall be load rated according to the provisions of BDM Sections 911 through 925 (as applicable).

**927.4 LOAD RATING OF EXISTING BRIDGES WITH NO REPAIR PLANS**

**927.4.1 HOW THE LOAD RATING SHALL BE DONE**

The rater shall analyze and load rate all spans which are designed to carry vehicular traffic.

Existing structures shall be analyzed using the information from the original design plans and the actual field conditions.

A complete review of all the available inspection information as well as a thorough site inspection of the existing bridge must be performed to establish the current conditions prior to proceeding with the analysis.

The bridges rated using design plans shall be noted as such in the load rating report.

Allowable stresses for the working stress and the ultimate or yield strengths of materials for Load Factor ratings shall be as specified on the original design plans, unless it is required in the scope of services to conduct specific tests to determine the material strengths.

For existing bridges, the rater should review the original design plans as the first source of information for material strengths and stresses. If the material strengths are not explicitly stated on the design plans, ODOT Construction and Material Specifications (CMS) applicable at the time of the bridge construction shall be reviewed. This may require investigations into old ASTM or AASHTO Material Specifications active at the time of construction.

Total thickness of the composite concrete slab shall be used in load rating for the calculations of section properties. Do not subtract for the monolithic wearing surface.

Figures 906 and 907 provide general information about ODOT Allowable Stresses in bending and shear and material strengths based on the year of construction.
The rater is cautioned to pay extra attention to the design plans and the year of construction, when determining material strengths for bridges built during transition years of Figures 906 and 907 (e.g., for member type SS, years 1964-68, or 1988-93, etc.), as materials may have been substituted.

Any material stresses and specifications specified on the design plans shall supersede the values given in Figures 906 and 907.

927.4.2 WHEN THE BRIDGE LOAD RATING SHALL BE DONE

The load rating of existing bridges shall be done as per the Scope of Services.

927.4.3 LOAD RATING OF EXISTING BURIED BRIDGES

A. The load rating analysis will be performed as per the Scope of Services.
B. Unless specified otherwise, structures shall be load rated for the Loads as per BDM Section 927.3.3.

927.4.4 LOAD RATING OF NON-BURIED STRUCTURES

All structures including flat slabs, arch structures, frames, box sections, etc., having a fill or pavement material of less than 2'-0" [600 mm] on top of the structures shall be load rated according to the provisions of BDM Sections 911 through 925 (as applicable).

928 LOAD RATING OF NON-ODOT BRIDGES

Provisions of BDM Section 900 may also be applied to load rating of Non-ODOT buried and non-buried bridges at the discretion of the respective bridge owners.

The load rating files and report of a Non-ODOT bridge shall be submitted to the respective bridge owner. The bridge owner shall keep the bridge load rating report in bridge file for future reference and use.

Based on the field conditions and the load rating calculations, if the rating engineer determines a bridge should be posted for reduced load capacity, the engineer must immediately forward the recommendation to the respective bridge owner. Applicable portions of Section 918, Bridge Posting for Reduced Load Limits may be followed.

It is the responsibility of the respective bridge owner (or designated consultant/rating engineer) to ensure that the load rating information is finally updated in the ODOT BMS.
REFERENCES


C. AASHTO, 1989, “Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges.”


E. BRASS-Culvert software developed by the Wyoming Department of Transportation (PO Box 1708, Cheyenne, WY 82003).


AASHTO HS20-44 LOADING

Figure 901
AASHTO HL93 LOADING

Figure 902
<table>
<thead>
<tr>
<th>Load Designation</th>
<th>Load Configuration</th>
<th>Gross Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>2F1</td>
<td><img src="image1" alt="Configuration" /></td>
<td>15 Tons</td>
</tr>
<tr>
<td>3F1</td>
<td><img src="image2" alt="Configuration" /></td>
<td>23 Tons</td>
</tr>
<tr>
<td>4F1</td>
<td><img src="image3" alt="Configuration" /></td>
<td>27 Tons</td>
</tr>
<tr>
<td>5C1</td>
<td><img src="image4" alt="Configuration" /></td>
<td>40 Tons</td>
</tr>
<tr>
<td>Load Designation</td>
<td>Load Configuration</td>
<td>Gross Weight</td>
</tr>
<tr>
<td>------------------</td>
<td>--------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>2F1</td>
<td><img src="image1" alt="Figure 2F1" /></td>
<td>13.608 Metric Tons</td>
</tr>
<tr>
<td>3F1</td>
<td><img src="image2" alt="Figure 3F1" /></td>
<td>20.865 Metric Tons</td>
</tr>
<tr>
<td>4F1</td>
<td><img src="image3" alt="Figure 4F1" /></td>
<td>24.494 Metric Tons</td>
</tr>
<tr>
<td>5C1</td>
<td><img src="image4" alt="Figure 5C1" /></td>
<td>36.287 Metric Tons</td>
</tr>
</tbody>
</table>

Figure 904
Bridge Ahead Sign (R-79)  Bridge Weight Limit Sign (R-78)
<table>
<thead>
<tr>
<th>Material of Construction</th>
<th>Type of Rating</th>
<th>Year of Construction</th>
<th>Fy / Fc' (ksi)</th>
<th>Fy / Fc' (MPa)</th>
<th>Inventory (ksi)</th>
<th>Inventory (MPa)</th>
<th>Operating (ksi)</th>
<th>Operating (MPa)</th>
<th>Posting (ksi)</th>
<th>Posting (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel (SS),(CSC)</td>
<td>&lt; 1900</td>
<td>26.00</td>
<td>179</td>
<td>14.00</td>
<td>97</td>
<td>19.00</td>
<td>131</td>
<td>19.00</td>
<td>131</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1901 To 1930</td>
<td>30.00</td>
<td>207</td>
<td>16.00</td>
<td>110</td>
<td>22.00</td>
<td>152</td>
<td>22.00</td>
<td>152</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1931 To 1965</td>
<td>33.00</td>
<td>228</td>
<td>18.00</td>
<td>124</td>
<td>25.00</td>
<td>172</td>
<td>25.00</td>
<td>172</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1966 To 1990</td>
<td>36.00</td>
<td>248</td>
<td>20.00</td>
<td>138</td>
<td>27.00</td>
<td>186</td>
<td>27.00</td>
<td>186</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1991 To Date</td>
<td>50.00</td>
<td>345</td>
<td>27.00</td>
<td>186</td>
<td>37.50</td>
<td>259</td>
<td>37.50</td>
<td>259</td>
<td></td>
</tr>
<tr>
<td>Reinforcing Steel (RC)</td>
<td>&lt; 1935</td>
<td>32.00</td>
<td>221</td>
<td>16.00</td>
<td>110</td>
<td>24.00</td>
<td>165</td>
<td>24.00</td>
<td>165</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1936 To 1950</td>
<td>36.00</td>
<td>248</td>
<td>18.00</td>
<td>124</td>
<td>27.00</td>
<td>186</td>
<td>27.00</td>
<td>186</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1951 To 1983</td>
<td>40.00</td>
<td>276</td>
<td>20.00</td>
<td>138</td>
<td>30.00</td>
<td>207</td>
<td>30.00</td>
<td>207</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1984 To Date</td>
<td>60.00</td>
<td>414</td>
<td>24.00</td>
<td>165</td>
<td>36.00</td>
<td>248</td>
<td>36.00</td>
<td>248</td>
<td></td>
</tr>
<tr>
<td>Prestress. Strands (Fs')</td>
<td>All Years</td>
<td>270.0</td>
<td>1862</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Cast-in-Place Reinf. Conc. (Compression in Bending) (RC),(CSC)</td>
<td>&lt; 1930</td>
<td>2.00</td>
<td>14</td>
<td>0.70</td>
<td>5</td>
<td>1.30</td>
<td>9</td>
<td>1.30</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1931 To 1950</td>
<td>3.00</td>
<td>21</td>
<td>1.00</td>
<td>7</td>
<td>1.50</td>
<td>10</td>
<td>1.50</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1951 To 1980</td>
<td>4.00</td>
<td>28</td>
<td>1.30</td>
<td>9</td>
<td>2.00</td>
<td>14</td>
<td>2.00</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1981 To Date</td>
<td>4.50</td>
<td>31</td>
<td>1.50</td>
<td>10</td>
<td>2.20</td>
<td>15</td>
<td>2.20</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Prestressed Concrete (Fc') (PSC),(CPS)</td>
<td>All Years</td>
<td>5.50</td>
<td>38</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Cast-in-Place Comp. Slab for Prestress. Conc. (Fc') (CPS)</td>
<td>All Years</td>
<td>4.00</td>
<td>28</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Timber (fb) (TMB)</td>
<td>All Years</td>
<td>1.60</td>
<td>11</td>
<td>2.128</td>
<td>15</td>
<td>2.128</td>
<td>15</td>
<td>2.128</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Cast-in-Place Slab for Composite Reinforced Concrete</td>
<td>&lt; 1930</td>
<td>2.00</td>
<td>14</td>
<td>0.70</td>
<td>5</td>
<td>1.30</td>
<td>9</td>
<td>1.30</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1931 To 1950</td>
<td>3.00</td>
<td>21</td>
<td>1.00</td>
<td>7</td>
<td>1.50</td>
<td>10</td>
<td>1.50</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1951 To 1980</td>
<td>4.00</td>
<td>28</td>
<td>1.30</td>
<td>9</td>
<td>2.00</td>
<td>14</td>
<td>2.00</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1981 To Date</td>
<td>4.50</td>
<td>31</td>
<td>1.50</td>
<td>10</td>
<td>2.20</td>
<td>15</td>
<td>2.20</td>
<td>15</td>
<td></td>
</tr>
</tbody>
</table>

Figure 906
### CUSTOM ALLOWABLE STRESSES IN SHEAR

<table>
<thead>
<tr>
<th>Material of Construction</th>
<th>Type of Rating</th>
<th>Year of Construction</th>
<th>Fy / Fc’ (ksi)</th>
<th>Fy / Fc’ (MPa)</th>
<th>Inventory (ksi)</th>
<th>Inventory (MPa)</th>
<th>Operating (ksi)</th>
<th>Operating (MPa)</th>
<th>Posting (ksi)</th>
<th>Posting (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Steel (SS),(CSC)</td>
<td>&lt; 1900</td>
<td>26.00</td>
<td>179</td>
<td>8.50</td>
<td>59</td>
<td>11.50</td>
<td>79</td>
<td>11.50</td>
<td>79</td>
<td></td>
</tr>
<tr>
<td>1901 To 1930</td>
<td>30.00</td>
<td>207</td>
<td>9.50</td>
<td>66</td>
<td>13.50</td>
<td>93</td>
<td>13.50</td>
<td>93</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1931 To 1965</td>
<td>33.00</td>
<td>228</td>
<td>11.00</td>
<td>76</td>
<td>15.00</td>
<td>103</td>
<td>15.00</td>
<td>103</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1966 To 1990</td>
<td>36.00</td>
<td>248</td>
<td>12.00</td>
<td>83</td>
<td>16.00</td>
<td>110</td>
<td>16.00</td>
<td>110</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1991 To Date</td>
<td>50.00</td>
<td>345</td>
<td>17.00</td>
<td>117</td>
<td>22.50</td>
<td>155</td>
<td>22.50</td>
<td>155</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforcing Steel (RC)</td>
<td>&lt; 1935</td>
<td>32.00</td>
<td>221</td>
<td>16.00</td>
<td>110</td>
<td>24.00</td>
<td>165</td>
<td>24.00</td>
<td>165</td>
<td></td>
</tr>
<tr>
<td>1936 To 1950</td>
<td>36.00</td>
<td>248</td>
<td>18.00</td>
<td>124</td>
<td>27.00</td>
<td>186</td>
<td>27.00</td>
<td>186</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1951 To 1983</td>
<td>40.00</td>
<td>276</td>
<td>20.00</td>
<td>138</td>
<td>30.00</td>
<td>207</td>
<td>30.00</td>
<td>207</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1984 To Date</td>
<td>60.00</td>
<td>414</td>
<td>24.00</td>
<td>165</td>
<td>36.00</td>
<td>248</td>
<td>36.00</td>
<td>248</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cast-in-Place Reinforced Conc. (RC),(CSC)</td>
<td>&lt; 1930</td>
<td>2.00</td>
<td>14</td>
<td>0.70</td>
<td>5</td>
<td>1.30</td>
<td>9</td>
<td>1.30</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>1931 To 1950</td>
<td>3.00</td>
<td>21</td>
<td>1.00</td>
<td>7</td>
<td>1.50</td>
<td>10</td>
<td>1.50</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1951 To 1980</td>
<td>4.00</td>
<td>28</td>
<td>1.30</td>
<td>9</td>
<td>2.00</td>
<td>14</td>
<td>2.00</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1981 To Date</td>
<td>4.50</td>
<td>31</td>
<td>1.50</td>
<td>10</td>
<td>2.20</td>
<td>15</td>
<td>2.20</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestressed Concrete (Fc’) (PSC),(CPS)</td>
<td>All Years</td>
<td>5.50</td>
<td>38</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Timber (Horizontal Shear Stress) (fb) (TMB)</td>
<td>All Years</td>
<td>-</td>
<td>-</td>
<td>0.09</td>
<td>1</td>
<td>0.12</td>
<td>1</td>
<td>0.12</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>SFN</td>
<td>BRIDG Number</td>
<td>DISTRICT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>--------------</td>
<td>----------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Feature Intersected:**

**Special Assumptions & Comments:**

**Rating & Analysis Option:** Select from list on the left where appropriate:

<table>
<thead>
<tr>
<th>SFN:</th>
<th>BRIDGE NO.:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Structure Rating Summary**

<table>
<thead>
<tr>
<th>Loading &amp; Rating Type</th>
<th>Rating Factor - RF (Rounded to 2 decimal points)</th>
<th>Rating Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inventory Current Design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Operating Current Design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ohio Legal - 2/1</td>
<td>Ohio legal Loads Overall Minimum Rating Factor</td>
<td></td>
</tr>
<tr>
<td>Ohio Legal - 3/1</td>
<td>Ohio legal Loads Overall Controlling Truck</td>
<td></td>
</tr>
<tr>
<td>Ohio Legal - 4/1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ohio Legal - 5/1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rated By, PE#</td>
<td>Reviewed By, PE#</td>
<td>Report Date</td>
</tr>
<tr>
<td>Agency/Firm</td>
<td>Phone Number</td>
<td>Email</td>
</tr>
</tbody>
</table>

BR 100 (REV 2010)

Figure 908
### Plastic Moment Capacity of Aluminum Structural Plate with and without Stiffening Ribs

<table>
<thead>
<tr>
<th>Uncoated thickness in inches (cm)</th>
<th>Plastic Moment - Mp in kip.ft / ft (kN.m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Structural plate only</td>
</tr>
<tr>
<td>0.125 (0.3175)</td>
<td>2.6 (11.565)</td>
</tr>
<tr>
<td>0.150 (0.381)</td>
<td>3.2 (14.234)</td>
</tr>
<tr>
<td>0.175 (0.445)</td>
<td>3.7 (16.458)</td>
</tr>
<tr>
<td>0.200 (0.508)</td>
<td>4.2 (18.683)</td>
</tr>
<tr>
<td>0.225 (0.572)</td>
<td>4.8 (21.351)</td>
</tr>
<tr>
<td>0.250 (0.635)</td>
<td>5.3 (23.576)</td>
</tr>
</tbody>
</table>

APPENDIX – MISC. BRIDGE INFORMATION

APPENDIX PURPOSE

The Bridge Design Manual’s appendix serves three purposes.

A. One is to serve as a repository for special plan notes that are infrequently used or subject to frequent revision. These notes are generally large and detailed documents. When a bridge design requires the use of appendix notes one of two methods should be used to incorporate the notes into the project plans. One, the designer transfers the notes to plan sheets for inclusion into the bridge plans. The second method is to treat the note as an un-numbered proposal note. This method requires the designer to include with the bid item(s) a reference to the proposal and supply electronic versions, or typed hard copies, of the note with the final plan submission. If the proposal note method is used, the designer shall ensure the notes are presentable, that it is clear what notes are to be used as proposal notes, and that the agency receiving the completed plans understands the notes must be included in the project’s actual proposal. The choice of methods is the option of the owner.

B. The second purpose is to serve as a historical archive for old plan notes, old general notes or old proposal notes which are no longer active or not recommended for use.

C. The third purpose is to serve a repository for special bridge policy criteria and other items of similar concept.
AN-1 ACCREDITATION PROCEDURE FOR MSE WALLS

1.0 DESCRIPTION
This document describes the process by which the Department will review and approve mechanically stabilized earth (MSE) wall systems that are listed as “Accredited MSE Wall Systems” in Supplemental Specification 840.

2.0 ACCREDITATION
Submit the following information to the Office of Structural Engineering.

A. A copy of the HITEC evaluation report for the MSE wall system. HITEC is the Highway Innovative Technology Evaluation Center. Identify any aspects of the MSE wall system that have changed since the HITEC evaluation was performed.
B. A letter from the MSE wall supplier that states the following:
   1. The MSE wall supplier is aware of the following Department publications that pertain to MSE walls: Supplemental Specification 840, the Bridge Design Manual, Location and Design Manuals, Construction Inspection Manual of Procedures, and Construction and Material Specifications.
   2. The MSE wall supplier is responsible for the design for internal stability of MSE walls using the system and that the design will be according to the latest AASHTO Standard Specifications or the latest AASHTO LRFD Bridge Design Specifications, whichever specification is indicated on the project plans.
   3. The design of the MSE wall system will be in accordance with Supplemental Specification 840.
C. Provide the names of at least two engineers who will design and check MSE wall working drawings and calculations prepared by the MSE wall supplier. Also include their Ohio Professional Engineer registration number and contact information (address, phone number, e-mail address, etc.). This information is required to conform with Ohio Revised Code and Department policy for prequalification of design consultants.
D. Attach a recent set of sample working drawings and calculations.
E. Provide design drawings and structural design calculations for all standard facing panels.
F. Provide details and materials specifications for standard connections, related hardware and soil reinforcement used with the MSE wall system.
G. Provide details of typical frames and frame connections used with the MSE wall system to avoid obstructions in the reinforced soil zone.
H. Provide information on differential settlements that the MSE wall system is able to accommodate without distress.
I. Provide all laboratory and field test data for the MSE wall system. This should include, at a minimum, the following: durability test data for the facing panels; connection strength test data; soil reinforcement pullout resistance test data and failure modes; for metallic soil reinforcements, the corrosion coating requirements and related corrosion test data; for welded wire mesh soil reinforcement, the weld test data and weld requirements; for geogrid soil reinforcements, creep performance and related test data. Also provide other test data unique to the MSE wall system.
J. List all precast concrete manufacturers that will manufacture the facing panels for the MSE wall system, and confirm that they have been certified according to Supplemental Specification 1073.

K. Provide the precast concrete manufacturer’s quality control plan that applies to the MSE wall system. Also provide the criteria for acceptance or rejection of precast concrete facing panels.

L. Provide the quality control plan that applies to materials, other than concrete, that are part of the MSE wall system. Also provide the criteria for acceptance or rejection of those materials. This quality control plan does not apply to the soil or granular material that is used to construct the MSE wall system.

M. Provide detailed repair methods for replacing full panels, repairing parts of panels, and stopping the loss of select granular backfill from between the panel joints.

3.0 LOSS OF ACCREDITATION

The Department will remove an MSE wall system from the list of Accredited MSE Wall Systems in Supplemental Specification 840 if the MSE wall system has substantially or repeatedly failed to perform satisfactorily in the field. An MSE wall system which has been removed from the list will not be placed back on the list until the MSE wall supplier identifies the reason for the failure and the problem has been corrected to the satisfaction of the Department. The Department may consider the experiences of other state or governmental agencies in this decision.

3.0 MODIFICATIONS TO AN ACCREDITED MSE WALL SYSTEM

The Department recognizes that MSE wall suppliers may wish to modify certain details of their MSE wall system after the Department’s review and accreditation. Notify the Department of any proposed modifications to an Accredited MSE Wall System and resubmit all the submittal items listed in “2.0 Accreditation” that are affected by the modifications. The Department will review the proposed modifications and determine if the MSE wall system will remain on the list of Accredited MSE Wall Systems with the proposed modifications. MSE wall systems that are modified without notifying the Department will be removed from the list of Accredited MSE Wall Systems.
PAGES APPENDIX – 4 THROUGH APPENDIX – 36
HAVE BEEN INTENTIONALLY DELETED
AN-5 3 COAT SHOP PAINT SYSTEM IZEU

Un-numbered plan note to define specification requirements for shop application of the 3 coat IZEU paint system. To use this note The 863 item shall be AS PER PLAN.

As example:
Item 863  Lump Sum  STRUCTURAL STEEL MEMBERS, LEVEL ?, AS PER PLAN

The note AN-4 follows as the AS PER PLAN specifications to have a a three (3) coat shop paint system applied.

1.0 DESCRIPTION

In addition to the requirements of Supplemental Specification 863, this item shall consist of furnishing all necessary labor, materials and equipment to clean, apply a three(3) coat shop applied IZEU system for Item 863 Structural Steel, including requirements for field cleaning and coating of surfaces only prime coated at the shop, and methods of repair for surfaces damaged in shipping, handling and erecting the structural steel and any other damages during construction. Section 863.29 and 863.30 shall not apply.

This specification shall also include galvanizing, CMS 711.02, of all nuts, washers, bolts, anchor bolts, and any other structural steel items requiring galvanizing as part of item 863.

All shop painting shall be applied in a structural steel fabrication shop having permanent buildings per SS863.07 and pre qualified at the same SS863 level as the structural steel fabricator. The painter is under the supervision of a QCPS and is the SS863 Fabricator, the field painting sub-contractor performing touch up work in the field and or shop coating at the 863 Fabricator’s facility or an independent painter meeting the qualifications of SSPC QP3 with facilities evaluation and acceptance by the Department.

2.0 MATERIAL

A. A three coat paint system consisting of an:

1. Inorganic Zinc Prime Coat meeting the requirements of CMS 708.17

2. Epoxy Intermediate Coat meeting the requirements of Supplemental Specification 910 entitled "OZEU Structural Steel Paint”.

3. Urethane Finish Coat meeting the requirements of Supplemental Specification 910 entitled "OZEU Structural Steel Paint”.

B. A tie coat consisting of an Epoxy Intermediate Coat, meeting the requirements of Supplemental Specification 910, “Epoxy Intermediate Coat” and thinned 50%, by volume,
with a thinner as recommended by the paint manufacturer.

Approved paint, items A and B, shall be from one manufacturer, regardless of shop or field application.

3.0 PRE-PAINT CONFERENCE

A pre-paint conference shall be held during the Pre-Fabrication meeting per 863.081. Attendees may include the Contractor, field painting sub-contractor, structural steel erector, fabricator, quality control paint specialist, the Engineer, and a representative from the Office of Structural Engineering. The discussion shall include methods of operation, quality control, validation of QCPS qualifications, transportation, erection and repairs methods necessary to achieve the specified system.

4.0 QUALITY CONTROL

4.1 QUALITY CONTROL SPECIALIST

The QCPS (Quality Control Paint Specialist) required per SS863, shall be responsible for all quality control requirements of this specification. The Contractor and the Fabricator shall make the testing equipment specified in SS863.29, available to the QCPS, the QA inspector and the Engineer.

QCPS will be required in the shop and in the field. At the pre-paint meeting, the Contractor shall designate one shop QCPS and one field QCPS.

4.2 QUALITY CONTROL POINTS (QCP)

Quality control points (QCP) are points in time when one phase of the work is complete and ready for inspection by the fabricator’s QCPS and QA Inspector (shop application) or Engineer (field application). The next operational step shall not proceed unless the QCP has been accepted or QA inspection waived by the QA Inspector (shop application) or Engineer (field application). At these points the Fabricator shall afford access to inspect all affected surfaces. If Inspection indicates a deficiency, that phase of the work shall be corrected in accordance with these specifications prior to beginning the next phase of work. Discovery of defective work or material after a Quality Control Point is past or failure of the final product before final acceptance, shall not in any way prevent rejection or obligate the Department to final acceptance.
<table>
<thead>
<tr>
<th>Quality Control Points (QCP)</th>
<th>PURPOSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Solvent Cleaning</td>
<td>Remove asphaltic cement, oil, grease, salt, dirt, etc.</td>
</tr>
<tr>
<td>B. Grinding Edges</td>
<td>Remove sharp corners per AWS.</td>
</tr>
<tr>
<td>C. Abrasive Blasting</td>
<td>Blast surface to receive paint, including repair fins, tears, slivers or sharp edges.</td>
</tr>
<tr>
<td>D. Prime Coat Application</td>
<td>Check surface cleanliness, apply prime coat, check coating thickness</td>
</tr>
<tr>
<td>E. Intermediate Coat Application</td>
<td>Check surface cleanliness, apply intermediate coat, check coating thickness</td>
</tr>
<tr>
<td>F. Finish Coat Application</td>
<td>Check surface cleanliness, apply finish coat, check coating thickness</td>
</tr>
<tr>
<td>G. Visual Inspection</td>
<td>Visually inspect paint before shipment of steel and check of total system thickness.</td>
</tr>
<tr>
<td>H. Repair of Damage Areas</td>
<td>Check for damage areas after completion of structure and repeat QCP 1 - 7 for damage areas.</td>
</tr>
<tr>
<td>I. Final Review</td>
<td>Clean structure as per QCP#1. Visually inspect system for acceptance.</td>
</tr>
</tbody>
</table>

A. **Solvent Cleaning (QCP #1)**

The steel shall be solvent cleaned were necessary to remove all traces of asphaltic cement, oil, grease, diesel fuel deposits, and other soluble contaminants per SSPC-SP 1 Solvent Cleaning. Under no circumstances shall any abrasive blasting be done to areas with asphaltic cement, oil, grease, or diesel fuel deposits. Steel shall be allowed to dry before blast cleaning begins.

B. **Grinding Edges (QCP #2)**

All corners of thermally cut or sheared edges shall have a $1/16\text{O}[2 \text{ mm}]$ radius or equivalent flat surface at a suitable angle. Thermally cut material thicker than $1\frac{1}{2}\text{O}[38 \text{ mm}]$ shall have the sides ground to remove the heat effected zone, as necessary to achieve the specified surface profile.

C. **Abrasive Blasting (QCP #3)**

All steel to be painted shall be blast cleaned according to SSPC-SP10. Steel shall be maintained in a blast cleaned condition until it has received a prime coat of paint.
Metalized or Galvanized steel, and other surfaces not intended to be painted, shall be covered and protected to prevent damage from blasting and painting operations. Any adjacent coatings damaged during the blasting operation shall be repaired at the fabricator's expense. The abrasive shall produce an angular profile. After each use and prior to reuse, the abrasive shall be cleaned of paint chips, rust, mill scale and other foreign material by equipment specifically designed for such cleaning.

Abrasives shall also be checked for oil contamination before use. A small sample of abrasives shall be added to ordinary tap water. Any detection of a oil film on the surface of the water shall be cause for rejection. The QCPS shall perform and record this test at the start of each shift.

The surface profile shall be a minimum of 1.5 mils [40 μm] and a maximum of 3.5 mils [90 μm]. The QCPS shall record the surface profile with replica tape ASTM D4417-93 method C.

For **Automated blasting process**:

Five (5) each recorded readings at random locations on one member for 20% of the main members or one beam per shift (which ever is greater) and one (1) recorded reading for 10% of all secondary material.

For **Manual blasting process**:

Five (5) each recorded readings at random locations for each main member and one (1) recorded reading for 25% of all secondary material.

Abrasives of a size suitable to develop the required surface profile shall be used. Any abrasive blasting which is done when the steel temperature is less than 5°F [3°C] above the dew point shall be re-blasted when the steel temperature is at least 5°F [3°C] above the dew point. The QCPS shall record temperature and dew point prior to blasting and at the start of each shift.

All abrasives and residue shall be removed from all surfaces to be painted with a vacuum cleaner equipped with a brush-type cleaning tool, or by double blowing. All blast cleaned steel shall be kept dust free, dry and shall be prime coated within 24 hours. The QCPS shall perform and record the following test to ensure that the compressed air is not contaminated: blow air from the nozzle for 30 seconds onto a white cloth or blotter held in a rigid frame. If any oil or other contaminants are present on the cloth or blotter, abrasive blasting shall be suspended until the problem is corrected and the operation is verified by a repeated test. This test shall be done prior to blowing and at the start of each shift.

Ablasive blasting and painting may take place simultaneously as long as abrasive blasting debris and/or dust by the blowing operation does not come in contact with freshly painted surfaces. Work areas for blasting and painting shall be physically separated to eliminate contamination of the priming operation.

All fins, tears, slivers and burred or sharp edges that are present on any steel member or that
appear after the blasting operation shall be conditioned per ASTM A6 and the area re-blasted to provide the specified surface profile. Welding repairs shall only be performed by the SS863 Fabricator.

**D. thru F. Shop Coat Application (QCP # 4, 5 and 6)**

The surfaces to be painted shall be clean and dry. Paint shall not be applied in rain, snow, fog or mist, or to frosted or ice-coated surfaces. After QCP #3 has been accepted, prime painting shall be completed before the cleaned surfaces have degraded from the prescribed standards, but in every case within 24 hours. The QCPS shall record the time between blasting, priming, intermediate epoxy coat and Final urethane coating. Failure to prime coat the within 24 hours will require re-blasting before prime coating. The QCPS shall record that the paint is applied when the ambient temperature and humidity are as specified. IZEU coatings shall be applied by spray methods. The paint may be thinned for spraying. The type of thinner and the amount used shall be as recommended by the printed instructions of the manufacturer.

Before the paint is applied, it shall be mixed to a uniform consistency and maintained during its application. Primer shall be spray applied and continuously agitated by an automated agitation system (hand held mixers are not allowed) during application. The paint shall be mixed with a high shear mixer. Paddle mixers or paint shakers shall not be used. Paint shall also not be mixed or kept in suspension by means of an air stream bubbling under the surface.

The IZEU coatings shall be applied in a neat workmanlike manner as a continuous film of uniform thickness which is free of holidays, pores, runs or sags. Spray application must produce a wet coat at all times; the deposition of semi-dry particles on the surface is not allowed. The Fabricator shall take precaution to prevent contamination of surfaces that have been prepared for painting and surfaces freshly painted. The IZEU coatings shall be applied within the shop. The steel shall not be handled unnecessarily or removed from the shop until paint has dried sufficiently to allow thickness gaging and to resist being marred in handling and shipping.

The IZEU coatings shall coat all surfaces including insides of holes, behind stiffener clips and the prime coating applied to contact surfaces of connection or splice material which are to be fastened with shop or field bolts. Surfaces which are to be imbedded in concrete and surfaces within 2 inches [50 mm] of field welds other than those attaching intermediate or end cross frames to beams or girders must only receive a mist coat not less that 0.5 mils[12.5 Fm] nor more than 1.5 mils [38 Fm]. Pins, pin holes and contact surfaces of bearing assemblies, except those containing self-lubricating bronze inserts, must be painted with one coat of prime paint. Erection marks must be applied after the IZEU coating is dry, using a thinned paint of a type that is compatible with the IZEU coatings. The coating of intermediate or end cross frames within 2 inches [50 mm] of the girder or beam may be field coated as described under “Coating of Faying Surfaces.”

The QCPS must record the actual dry film thickness for the IZEU coatings as specified. Thick primer coat films must be reduced by screening, sanding, or sweep blasting. Any re-coating of prime paint that has cured longer than 24 hours with prime paint must be done as recommended.
by the paint manufacturer's printed instructions. If "mud cracking" occurs, the affected area must be scraped to soundly bonded paint and the area re-coated. Uncured paint damaged by rain, snow or condensation must be permitted to dry; the damaged paint must then be removed and the surface repainted.

Each coating must be adequately cured before the next coat is applied. This curing time must be not less than that recommended by the paint manufacturer's printed instructions.

G. Visual Inspection (QCP #7)

Visually inspect paint before shipment of steel and check of total system thickness.

H. Repair of Damaged Areas (QCP #8)

Damaged areas of paint and areas which do not comply with the requirements of this specification must have the paint removed and all defects corrected. The steel must then be retextured to a near white condition to produce a profile of between 1.0 to 3.5 mils [25 to 90 Fm]. This profile must be measured immediately prior to the application of the prime coat to insure that the profile is not destroyed during the feathering procedure. All abrasive blasting and painting must be done as specified herein.

The existing paint must be feathered to expose a minimum of ½ inch [13 mm] of each coat.

During the reapplication of the paint, care must be used to insure that each coat of paint is only applied within the following areas. The prime coat must only be applied to the surface of the bare steel and the existing prime coat, which has been exposed by feathering. The prime coat must not be applied to the adjacent intermediate coat. The intermediate coat must only be applied to the new prime coat and the existing feathered intermediate coat. The intermediate coat and the existing finish coat which has been feathered or lightly sanded. The finish coat must not extend beyond the areas which has been feathered or lightly sanded.

All repairs should be made in a manner to blend the patched area with the adjacent coating. The finished surface of the patched area must have a smooth even profile with the adjacent surface. The first repair area must be used as a test section and no more repairs made until the methods are approved. The Engineer must approve field repairs, while the Office of Structural Engineering approves shop repairs.

Field primer repairs must be made with organic zinc of the same manufacturer as the shop system meeting SS910.

The first two coats must be applied by brush. The finish coat must be applied by either brush or spray. During the application of the prime coat, the paint should be continuously mixed. It may be necessary to make several applications in order to achieve the proper thickness for each coat.
Damaged paint which will be inaccessible for coating after erection must be repaired and re-coated prior to erection.

In order to minimize damage to the painted steel, concrete splatter and form leakage must be washed from the surface of the steel shortly after the concrete is placed and before it is dry. If the concrete dries, it must be removed and the paint repaired.

Temporary attachments, supports for scaffolding and finishing machine or forms must not damage the coating system. In particular, sufficient size support pads must be used on the fascias where bracing is used.

After the erection work has been completed, including all connections and the approved repair of any damaged beams, girders or other steel members, and the deck has been placed, the Contractor and Engineer must inspect the structure for damaged paint. (QCP #9). Damaged areas must be repaired by repeating QCPs #1 to #8. The Engineer may accept alternate methods of repair from the Contractor, if the repair is accompanied by written acceptance by the paint manufacturer’s Technical Director. At the completion of construction, the paint must be undamaged and the surfaces free from grease, oil, chalk marks, paint, concrete splatter or other silage.

5.0 TESTING EQUIPMENT

The Fabricator must provide the QCSP inspector the following testing equipment in good working order for the duration of the project. When the Fabricator's people are working at different locations simultaneously, additional test equipment must be provided for each crew for the type of work being performed. When test equipment is not available, no work must be performed.

A. One Spring micrometer and 3 (unless otherwise specified on plans) rolls of extra-coarse replica tape.

B. One (Positector 2000 or 6000, Quanix 2200, or Elcometer A345FB11) and the calibration plates, 1.5 - 8 mils and 10-25 mils [38-200 Fm and 250-625 Fm] as per the NBS calibration standards in accordance with ASTM D-1186.

C. One Sling Psychrometer including Psychometric tables - Used to calculate relative humidity and dew point temperature.

D. Two steel surface thermometers accurate within 2E F [1E C] or One portable infrared thermometer available from:

Model: Raynger ST Series 0E F to 750E F [-18E C to 400E C]
Manufacturer: Raytek Inc.
Santa Cruz, Ca.
(800)227-8074
or accepted equal to the portable infrared thermometer

E. Flashlight 2-D cell

F. SSPC Visual Standard for Abrasive Blast Cleaned Steel SSPC-Vis 1-89

G. High low thermometer

6.0 HANDLING

All paint and thinner must be delivered to the fabricator in original, unopened containers with labels intact. Minor damage to containers is acceptable provided the container has not been punctured. Thinner containers must be a maximum of 5 gallons [19 L].

Paint must be stored at the temperature recommended by the manufacturer to prevent paint deterioration. The QCPS must record storage temperatures.

Each container of paint and thinner must be clearly marked or labeled to show paint identification, component, color, lot number, stock number, date of manufacture, and information and warnings as may be required by Federal and State laws. The QCPS must record the lot number, stock number and date of manufacture.

All containers of paint and thinner must remain unopened until used. The label information must be legible and checked at the time of use. Solvent used for cleaning equipment is exempt from the above requirements.

Paint which has livered, gelled or otherwise deteriorated during storage must not be used. However, thixotropic materials which can be stirred to attain normal consistency may be used. The oldest paint of each kind must be used first. No paint must be used which has surpassed its shelf life.

The Fabricator must provide thermometers capable of monitoring the maximum high and low temperatures within the storage facility. The Fabricator is responsible for properly disposing of all unused paint and paint containers.

The Fabricator must furnish TE-24 and the QCPS records for all materials used on the project to the QA Inspector.

7.0 MIXING AND THINNING

All ingredients in any container of paint must be thoroughly mixed immediately before use and the primer must be continuously mixed by an automated agitation system (hand held mixers not allowed). Paint must be carefully examined after mixing for uniformity and to verify that no unmixed pigment remains on the bottom of the container. The paint must be mixed with a high shear mixer (such as a Jiffy Mixer). Paddle mixers or paint shakers are not allowed. Paint must not be mixed or kept in suspension by means of an air stream bubbling under the paint surface.
The QCPS must record that all equipment is working correctly.

All paint must be strained after mixing. Strainers must be of a type to remove only skins and undesirable matter, but not pigment.

No thinner must be added to the paint without the QCPS’s approval, and only if necessary for proper application as recommended by the manufacturer. When the use of thinner is permissible, thinner must be added slowly to the paint during the mixing process. All thinning must be done under supervision of the QCPS. In no case must more thinner be added than that recommended by the manufacturer's printed instructions. Only thinners recommended and supplied by the paint manufacturer may be added to the paint. No other additives must be added to the paint.

Catalysts, curing agents, or hardeners which are in separate packages must be added to the base paint only after the base paint has been thoroughly mixed. The proper volume of catalyst must then be slowly poured into the required volume of base with constant agitation. Liquid which has separated from the pigment must not be poured off prior to mixing. The mixture must be used within the pot life specified by the manufacturer. Therefore only enough paint must be catalyzed for prompt use. Most mixed, catalyzed paints cannot be stored, and unused portions of these must be discarded at the end of each working day.

8.0 COATING APPLICATION

8.01 General

Galvanized or metalized surfaces must not be painted. All new structural steel must be painted. The following methods of application are permitted for use by this specification, as long as they are compatible with the paint being used: air-less or conventional spray. Brushes, daubers, small diameter rollers or sheepskins may be used for places of difficult access when no other method is practical.

Cleaning and painting must be so programmed that dust or other contaminants do not fall on wet, newly-painted surfaces. Surfaces not intended to be painted must be suitably protected from the effects of cleaning and painting operations. Over spray must be removed with a stiff bristle brush or wire screen without damaging the paint. No visible abrasives from adjacent work must be left on the IZEU system. Abrasives on the IZEU system coat must be removed.

8.02 Coating of Bolted Faying Surfaces

Surfaces indicated below must be treated according to Method A as described in this specification:

A. Faying surfaces of main beam or girder bolted field splices.
B. All internal contact surfaces of filler and splice plates.
C. Other surfaces indicated in the plans.
Bolted cross frames on straight beams or girders do not need to meet the requirements of Method A unless requested by the Contractor or specified by plan notes. If plan notes have not specified otherwise and the cross frames connections have all 3 coats applied in the shop, any damage to the paint system in the field due to the bolt tightening operations shall be repaired by the Contractor.

8.03 Method A

The faying surfaces of bolted splices must be coated with inorganic zinc primer in the shop. After erection is complete the final coatings of epoxy intermediate coat and urethane protective coats must be field applied as to overlap the shop coatings shown in Figure 1 with the field coats shown in Figure 2.

All shop bolted connections and shop bolted cross frames must be removed and disassembled prior to the blasting and coating of the girders or beams. The parts must be blasted separately and primed, then reassembled and the bolts fully tightened using the turn of the nut method. After bolting is complete the coatings of epoxy intermediate and urethane protective coat must be shop applied.
All galvanized nuts, bolts, and washers, must be solvent cleaned after installation. The epoxy coat and the urethane protective coat must then be applied.

Erection marks added by the fabricator to highlight or enhance the required steel stamped erection marks must be made without damaging the paint system. Erection marks must be applied only after the finish coat is cured and must be removed at the end of the project. Erection marks may be applied to the faying surfaces. These marks to the inorganic coating need not be removed but must be of a paint supplied by the IZEU system manufacture.

Unless otherwise specified, all coats must be applied by spray methods.

The Contractor for field application and the Fabricator for shop application, must supply the Engineer with the product data sheets before any coating is done. The product data sheets must indicate the mixing and thinning directions, the recommended spray nozzles and pressures and the minimum drying times for: testing thickness, re-coating and handling the shop applied coats.

These product data sheets must be followed except when they conflict with these specifications, in which case the specifications must govern.

If the surface is degraded or contaminated after surface preparation and before painting, the surface must be restored before painting application. In order to prevent degradation or contamination of cleaned surface, the prime coat of paint must be applied within 24 hours in the shop and 12 in the field after blast cleaning as required in surface preparation above.

Cleaning and painting must be scheduled so that dust or other contaminants do not fall on wet, newly painted surfaces. Surfaces not intended to be painted must be suitably protected from the effects of cleaning and painting operations. Over spray must be removed with a stiff bristle brush, wire screen, or a water wash with sufficient pressure to remove over spray without damaging the paint. The over spray must be removed before applying the next coat. All abrasives and residue, including visible abrasives from adjacent work, must be removed from painted surfaces before re-coating, with a vacuum system equipped with a brush type cleaning tool.

Spray application for the intermediate coat (epoxy) must not be used where traffic (including railroad, highway and river traffic, public and private property) is affected unless the operation is totally contained to prevent over spray.

### 8.04 Spray Application (General)

All spray application of paint must be in accordance with the following:

Spray equipment must be kept clean so dirt, dried paint and other foreign materials are not deposited in the paint film. Any solvent left in the equipment must be completely removed before using.
Paint must be applied in a uniform layer with overlapping at the edges of the spray pattern. The border of the spray pattern must be painted first; with the painting of the interior of the spray pattern to follow, before moving to the next spray pattern area. A spray pattern area is such that the gun must be held perpendicular to the surface and at a distance which will ensure that a wet layer of paint is deposited on the surface. The trigger of the gun should be released at the end of each stroke. The QCPS must record that each spray operator demonstrated to the QCPS the ability to apply the paint as specified. Any operator who does not demonstrate this ability must not spray.

The QCPS must document that all spray equipment used follows the paint manufacturer's equipment recommendations. Equipment must be suitable for use with the specified paint, to avoid paint application problems.

If air spray is used, traps or separators must be provided to remove oil and condensed water from the air. The traps or separators must be of adequate size and must be drained periodically during operations. The following test must be made by the Fabricator and verified by the QCPS to insure that the traps or separators are working properly.

The QCPS must perform and record that air is blown from the spray gun for 30 seconds onto a white cloth or blotter held in a rigid frame. If any oil, water or other contaminants are present on the cloth or blotter: painting must be suspended until the problem is corrected and the operation is verified by repeating this test. This test must be made at the start of each shift and at 4 hour intervals. This is not required for an airless sprayer.

8.05 Application Approval

The end of the application of each IZEU coat for each beam or girder must be subject to QCPS inspection and approval to detect any defects which might result from the fabricator's methods. If defects are discovered, the fabricator must make all necessary adjustments to the method of application to eliminate defects before proceeding with additional IZEU coating application.

8.06 Temperature

Paint must not be applied when the temperature of the air, steel, or paint is below $40\,\text{F} \left[4\,\text{C}\right]$. Paint must not be applied when the steel surface temperature is expected to drop below $40\,\text{F} \left[4\,\text{C}\right]$ degrees F before the paint has cured for the minimum times specified below:
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Primer</td>
<td>5 hrs.</td>
<td>4 hrs.</td>
<td>3 hrs.</td>
<td>2 hrs.</td>
</tr>
<tr>
<td>Intermediate</td>
<td>7 hrs.</td>
<td>6 hrs.</td>
<td>5 hrs.</td>
<td>4 hrs.</td>
</tr>
<tr>
<td>Finish</td>
<td>10 hrs.</td>
<td>8 hrs.</td>
<td>6 hrs.</td>
<td>4 hrs.</td>
</tr>
</tbody>
</table>

The above temperatures and times must be monitored with recording thermometers or a log kept with manual thermometer at 4 hour intervals.

8.07 Moisture

Paint must not be applied when the steel surface temperature is less than 5°F [3°C] above the dew point. Paint must not be applied to wet or damp surfaces or on frosted or ice-coated surfaces. Paint must not be applied when the relative humidity is greater than 85%. Paint must not be applied outdoors. The QCPS must record the relative humidity prior to painting, at every shift and 4 hour intervals.

8.08 Repair Procedures

Damaged areas, and areas which do not comply with the requirements of this specification, must be repaired in a manner to blend the patched area with the adjacent coating. The finished surface of the patched area must have a smooth, even profile with the adjacent surface.

The QCPS must submit his method of conducting repairs, correcting runs, sags, mud cracking and un-workman like conditions in writing to the Office of Structural Engineering.

8.09 Dry Film Thickness

Prime thickness, cumulative prime and intermediate thickness and cumulative prime, intermediate and finish thickness must be determined by use of type 2 magnetic gage in accordance with the following:

Five separate spot measurements must be made, spaced evenly over each 100 Square feet [9 square meters] of painted surface area. Three gage readings must be made for each spot measurement. The probe must be moved a distance of 1 to 3 inches [25 to 75 mm] for each new gage reading. Any unusually high or low gage reading that cannot be repeated consistently must be discarded. The average (mean) of the 3 gage readings must be used as the spot measurement. The average of five spot measurements for each such 100 square feet [9 square meters] area must not be less than the specified thickness. No single spot measurement in any 100 square feet [9 square meter] area must be less than 80% of the specified minimum thickness nor greater than 120% of the maximum specified thickness. Any one of 3 readings which are averaged to produce each spot measurement, may under-run or over-run by a greater amount. The 5 spot
measurements must be made for each 100 square feet [9 square meters] of area.

Each coat of paint must have the following mil thickness measured above the peaks:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Prime</td>
<td>3.0 [75]</td>
<td>5.0 [125]</td>
<td>2.4 [60]</td>
<td>6.0 [150]</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5.0 [125]</td>
<td>7.0 [175]</td>
<td>4.0 [100]</td>
<td>8.4 [210]</td>
</tr>
<tr>
<td>Sub Total</td>
<td>8.0 [200]</td>
<td>12.0 [300]</td>
<td>6.4 [160]</td>
<td>14.4 [360]</td>
</tr>
<tr>
<td>Finish</td>
<td>2.0 [50]</td>
<td>4.0 [100]</td>
<td>1.6 [40]</td>
<td>4.8 [120]</td>
</tr>
<tr>
<td>Total</td>
<td>10.0 [250]</td>
<td>16.0 [400]</td>
<td>8.0 [200]</td>
<td>19.2 [480]</td>
</tr>
</tbody>
</table>

For any spot or maximum average thickness over 19.2 mils [480 Fm], it will be necessary for the Contractor to prove to the Department the excess thickness will not be detrimental to the coating system. This must be accomplished by providing the Office of Structural Engineering, for approval, certified test data proving the excessive thickness will adequately bond to the steel when subjected to thermal expansion and contraction. After the thermal expansion and contraction cycles have taken place, the tested system must be subjected to pull off tests and the results compared to the results of pull off tests which have been performed on a paint system with the proper thicknesses. In addition to the certified test results, it will also be necessary for the Contractor to provide the Office of Structural Engineering a written statement from the paint manufacturer stating that this excessive thickness is not detrimental.

If the Office of Structural Engineering does not approve the excessive coating thicknesses or the Contractor elects not to provide the required written statement from the plant manufacturer and the certified test results when required, the Contractor, at his own expense, must remove and replace the coating. The removal and replacement of the coating must be done as specified in the section of this specification titled “Repair Procedures”.

The QCPS must provide a cover letter and specified check point data documenting QCPS acceptance that shop painting has been performed per specification.

8.10 Continuity

Each coat of paint must be applied as a continuous film of uniform thickness free of all defects such as holidays, runs, sags, etc. All thin spots or areas missed must be repainted and permitted to dry before the next coat of paint is applied.
9.0 HANDLING AND SHIPPING

Extreme care must be exercised in handling the steel in the shop, during shipping, during erection, and during subsequent construction of the bridge. Painted steel must not be moved or handled until sufficient cure time has elapsed and approval has been obtained from the inspector. The steel must be insulated from the binding chains by softeners. Hooks and slings used to hoist steel must be padded. Diaphragms and similar pieces must be spaced in such a way that no rubbing will occur during shipment that may damage the coatings. The steel must be stored on pallets at the job site, or by other means, so that it does not rest on the ground or so that components do not fall or rest on each other. All shipping and job site storage details must be presented to the Engineer prior to fabrication in writing and be approved prior to shipping the steel. Approval of the above does not relieve the Contractor of responsibility of shipping or storage damage.

10.0 SAFETY REQUIREMENTS AND PRECAUTIONS

The Contractor must meet the safety requirements of the Ohio Industrial Commission and the Occupational Safety and Health Administration (OSHA), in addition to the scaffolding requirements below.

The Contractor is required to meet the applicable safety requirements of the Ohio Industrial Commission in addition to the scaffolding requirements specified below.

The Material Safety Data Sheets (MSDS) must be provided at the preconstruction meeting for all paint, thinners and abrasives used on this project. No work must start until the MSDS has been submitted.

The fabricator must also provide MSDS for all abrasives to be used on this project to the shop inspector. No work must start until MSDS have been submitted.

11.0 SCAFFOLDING

Rubber rollers, or other protective devices meeting the approval of the Engineer, must be used on scaffold fastenings. Metal rollers or clamps and other types of fastenings which will mar or damage coated surfaces must not be used.

12.0 INSPECTION ACCESS FOR FIELD TOUCH UP

In addition to the requirement of 105.11, the Contractor must furnish, erect, and move scaffolding and other appropriate equipment, to permit the Inspector the opportunity to inspect closely observe, all affected surfaces. This opportunity must be provided to the Inspector during all phases of the work and continue for a period of at least ten (10) working days after the touch-up work has been completed. When scaffolding is used, it must be provided in accordance with the following requirements. When scaffolding, or the hangers attached to the scaffolding are supported by horizontal wire ropes, or when scaffolding is placed
directly under the surface to be painted, the following requirements must be complied with:

When scaffolding is suspended 43" [1100 mm] or more below the surface to be painted, two rows of guardrail must be placed on all sides of the scaffolding. One row of guardrail must be placed at 42" [1050 mm] above the scaffolding and the other row at 20" [500 mm] above the scaffolding.

When the scaffolding is suspended at least 21" [530 mm], but less than 43" [1100 mm] below the surface to be painted, a row of guardrail must be placed on all sides of the scaffolding at 20" [500 mm] above the scaffolding.

Two rows of guardrail must be placed on all sides of scaffolding not previously mentioned. The rows of guardrail must be placed at 42" [1050 mm] and 20" [500 mm] above scaffolding, as previously mentioned.

All scaffolding must be at least 24" [610 mm] wide when guardrail is used and 28" [710 mm] wide when the scaffolding is suspended less than 21" [530 mm] below the surface to be painted and guardrail is not used. If two or more scaffolding are laid parallel to achieve the proper width, they must be rigidly attached to each other to preclude any differential movement.

All guardrail must be constructed as a substantial barrier which is securely fastened in place and is free from protruding objects such as nails, screws and bolts. There must be an opening in the guardrail, properly located, to allow the Inspector access onto the scaffolding.

The rails and uprights must be either metal or wood. If pipe railing is used, the railing must have a nominal diameter of no less than one and one half inches. If structural steel railing is used, the rails must be 2 X 2 X 3/8 inch [50 x 50 x 10 mm] steel angles or other metal shapes of equal or greater strength. If wood railing is used, the railing must be 2 X 4 inch [50 x 100 mm] (nominal) stock. All uprights must be spaced no more than 8 feet [2.4 m] on center. If wood uprights are used, the uprights must be 2 X 4 inches [50 x 100 mm] (nominal) stock.

When the surface to be inspected is more than 15 feet [4.6 m] above the ground or water, and the scaffolding is supported from the structure being painted, the Contractor must provide the Inspector with a safety belt and lifeline. The lifeline must not allow a fall greater than 6 feet [2 m]. The Contractor must provide a method of attaching the lifeline to the structure independent of the scaffolding, cables, or brackets supporting the scaffolding.

When scaffolding is more than two and one half feet [0.75 m] above the ground, the Contractor must provide a ladder for access onto the scaffolding. The ladder and any equipment used to attach the ladder to the structure must be capable of supporting 250 pounds [115 kg] with a safety factor of at least four (4). All rungs, steps, cleats, or treads must have uniform spacing and must not exceed 12" [305 mm] on center. At least one side rail must extend at least 36" [915 mm] above the landing near the top of the ladder.

An additional landing must be required when the distance from the ladder to the point where the
scaffolding may be accessed, exceeds 12" [305 mm]. The landing must be a minimum of at least 24" [610 mm] wide and 24" [610 mm] long. It must also be of adequate size and shape so that the distance from the landing to the point where the scaffolding is accessed does not exceed 12" [305 mm]. The landing must be rigid and firmly attached to the ladder; however, it must not be supported by the ladder. The scaffolding must be capable of supporting a minimum of 1000 lbs [455 kg].

In addition to the aforementioned requirements, the Contractor is still responsible to observe and comply with all Federal, State and local laws, ordinances, regulations, orders and decrees.

The Contractor must furnish all necessary traffic control to permit inspection during and after all phases of the project.

13.0 PROTECTION OF PERSONS AND PROPERTY

The Contractor must collect, remove and dispose of all buckets, rags or other discarded materials and must leave the job site in a clean condition.

The Contractor must protect all portions of the structure which are not to be painted, against damage or disfigurement by splashes, spatters, and smirches of paint.

The Contractor must install and maintain suitable shields or enclosures to prevent damage to adjacent buildings, parked cars, trucks, boats, or vehicles traveling on, over, or under structures being painted. They must be suitably anchored and reinforced to prevent interfering with normal traffic operations in the open lanes. Payment for the shields must be included as incidental to the applicable field coating operation. Work must be suspended when damage to adjacent buildings, motor vehicles, boats, or other property is occurring.

When or where any direct or indirect damage or injury is done to public or private property, the Contractor must restore, at his own expense, such property, to a condition similar or equal to that existing before such damage or injury was done.

14.0 POLLUTION CONTROL

The Contractor must take all necessary precautions to comply with pollution control laws, rules or regulations of Federal, State or local agencies.

15.0 WORK LIMITATIONS

Abrasive blasting and painting done in the field must be performed between April 15 and October 15. Even though the Contractor is permitted to work prior to May 1, April is considered a winter month and no extension due to adverse weather conditions will be granted for this period. Additional work limitations on specific bridges/projects may be required by plan note.
16.0 METHOD OF PAYMENT

Payment will be made at contract prices for:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNIT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>863</td>
<td>Lump Sum</td>
<td>STRUCTURAL STEEL MEMBERS, LEVEL ?, AS PER PLAN</td>
</tr>
</tbody>
</table>
AN-6 STEEL POT BEARINGS

This un-numbered plan note is to be supplied when pot bearings are being specified for a specific bridge. The note may require some revisions for the specific project. Generally this note will be used as an un-numbered proposal note unless the owner requires a plan note be furnished.

ITEM SPECIAL - STEEL POT BEARINGS

1.0 DESCRIPTION

1.1 This item shall consist of furnishing all materials, services, labor, tools, equipment and incidentals necessary to design, fabricate, test and install pot bearings in accordance with the plans and this specification. Unless modified by this specification, the requirements of Supplemental Specification 863 shall apply. The fabricator shall be pre-qualified for Miscellaneous Level Fabrication.

1.2 The pot bearing shall consist of the following parts:

A. Rectangular Sole Plate - Top side beveled to the slope of the girder and field welded to the girder flange. Bottom side level and faced with stainless steel for expansion bearings.

B. Circular piston - Top faced with PTFE for expansion bearings.

C. Sole Plate - Top side beveled to the slope of the girder and welded to the girder flange. Sole plate and piston can be machined from one piece of steel or made up of pieces connected by welding.

D. Elastomeric Disc - Confined within pot for the purpose of providing rotation and support for the piston. Silicone grease shall be provided above and below the elastomeric disc. The disc is sealed with brass sealing rings.

E. Sealing Rings - Seal between pot and piston used to contain the elastomeric disc.

F. Guide Bar/Bars (for Guided Bearings) - Attached to or integral with sole plate for purpose of guiding expansion bearings and transmitting lateral loads to the pot. Bearings may be edge or center guided.

G. Pot - Containment for the elastomeric disc and transmission of vertical and lateral loads to masonry plates. Shop welded to masonry plates.

H. Masonry Plate - Distribute vertical and horizontal forces from the steel pot to the concrete bridge seat. Masonry plate sits on a bearing pad and is connected to the concrete with anchor bolts.
I. Horizontal bridge movement shall be accommodated through the provision of mated sliding surfaces consisting of stainless steel and PTFE. See the applicable sections of this specification for specific requirements.

1.3 Bearing Height

If necessary, the Contractor shall adjust beam seat elevations to account for differences in the bearing height detailed in the plans versus the bearing height provided.

As an alternative, the total bearing height shown in the plans can be met by increasing the sole plate thickness, pot base thickness, masonry plate thickness, piston thickness or a combination thereof.

2.0 DESIGN AND MATERIALS REQUIREMENTS

2.1 The design criteria and materials requirements shall be governed by these provisions and all applicable sections of AASHTO Standard Specifications for Highway Bridges, including the current interim specifications, Division I, Section 14 and Division II, Section 18. Bearings shall be designed to accommodate the loads, forces and movements specified herein or in the bearing schedule which is located in the plans.

2.2 Sole Plate

A. ASTM A588 or A572 Grade 50 Steel [ASTM A588M or A572M, Grade 345]. Top side beveled to the slope of the girder under full un-factored permanent load. Bottom side level.

B. Rectangular or square in plan.

C. Minimum thickness shall be 0.75 inches [19 mm].

2.3 Piston

A. ASTM A572 Grade 50 or A588 Steel [ASTM A588M or A572M, Grade 345].

B. Diameter of piston shall be 0.03 inch [0.76 mm] less than the inside diameter of the pot.

C. Piston thickness shall be sufficient to provide a clearance between the top of the pot wall and surface above the pot wall when the piston is in a fully rotated position.

1. Pots with square exterior:
   \[ h_{p2} = (0.7 \times \theta_u \times S) + 0.125 \text{ (inch)} \]
   \[ h_{p2} = (0.7 \times \theta_u \times S) + 3.175 \text{ (mm)} \]
2. Pots with circular exterior:
\[ h_{p2} = (R_o \times \theta_u) + 0.125 \text{ (inch)} \]
\[ h_{p2} = (R_o \times \theta_u) + 3.175 \text{ (mm)} \]

\[ \theta_u = \text{maximum rotation; unless otherwise shown in the plans,} \]
\[ \theta_u \text{ shall be taken as } 2 \theta_s = 2(0.02) = 0.04 \text{ radians} \]

D. Piston walls shall be tapered inward, toward the top, to prevent binding against the pot walls during rotation. A piston rim of minimum height “w” shall be provided.

\[ w \geq \frac{2.5H_s}{D_pF_y} \geq 0.125'' \]

\[ w \geq \frac{2500H_s}{D_pF_y} \geq 3.175\text{mm} \]

Where:
\[ H_s = \text{service lateral load} = 0.1(1.0 \times DLR + 1.3 \times LLR) \text{kips/}[kN] \]

Or
\[ H_s = \text{service lateral load} = 0.2(1.0 \times DLR) \text{kips/}[kN] \]

(whichever is greater)

DLR = Dead Load Reaction
LLR = Live Load Reaction
\[ D_p = \text{internal diameter of pot (inches or mm)} \]
\[ F_y = \text{yield of steel (ksi or MPa)} \]

E. The piston shall be machined from a single piece of structural steel.

2.4 PTFE

A. PTFE fabric fibers shall conform to the following:

1. The resin from which the fibers are produced shall be 100 percent PTFE
conforming to ASTM D4894.

2. Tensile Strength - ASTM D2256 - 24,000 PSI [165 MPa] (minimum).

3. Elongation - ASTM D2256 - 75 percent (Minimum).

4. The TFE fabric shall have a minimum thickness of 0.0625 inch [1.6 mm](compressed). Maximum thickness shall be 0.125 inch [3.2 mm](compressed).

B. Finished unfilled PTFE sheet shall be made from 100 percent virgin PTFE resin and shall conform to the following requirements:

1. Tensile strength D4894 - 2800 PSI [ 19.3 MPa] (minimum).

2. Elongation ASTM D4894 - 200 percent (minimum).


4. Melting Point - ASTM D4894 - 623°F +/- 2 [328°C +/- 1]

5. Minimum thickness shall be 0.125 inch [3.2 mm] when the maximum PTFE dimension is less than or equal to 24 inches [600 mm] and 0.1875 inch [4.8 mm] when the maximum dimension of the PTFE is greater than 24 inches [600 mm]. Sheet shall be recessed and epoxy bonded into a steel substrate. The shoulders of the recess shall be sharp and square and the depth shall be equal to one-half of the PTFE sheet thickness.

6. PTFE sheet shall be commercially etched on its bonding side.

C. Maximum contact stresses shall conform to the following:

<table>
<thead>
<tr>
<th>Material</th>
<th>Average Contact Stress</th>
<th>Edge Contact Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Permanent Loads</td>
<td>All Loads</td>
</tr>
<tr>
<td>Woven PTFE Fiber over a Metallic Substrate</td>
<td>4.0 ksi [27.5 MPa]</td>
<td>6.0 ksi [41.0 MPa]</td>
</tr>
<tr>
<td>Confined Unfilled Sheet PTFE</td>
<td>4.0 ksi [27.5 MPa]</td>
<td>6.0 ksi [41.0 MPa]</td>
</tr>
</tbody>
</table>

D. The mating surface to PTFE, shall be large enough to cover the PTFE during all conditions of expansion or contraction that the bridge will undergo plus an additional
two (2) inches [50 mm].

2.5 Stainless Steel

A. Stainless steel sheet surface shall conform to ASTM A167 or A240 Type 304. The minimum thickness shall be at least 16 gauge when the maximum dimension of the surface is less than 12 inches [300 mm] and 13 gauge when the maximum dimension of the surface is greater than 12 inches [300 mm]. Stainless steel in contact with PTFE shall have a #8 mirror finish or better. Material and finish shall be such that the requirements of 4.2 (2) are met.

B. When stainless steel is used as a mating surface to PTFE, the stainless steel surface shall be large enough to cover the PTFE during all conditions of expansion or contraction that the bridge will undergo plus an additional two (2) inches [50 mm].

2.6 Elastomeric Disc

A. The elastomeric disc shall meet the following average compressive stress requirements: Maximum of 3,500 psi [24 MPa] when the bearing vertical service load design capacity specified in the plan is applied to the area of the disc.

B. Minimum disc thickness: $h \geq 3.33D_p\theta_u$ (inches or mm)

\[ D_p = \text{inside diameter of pot (inches or mm)} \]
\[ \theta_u = \text{maximum rotation; unless otherwise shown in the plans,} \]
\[ \theta_u \text{ shall be taken as } 2 \theta_s = 2(0.02) = 0.04 \text{ radians} \]

C. The elastomeric disc shall consist of 100 percent virgin polychloroprene (Neoprene) meeting the requirements of CMS Item 711.23 or 100 percent virgin natural polyisoprene (natural rubber) meeting the requirements of the current AASHTO M251.

D. Hardness shall be 50 durometer +/- 10.

E. The disc shall consist of one solid piece of elastomer.

F. The elastomeric disc shall be lubricated with silicone grease meeting the requirements of MIL-S-8660C.

G. The upper edge of the elastomeric disc shall be recessed to receive the sealing rings so that they sit flush with the upper surface of the disc.

2.7 Sealing Ring

A. Rings shall be flat and shall be made of brass conforming to the requirements of
ASTM B36, half hard.

B. Minimum width shall be the larger of 0.25 inch [6 mm] or 0.02 \( D_p \) but not to exceed 0.50 inch [12.5 mm].

C. Minimum thickness shall be 0.2 times the width but shall not exceed 0.08 inch [2 mm].

D. The rings shall have a smooth finish of 64 micro-inch (RMS) or less.

E. Three rings are required.

F. The rings shall be split and snugly fit the recess in the elastomeric disc as well as the inside diameter of the pot. No where shall the gap between the ring and the wall exceed 0.01\"[0.25 mm]. The rings can be made oversize and sprung into position, however, no dishing or distortion will be permitted. The ends of the rings at the split shall be cut at 45 degrees to the vertical. The maximum gap shall be 0.050 inch [1.250 mm] when installed. The rings shall be arranged so that the splits on each of the three rings are equally spread around the circumference of the pot.

2.8 Guide Bars

A. ASTM A36 [A 36M], or A572 Grade 50 [ASTM A572M Grade 345] or A588 [A588M] faced with stainless steel.

B. Guide bars may be integral by machining from a solid sole plate or they may be attached to the sole plate by press fit into recess and welding the ends. The side surfaces of the guide bars shall be faced with stainless steel, see section 2.5. Welding of guide bars to the sole plate shall be performed prior to welding of stainless steel to the sole plate or guide bars.

C. The total space (both sides) between the guide bars and guided members shall be a maximum of 0.125 inch [3.2 mm].

D. The guide bars shall be designed for \( H_S \).

\[
H_S = \text{service lateral load} = 0.1(1.0 \times \text{DLR} + 1.3 \times \text{LLR})(\text{kips or kN})
\]

Or

\[
H_S = \text{service lateral load} = 0.2(1.0 \times \text{DLR} ) \ (\text{kips or kN})
\]

(whichever is greater)

E. Guiding arrangements shall be designed so that the stationary portion of the bearing is always within the guides at all points of translation and rotation of the bearing.

F. On guided bearings, guide bars shall be designed so that the clearances between
rotating and non-rotating parts are maintained in such a manner that binding of the bearing is prevented.

2.9 Pot

A. A572 Grade 50 [A 572M Grade 345] or A588 steel.

B. The pot shall consist of a solid plate into which a circular recess has been machined.

C. In order to ensure that the piston seal and rim remain in full contact with the pot wall, the depth of the pot cavity shall be equal to or greater than the following:

\[ h_{p1} \geq 0.5D_p \theta_u + h_r + h_w \]

\( \theta_u \) = maximum rotation; unless otherwise shown in the plans, \( \theta_u \) shall be taken as \( 2 \theta_s = 2(0.02) = 0.04 \) radians

\( h_r \) = thickness of elastomeric pad (inches or mm)

\( h_w \) = height from top of rim to underside of piston (inches or mm)

D. The pot inside diameter shall be the same as the elastomeric disc.

E. The thickness of the pot wall shall be sufficient to transmit a lateral horizontal force to the pot base without causing deflection/distortion to the pot wall or base and shall satisfy the following equation:

\[
0.75" \leq t_w \geq \sqrt{\frac{40H_s \theta_s}{F_y}}
\]

\[
19\text{mm} \leq t_w \geq \sqrt{\frac{4000H_s \theta_s}{F_y}}
\]

Where:

\( H_s \) = service lateral load = \( 0.1(1.0 \times \text{DLR} + 1.3 \times \text{LLR}) \) (kips or kN)

Or

\( H_s \) = service lateral load = \( 0.2(1.0 \times \text{DLR}) \) (kips or kN)

(whichever is greater)
\( \theta_s = \) maximum service rotation = 0.02 radians unless shown otherwise in the plans
\( F_Y = \) yield strength of steel (ksi or MPa)

F. The minimum thickness of the pot beneath the elastomer for a bearing directly on a
masonry plate shall be the greater of 0.045 x pot I.D. or 0.50 inch [13 mm]. The
minimum thickness of the pot beneath the elastomer for a bearing directly on concrete
shall satisfy the greater of 0.06 x pot I.D. or 0.75 inch [19 mm] whichever is the
greater. The design shall be sufficient to prevent distortion of the pot wall or base.

G. The inside surfaces of pot walls and base shall be machined to a fine surface finish of
64 microinches or better. No metalizing, galvanizing or paint shall be applied to these
surfaces.

2.10 Masonry Plate

ASTM A572 Grade 50 [A572M Grade 345] or A588 [A588M] steel.

2.11 Bearing pad shall be sheet lead shall conforming to CMS 711.19 or preformed pads
conforming to CMS 711.21.

3.0 FABRICATION

3.1 Attachment of Sheet PTFE to substrate.

A. PTFE sheet shall be recessed into and bonded to a steel substrate.

B. PTFE shall be recessed for one half its thickness.

C. The bonding surface of the steel shall be cleaned of rust, scale, oil and grease by blast
cleaning and then wiped clean with a cleaning solvent. Blast cleaning shall be
performed within a maximum of four hours prior to bonding.

D. The adhesive material, and the bonding procedures to be used, and surface
preparation shall conform to the requirements of federal specification MMM-A-134
and the manufacturer’s recommendations. Adhesion strength shall be such that the
requirements of Section 4.4 of this specification are satisfied.

E. After completion of the bonding operation, the PTFE surface shall be smooth and free
from bubbles.

3.2 Attachment of PTFE Fabric to Substrate

A. PTFE Fabric shall be mechanically interlocked and bonded to the steel substrate.

B. The bonding surface of the steel shall be cleaned of rust, scale, oil and grease by blast
cleaning and then cleaned with solvent. Blast cleaning shall be performed within a maximum of four hours prior to bonding.

C. The mechanical interlock, adhesive bonding material and procedures, and surface preparation shall conform to the requirements of federal specification MMM-A-134 and the manufacturer’s recommendations. Adhesion strength shall be such that the requirements of Section 4.4 of this specification are satisfied.

D. Migration of epoxy through the fabric will not be permitted.

E. Fabric shall be furnished in one piece. Edges shall be oversewn or recessed so that no cut fabric edges are exposed.

3.3 Attachment of Sheet Stainless Steel

Stainless steel shall be attached to its steel substrate by a continuous seal weld around its entire perimeter. Welds shall conform to the AWS requirements for stainless steel. The welder shall be pre-qualified by test welds prepared, welded and tested in accordance with 6.7 of ANSI/AWS D1.3, Structural Welding Code - Sheet Steel. After welding, the stainless steel sheet shall be flat, free from wrinkles and in continuous contact with its backing plate. After welding the entire stainless steel surface shall conform to the requirements of Section 2.5 of this specification after welding. No roughness from the weld protruding above the surface of the stainless steel will be permitted.

3.4 Corrosion Protection

All steel surfaces (including A588 Steel) exposed to the atmosphere, except stainless steel surfaces and the insides of the pot, shall be metalized. The thickness of the coating shall be 12 to 14 mils [300 to 350 µm]. The wire used for the metalizing shall consist of 100% zinc. Surface preparation and application shall conform to SSPC Coating System Guide 23.00 “Guide for Thermal Spray Metallic Coating Systems”.

3.5 Welding

Welding as a means of attachment shall be done in a controlled manner and shall conform to Supplemental Specification 863. Welding to a steel plate which has bonded TFE surface may be permitted providing welding procedures are established which restrict the maximum temperature reached by the bond area to less than 300°F [150°C], as determined by temperature, indicating pencils, or other suitable means.

3.6 Tolerances

All bearings shall be checked for tolerances.

General Flatness Criteria:
A. Flatness tolerances shall be defined as:

1. Class A Tolerance = 0.0005 x nominal dimension.
2. Class B Tolerance = 0.001 x nominal dimension.
3. Class C Tolerance = 0.002 x nominal dimension.
4. Nominal dimension shall be defined as the actual dimension of the plate, in inches [mm], spanned by the straightedge.

B. Flatness shall be determined by placing a straightedge, longer than the nominal dimension to be measured, in contact with the surface to be measured or as parallel to it as possible. Select a feeler gauge having a tolerance of + or - 0.001 inch [25 µm] and attempt to insert it under the straightedge (the smallest number of blades shall be used). Flatness is acceptable if the feeler does not pass under the straightedge. The straightedge may be located at any position on the surface and not necessarily at 90 degrees to the edges.

C. Tolerances - Sole Plate

1. Plan dimensions ± 0.125"[± 3 mm].
2. Thickness: ±0.0625" [± 1.5 mm]
3. Flatness of surface in contact with beam or girder - Class B.
5. Bevel slope (radians): ±0.002

D. Tolerances - Piston

1. Rim diameter: ±0.003" [± 75 µm]
2. Diameters less than 20" [500 mm] : +0.005"[125 µm]
3. For expansion bearings where upper side is faced with PTFE, flatness of upper side shall be Class A.
4. Flatness of lower side: Class B
5. Finish of side in contact with elastomeric disc shall be 63 micro-inches or less
6. Finish on piston rim shall be 32 micro-inches or less

E. Tolerances - Sole Plate/Piston

All applicable provisions of C and D

F. Tolerances - Elastomeric Disc (unstressed)

1. Diameter: -0.0", +0.125" [3 mm]
2. Thickness: -0.0, +0.125" [3 mm]

G. Tolerances - Guide Bar
1. Contact surface: -0.0", +0.125" [3 mm]
2. Flatness of backing surface for stainless steel: Class A
3. Inside of bar to inside of bar: -0.0", +0.030" [750 µm]
4. Parallelism of guides (radians): ±0.005

H. Tolerances - Pot

1. Inside diameter: ±0.003" [± 75 µm]
2. Pot underside shall be machined parallel to the inside and to a Class A flatness tolerance.
3. Wall thickness: -0.0", +0.125 [3 mm]

I. Tolerances - PTFE Substrates

Substrate Flatness: Class A

J. Tolerance of steel (not stainless) in contact with steel (not stainless): Class B

K. The edges of all parts shall be broken by grinding so there are no sharp edges.

L. Tolerances - overall heights of bearing: -0.0625, +0.125" [-1.5 mm, +3 mm]

M. Tolerances - Masonry Plate

1. Plan Dimensions: -0", +1/8" [3 mm]
2. Thickness: -1/32", +1/8" [-1 mm, +3 mm]
3. Flatness - Class C for the underside, Class B for the upper side.

4.0 TESTING

4.1 General

Tests shall be performed by the manufacturer or by an independent testing laboratory. The testing agent chosen by the Contractor will be subject to approval by the Director. Approval will be based on 1) the ability of the testing facility to perform the required test - possession of proper testing equipment and trained personnel, and 2) submittal of a report describing the testing procedures to be used including setup of testing apparatus, steps to be followed in the testing apparatus, steps to be followed in the testing procedures, readings, conversion of readings to final data, and sample calculations showing how the final results are obtained from the raw data.

4.2 Sampling

One guided expansion bearing and one fixed bearing shall be chosen, selected at random from each applicable lot of completed bearings.
A. One lot shall consist of no more than 25 bearings of one load category.

B. One load category shall consist of bearings having vertical load capacity within a range of no more than 200 kips [900 kN].

4.3 Friction test shall be performed on expansion bearing samples chosen as described in Section 4.2 above.

A. The test shall be conducted at the maximum service load stress (100% of dead load plus live load) applied to the PTFE area for the bearing with the load being held constant for one hour prior to and throughout the duration of the sliding test.

B. At least 100 cycles of sliding, each consisting of at least ±1 inch [± 25 mm] of movement, shall then be applied at a temperature of 68°F ± 10°F [20°C ± 5°C]. The breakaway friction coefficient shall be computed for each direction of cycle. The initial static breakaway friction coefficient for the first cycle shall not exceed twice the design coefficient of friction and the maximum value for all subsequent cycles shall not exceed the design coefficient of friction.

Following the 100 cycles of testing, the breakaway coefficient of friction shall be determined again and shall not exceed the initial value.

C. Unless shown otherwise in the plans, the design coefficient of friction shall be 0.03.

4.4 Proof load test shall be performed on bearing samples chosen as described in Section 4.2 above. The expansion bearing may be the ones used for the friction test described in 4.2 above.

A test bearing shall be loaded to 150 percent of the bearing's service load design capacity and simultaneously subjected to a rotational range of 0.02 radians (1.146°) or design rotation, whichever is greater, for a period of one (1) hour. The bearing will be visually examined both during the test and upon disassembly after the test. Any resultant visual defects, such as extruded or deformed elastomer, polyether urethane or TFE, damaged seals, or cracked steel, or interference between rotating and stationary parts shall be cause for rejection of the lot.

During the test, for pot bearings the steel bearing plate and steel piston shall maintain continuous and uniform contact for the duration of the test. Any observed lift-off will be cause for rejection of the lot. Bearings not damaged during testing may be used in the work.

4.5 Adhesion between the PTFE and substrate shall be tested on expansion bearing samples, chosen in accordance with section 4.2 and tested in accordance with ASTM D429,
Method B. The minimum peel strength shall be 25 lbs. per inch \([4.4 \text{ N per mm}]\). This test is in addition to adhesion determined under 4.2 and 4.3 above.

4.6 Test results shall be presented in a quality control report showing raw test data, reduced test data, sample calculations, measured tolerances and final results along with photographs and conclusions. Included shall be a statement of compliance.

4.7 Certified test data for all stainless steel, A36 [A36M] steel A572 [A572M] steel, A588 [A588M] steel, metalizing wire and PTFE shall be furnished to the Director showing compliance with the requirements of this specification.

5.0 SHIPPING AND PACKING

5.1 Bearings shall be securely banded together as units so that they may be shipped to the job site and stored without relative movement of the bearing parts or disassembly at any time. Bearings shall be wrapped in moisture proof and dust proof material to protect against shipping and job site conditions.

5.2 Care shall be taken to ensure that bearings at the job site are stored in a dry, sheltered area free from dirt or dust until installation.

5.3 Centerlines shall be marked on appropriate bearing parts for checking alignment in the field and be shown on shop drawings.

5.4 Each bearing, masonry plate and pad shall have a mark number and the mark number and placement location shall be shown on the shop drawings.

6.0 INSTALLATION

6.1 A representative from the bearing manufacturer shall be present on site for a sufficient period of time to ensure that the Contractor is installing the bearings properly.

6.2 Field welding of bearing to masonry plate or sole plate to beam/girder flange shall meet the requirement of Section 3.5 of this specification.

6.3 Bearings shall be evenly supported over their upper and lower surfaces under all erection and service conditions. Bearings shall not be dis-assembled for erection purposes.

6.4 Align the centerlines of the bearing assembly with those of the substructure and superstructure. On guided bearings align the bearings, taking into consideration the ambient temperature (to allow for the design expansion or contraction of the structure). Upper and lower bearing parts shall be offset to compensate for ambient temperature.

6.5 Bearing straps or retaining clamps shall be left in place as long as possible to ensure parts of bearings are not inadvertently displaced relative to each other.
6.6 Concrete bearing seats shall be prepared at the correct elevation and shall be level within 1:200.

6.7 Field repair metalized coating in accordance with ASTM 780, type A1 or A3.

6.8 If the bearing has been unsealed for any reason, at the discretion of the engineer, it may be required to ship the bearing back to the manufacturer for resealing.

7.0 METHOD OF MEASUREMENT

The quantity shall be the actual number of pot bearings furnished within the categories listed below. A complete and acceptable bearing system furnished and installed including bearing, masonry plate, bearing pad and anchor bolts will be measured on an each basis. No distinction will be made between fixed or expansion bearings. The category of each bearing is determined by the maximum vertical reaction listed in the construction drawings.

8.0 BASIS OF PAYMENT

8.1 Payment for pot bearings will be made at the contract unit price per each listed under:

    Item Special, Each, Steel Pot Bearings

8.2 No separate payment will be made for the work listed under testing and acceptance of this specification. This work shall be included in the unit price bid for the bearings.
AN-7 METALLIC COATING SYSTEM FOR SHOP APPLICATION

This un-numbered note establishes specification requirements for the application of a metalized coating in the fabrication shop for new structural steel bridges. To use this note the designer must have designed the steel bridge’s high strength bolted connections for a slip resistance based on the coefficient for galvanized coatings as defined in AASHTO. Additionally all other connections, cross frames, end frame connections to the main steel members shall also be bolted connections. Bolted connections are required to eliminate the need for field welding therefore eliminating possible damage to the metalized coating and the need for a great deal of field touch up and repair. Shear studs shall be field welded. Bid items shall be modified to use this note.

As example:

Item 863 Lump Sum STRUCTURAL STEEL MEMBERS, LEVEL ?, AS PER PLAN
The note AN-6 follows as the AS PER PLAN specifications to have a shop applied metallized coating on the structural steel members.

1.0 DESCRIPTION

In addition to the requirements of Supplemental Specification 863, this item shall consist of furnishing all necessary labor, materials and equipment to clean, apply a 100% zinc metalized coating and seal the metalized coating on all structural steel surfaces, as specified herein. The metallic coating system shall be applied in a structural steel fabrication shop qualified per Supplemental Specification 863 (SS 863) and having permanent buildings as per Supplemental Specification 863.07. Section 863.29 and 863.30 shall not apply.

The Contractor may select an independent metalizer with approval by the Director. The Contractor shall request such approval in writing. The Director's approval is based on a facility inspection verifying a metalizing facility with permanent buildings of adequate size with equipment, heating, lighting and experienced personnel to satisfactorily perform all specified operations. The fabricator’s QCPS (Quality Control Paint Specialist) required under SS863, is responsible for all specified quality control requirements at the independent metalizing facility.

This item shall also include galvanizing, per 711.02, of all nuts, washers, bolts and anchor bolts.

2.0 PRE-FABRICATION MEETING

In addition to the pre-fabrication meeting requirements under SS 863.081, both the fabricator’s Quality Control Specialist, (QCPS) and metalizing applicator shall present and discuss methods of operation, including repairs, to accomplish all phases of the preparation and coating work required by this specification. Attendance at this meeting and initial acceptance of methods discussed does not eliminate the applicator qualification requirements listed in this specification.
3.0 APPLICATION AND SPECIFICATION CHANGES

The metalized coating system shall conform to the requirements of this specification. If the fabricator and/or applicator propose variations from the specification, such as blast profile, blast medium, time between operations and thickness of metalizing passes, those variations shall be submitted before shop drawings, per SS 863.08, are approved. Approval of the variations will be based on comparison of qualification tests plates prepared per this specification and plates prepared with the proposed variations to the specification. The test plates representing the proposed specification variations shall equal or exceed the adhesion and bend test values of the test plates representing the specification requirements.

4.0 QUALITY CONTROL

4.1 Quality Control Specialist

The fabricator’s QCPS (Quality Control Paint Specialist) required under SS863, is responsible for all quality control requirements of this specification. The QCPS shall have the testing equipment specified per SS863.29.

Quality Control Points (QCP)

Quality control points (QCP) are points in time when one phase of the work is complete and ready for inspection by the fabricator’s QCPS and the Department’s QA representative. The next operational step must not proceed unless the QCP has been accepted or QA inspection waived by the Department’s QA representative. At these points the Fabricator must afford access to inspect all affected surfaces. If Inspection indicates a deficiency, that phase of the work must be corrected in accordance with these specifications prior to beginning the next phase of work. Discovery of defective work or material after a Quality Control Point is past or failure of the final product before final acceptance, must not in any way prevent rejection or obligate the Department to final acceptance.
# Quality Control Points

<table>
<thead>
<tr>
<th>Quality Control Points (QCP)</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Qualification of operator and equipment</td>
<td>To assure the proposed equipment and operators can perform the work.</td>
</tr>
<tr>
<td>B. Solvent Cleaning</td>
<td>Remove asphaltic cement, oil, grease, salt, dirt etc.</td>
</tr>
<tr>
<td>C. Edge Grinding</td>
<td>Remove sharp corners per AWS</td>
</tr>
<tr>
<td>D. Abrasive Blasting</td>
<td>Blasted surface to receive metallic coating, including repair of fins, tears, slivers or sharp edges, per AWS.</td>
</tr>
<tr>
<td>E. Metalized Coat Application</td>
<td>Check surface cleanliness, apply metalizing, check coating thickness.</td>
</tr>
<tr>
<td>F. Adhesion</td>
<td>Check adhesion of metalizing.</td>
</tr>
<tr>
<td>G. Seal Coat Application</td>
<td>Check surface cleanliness, apply seal coat, check coating thickness.</td>
</tr>
<tr>
<td>H. Field Review</td>
<td>Visual inspection of the system for acceptance.</td>
</tr>
</tbody>
</table>

## A. Qualification of operator and equipment (QCP #1)

The QCPS shall witness and record operator and equipment qualification tests and assure the tests are performed as per the requirements. The QA inspector shall be present to witness the tests.

Each equipment operator must demonstrate the ability to adequately set up and operate the equipment and produce an acceptable coating. Acceptance shall be based on the operator producing adhesion and bend test plates meeting the minimum values required in this specification.

The QCPS must record that each operator demonstrated the ability to apply the metalized coating as specified.

1. **Adhesion Test Plates**
   
   In the presence of the QCPS and QA Inspectors, the applicator must prepare and metalize, per this specification, a 4"x12"x1/4" [100 x 300 x 6 mm] steel plate of the same material as the structural steel to be metalized. An adhesion test shall be run in accordance with ASTM D-4541 and the minimum acceptable value shall be 1000 psi [6.9 MPa]. If a test plate fails the operator will adjust the procedure and re-run the test plate. The maximum number of test plates per operator shall be three. No metalizing work must be performed prior to the approval of the adhesion test plate.
2. Bend Test Plates
In the presence of the QCPS and QA Inspectors, the Applicator must prepare and metalize, as previously specified, a 2" x 8" x 1/8" [50 x 200 x 3.2 mm] coupon of A606 or A588 low carbon steel. The metalizing must be the same thickness as previously specified. Once sprayed, the coupons must be cold bent 180 degrees around a ½ inch [13 mm] diameter mandrel. The metalizing must be on the outside of the coupon. No delamination of the coating is permitted. Cracking of the coating is permitted, provided the coating adheres to the coupon. The coating must adhere to the face of the coupon. The bend test fails if the coating can be picked off with a knife blade.

B. Solvent Cleaning (QCP #2)
The steel must be solvent cleaned were necessary to remove all traces of asphaltic cement, oil, grease, diesel fuel deposits, and other soluble contaminants per SSPC-SP 1. Under no circumstances must any abrasive blasting be done to areas with asphaltic cement, oil, grease, or diesel fuel deposits. The steel must be allowed to dry before blast cleaning begins. The QCPS shall check and document the performance of the solvent cleaning and that defects on exposed surfaces are corrected prior to release to the metalizing process.

C. Grinding Edges (QCP #3)
All corners of thermally cut or sheared edges must have a 1/16 inch [1.6 mm] radius or equivalent flat surface at a suitable angle. Thermally cut material thicker than 1 ½ inch [40 mm] must have the sides ground to remove the heat effected zone, as necessary to achieve the specified surface cleaning. The QCPS must visually inspect and document that the grinding conforms to this specification and provide a cover letter listing each main member inspected.

D. Abrasive Blasting (QCP #4)
Abrasives shall be checked for oil contamination before use. A small sample of abrasives shall be added to ordinary tap water. Any detection of a oil film on the surface of the water is cause for rejection. The QCPS must perform and record this test at the start of each shift. The QCPS shall record the surface profile with replica tape ASTM D4417-93 method C.

For Automated blasting process: Five (5) each recorded readings at random locations on one member for 20% of the main members or one (1) beam per shift (which ever is greater) and one (1) recorded reading for 10% of all secondary material.

For Manual blasting process: five (5) each recorded readings at random locations for each main member and one (1) recorded reading for 25% of all secondary material.

The QCPS shall perform and record the following test to ensure the compressed air is not contaminated: blow air from the nozzle for 30 seconds onto a white cloth or blotter held in a
rigid frame. If any oil or other contaminants are present on the cloth or blotter, abrasive blasting shall be suspended until the problem is corrected and the operation is verified by a repeated test. This test shall be done prior to blowing, at the start of each shift.

All fins, tears, slivers and burried or sharp edges that appear after the blasting operation must be conditioned per ASTM A6 and the area re-blasted to provide the specified surface profile.

All structural steel surfaces must be blast cleaned according to SSPC-SP5. The appearance of the blast-cleaned surface shall match the pictorial standards of SSPC-VIS 1-89. Steel must be maintained in a blast cleaned condition until it has received a metallic coating.

Any abrasive blasting which is done when the steel temperature is less than 5°F [2.8°C] above the dew point must be re-blasted when the steel temperature is at least 5°F [2.8°C] above the dew point.

The abrasive must be recyclable steel grit producing a minimum angular surface profile of three (3) mils. Prior to reuse, the abrasive must be cleaned of paint chips, rust, mill scale and other foreign material by equipment specifically designed for such cleaning.

All abrasives and residue must be removed from all surfaces to be metalized with a vacuum cleaner equipped with a brush-type cleaning tool, or by double blowing.

All corners of thermally cut or sheared edges must have a 1/16 inch radius [1.6 mm] or equivalent flat surface at a suitable angle. Prior to blasting, thermally cut material must have the sides ground as necessary to remove the hardened edges.

E. Metallic Coat Application and Thickness (QCP #5)

The QCPS shall record the time between blasting and application of the metalizing. The QCPS shall record the ambient temperature and dew point no more than one (1) hour before application of the metalizing. Environmental conditions shall be monitored every four (4) hours during the metalizing operation.

Gauges shall be calibrated on the steel surface being metalized. Thickness shall be determined by use of Type 2 magnetic gage in accordance with the following:

Five separate spot measurements must be made, spaced evenly over each 100 square feet [10 square meter] of metalized surface area. Three gage readings must be made for each spot measurement. The gage must be moved a distance of one to three inches [25 to 75 mm] for each new gauge reading. Any unusually high or low gage reading that cannot be repeated consistently must be discarded. The average (mean) of the three (3) gage readings must be used as the spot measurement. The average of five (5) spot measurements for each such 100 square feet [10 square meter] area must not be less that the specified thickness. No single spot measurement in any 100 square feet [10 square meter] area must be less than 80% of the specified minimum thickness. Any one of three (3) readings which are averaged to produce each
spot measurement, may under-run by a greater amount. The five (5) spot measurements must be made for each 100 square feet [10 square meters] of area.

1. General
   The metalizing must coat all surfaces of the structural steel fabricated under SS863, Structural Steel Members. Metalizing and welding in the vicinity of previously coated (metalized, sealed or other) surfaces must be conducted in a manner that prevents molten metal from striking the coating and otherwise minimizes coating damage. Coatings damaged by welding or metalizing operations must be repaired in accordance with this specification.

   Prior to application of the metallic coating, the steel surface to be metalized must meet the requirements under “Surface Preparation”.

   The metalizing must not be applied to a surface which shows any sign of surface moisture. Thermal spraying must not be performed when the steel temperature is less than 5°F [2.8°C] above the dew point. If flame spray equipment is used, the initial starting area must be pre-heated to 250°F [120°C] before application of the metalized coating. Each time the operator stops the coating operation the pre-heat needs to be re-done. The minimum application temperature is 0°F [-32°C]

2. Equipment and Techniques
   The metalizing may be applied using either combustion flame spraying equipment or by electric arc spraying equipment operated in accordance with the manufacturer's latest written instructions, including but not limited to air pressure and gun angle relative to the work surface.

   Thermal spray operators must apply the metalizing in a manner that promotes uniform coverage and prevents discontinuity of the applied coating. Spraying must be performed in a block pattern, typically two to three feet square [0.4 to 1.0 square meter]. The metalizing must overlap fifty percent (50%) on each pass to ensure uniform coverage. The required coating thickness must be obtained in multiple layers. Each layer must be applied at right angles to the previous layer. Spraying distance should be between 6 to 8 inches [150 to 200 mm] from the work surface to ensure the metal is plastic on impact. Any defects must be immediately corrected.

   Startup and adjustment of thermal spray equipment must be done off the surface being metalized.

3. Metalizing Thickness
   The metalizing must be applied to a minimum thickness of twelve (12) mils [300 µm]. In general, any pass must not deposit more than four (4) mils [100 µm]. The top of the top flange must receive a coat not less than 0.5 mils [13 µm] nor more than 1.5 mils [40 µm].

   Metalizing to the specified thickness must be completed within twelve (12) hours of blasting and before deterioration of the steel from the specified surface cleanliness. The
time between metalizing passes must be four (4) hours.

4. Metalizing Wire

Pure zinc (99.99%) metalizing wire must be used, powder material will not be accepted. The wire used for metalizing must be in accordance with ASTM B833. The wire manufacturer must provide certified test data per SS863.09.

F. Adhesion (QCP #6)

Before sealing, the QCPS must test for adhesion in accordance with ASTM D 4541. The QCPS must perform five (5) tests at random locations selected by the QA Inspector on the first main member, including splice plates. Thereafter, ten percent (10%) of the main material must be tested in the same manner. Five percent (5%) of all secondary material must be tested at a rate of one (1) recorded reading. The minimum acceptable adhesion value shall be 1000 psi [6.9 MPa].

G. Seal Coat Application and Thickness (QCP #7)

1. General

The areas of structural steel that are embedded in, partially embedded in, or in contact with cast-in-place concrete must be sealed (e.g., top and sides of top flange). When the structure is an integral abutment, all metalized surfaces from the end of the beam to one (1) foot 9 [0.3 meter] past the abutment interface must be sealed.

2. Sealer Material

An air dried clear phenolic, either “Metallizing Sealer #9876” or “9127”, manufactured by Keeler and Long (PPG)P.O. Box 460, Watertown CT 06795, or “METCO AP” manufactured by METCO, or equal to METCO AP manufactured by Akron Paint & Varnish.

3. Sealers

All sealer must be delivered to the Fabricator in original, unopened containers with labels intact. Minor damage to containers is acceptable provided the container has not been punctured.

Sealer must be stored at the temperature recommended by the manufacturer to prevent sealer deterioration.

The Applicator must provide thermometers capable of monitoring the maximum high and low temperatures within the storage facility. The Applicator is responsible for properly disposing of all unused sealer and sealer containers.

The QCPS must record the storage temperature, lot and stock numbers, and date of manufacture of the sealant.
All containers of sealer must remain unopened until use. The label information must be legible and checked at the time of use. Solvent used for cleaning equipment is exempt from the above requirements.

Each container of sealer must be clearly marked or labeled to show sealer identification, component, lot number, stock number, date of manufacture, and information and warnings as may be required by Federal and State laws.

Sealer which has livered, gelled or otherwise deteriorated during storage must not be used. The oldest sealer of each kind must be used first. No sealer must be used which has surpassed its shelf life.

4. Environmental Conditions
Sealer must not be applied when the metalizing steel surface temperature is less than 5° F [2.8°C] above the dew point. The surfaces to be sealed must be dry. Sealers shall not be applied in rain, snow, fog or mist, or to frosted or ice-coated surfaces. The ambient temperature must be per the sealer manufacturer’s recommendations.

5. Equipment and Techniques
Sealers must be applied by spray methods. Spray equipment must be kept clean so dirt, dried sealer and other foreign materials are not deposited in the sealer film. Any solvent left in the equipment must be completely removed before using.

The QCPS must document that all spray equipment used follows the sealer manufacturer's equipment recommendations. Equipment must be suitable for use with the specified sealer to avoid sealer application problems.

If air spray is used, traps or separators must be provided to remove oil and condensed water from the air. The traps or separators must be of adequate size and must be drained periodically during operations.

The seal coat thickness must be a minimum of 1 mil [25 µm] over the surface and peaks of the metalizing. This thickness shall be verified by the QCPS. Pinholing must be avoided by method of application.

The Applicator must take precaution to prevent contamination of surfaces that have been prepared for sealing and surfaces freshly sealed. The surface to be sealed shall be clean and dry. The sealer must be applied within eight (8) hours of the completion of the metalizing.

The sealer must be applied in the shop. The steel must not be handled unnecessarily or removed from the shop until the sealer has dried sufficiently to resist being marred in handling and shipping.
The QCPS must record the time between metalizing and seal coat application. The QCPS must record the ambient temperature and humidity no more than one (1) hour before application of the seal coat. Environmental conditions must be monitored every four (4) hours during the sealing operation.

The same test for abrasive blasting air contamination must be made by the sealer applicator and verified by the QCPS to ensure that the traps or separators are working properly. This is not required for an airless sprayer.

**H. Field Review (QCP #8)**

While approval of the shop metallized coating is required before shipment there will be possible damage to the metalized coating and the sealer due to field erection and construction operations.

Specified areas of field welding must require the removal of the metalized coating by grinding or other methods acceptable to the Engineer.

Application of welded shear studs will require welding through a thin layer of zinc coating similar to in-organic zinc paint. The welding of the studs will not require removal of the metalized coating, but stud welding procedures may require adjustment to meet the production control requirements, section 7.7, of AWS D1.5 bridge welding code. Other areas of field welding will require removal of the metalized coating. Removal limits should be minimized to where the welding is applied.

Small areas of metalizing less than one (1) square foot [0.1 square meter] that are removed, marred, damaged, or rejected must be repaired according to ASTM A780, Annex 1 or 3. Larger areas of metalizing must be repaired by removal and replacement as specified herein. The coating at the welded shear stud locations will not require repair. At the completion of construction, the metalized coating must be undamaged and the surfaces free from grease, oil, chalk marks, paint, concrete splatter or other soilage.

The Engineer must visually inspect the metalizing to establish that field welded and damaged areas are repaired as per this specification.

**5.0 HANDLING AND SHIPPING**

Reasonable care must be exercised in handling the metalized steel during shipping, erection, and subsequent construction of the bridge. The steel must be insulated from the binding chains by softeners. Hooks and slings used to hoist steel must be padded. Diaphragms and similar pieces must be spaced in such a way that no rubbing will occur during shipment that may damage the metalizing. The steel must be stored on pallets at the job site, or by other means, so that it does not rest on the ground or so that components do not fall or rest on each other.
6.0 SAFETY REQUIREMENTS AND PRECAUTIONS

The Contractor must meet the safety requirements of the Ohio Industrial Commission and the Occupational Safety and Health Administration (OSHA), in addition to the scaffolding requirements below.

The Contractor is required to meet the applicable safety requirements of the Ohio Industrial Commission in addition to the scaffolding requirements specified below.

7.0 SCAFFOLDING

Rubber rollers, or other protective devices meeting the approval of the Engineer, must be used on scaffold fastenings. Metal rollers or clamps and other types of fastenings which will mar or damage coated surfaces must not be used.

8.0 INSPECTION ACCESS FOR FIELD REPAIR

In addition to the requirement of 105.11, the contractor must furnish, erect, and move scaffolding and other appropriate equipment, to permit the Inspector the opportunity to inspect closely observe), all affected surfaces. This opportunity must be provided to the Inspector during all phases of the work and continue for a period of at least ten (10) working days after the touch-up work has been completed. When scaffolding is used, it must be provided in accordance with the following requirements. When scaffolding, or the hangers attached to the scaffolding are supported by horizontal wire ropes, or when scaffolding is placed directly under the surface to be painted, the following requirements must be complied with:

When scaffolding is suspended 43" [1100 mm] or more below the coated surface to be repaired, two rows of guardrail must be placed on all sides of the scaffolding. One row of guardrail must be placed at 42" [1050 mm] above the scaffolding and the other row at 20" [500 mm] above the scaffolding.

When the scaffolding is suspended at least 21" [530 mm], but less than 43" [1100 mm] below the coated surface to be repaired, a row of guardrail must be placed on all sides of the scaffolding at 20" [500 mm] above the scaffolding.

Two rows of guardrail must be placed on all sides of scaffolding not previously mentioned. The rows of guardrail must be placed at 42" [1050 mm] and 20" [500 mm] above scaffolding, as previously mentioned.

All scaffolding must be at least 24" [610 mm] wide when guardrail is used and 28" [710 mm] wide when the scaffolding is suspended less than 21" [530 mm] below the coated surface to be repaired and guardrail is not used. If two or more scaffolding are laid parallel to achieve the proper width, they must be rigidly attached to each other to preclude any differential movement.

All guardrail must be constructed as a substantial barrier which is securely fastened in place and
is free from protruding objects such as nails, screws and bolts. There must be an opening in the guardrail, properly located, to allow the Inspector access onto the scaffolding.

The rails and uprights must be either metal or wood. If pipe railing is used, the railing must have a nominal diameter of no less than one and one half inches. If structural steel railing is used, the rails must be 2 X 2 X 3/8 inch [50 x 50 x 10 mm] steel angles or other metal shapes of equal or greater strength. If wood railing is used, the railing must be 2 X 4 inch [50 x 100 mm] (nominal) stock. All uprights must be spaced at no more than 8 feet [2.4 m] on center. If wood uprights are used, the uprights must be 2 X 4 inches [50 x 100 mm] (nominal) stock.

When the surface to be inspected is more than 15 feet [4.6 m] above the ground or water, and the scaffolding is supported from the structure being painted, the Contractor must provide the Inspector with a safety belt and lifeline. The lifeline must not allow a fall greater than 6 feet [2 m]. The Contractor must provide a method of attaching the lifeline to the structure independent of the scaffolding, cables, or brackets supporting the scaffolding.

When scaffolding is more than two and one half feet [0.75 m] above the ground, the Contractor must provide a ladder for access onto the scaffolding. The ladder and any equipment used to attach the ladder to the structure must be capable of supporting 250 pounds [115 kg] with a safety factor of at least four (4). All rungs, steps, cleats, or treads must have uniform spacing and must not exceed 12" [305 mm] on center. At least one side rail must extend at least 36" [915 mm] above the landing near the top of the ladder.

An additional landing must be required when the distance from the ladder to the point where the scaffolding may be accessed, exceeds 12" [305 mm]. The landing must be a minimum of at least 24" [610 mm] wide and 24" [610 mm] long. It must also be of adequate size and shape so that the distance from the landing to the point where the scaffolding is accessed does not exceed 12" [305 mm]. The landing must be rigid and firmly attached to the ladder; however, it must not be supported by the ladder. The scaffolding must be capable of supporting a minimum of 1000 lbs [455 kg].

In addition to the aforementioned requirements, the Contractor is still responsible to observe and comply with all Federal, State and local laws, ordinances, regulations, orders and decrees.

The Contractor must furnish all necessary traffic control to permit inspection during and after all phases of the project.

9.0 PROTECTION OF PERSONS AND PROPERTY

The Contractor must install and maintain suitable shields or enclosures to prevent damage to adjacent buildings, parked cars, trucks, boats, or vehicles traveling on, over, or under structures having galvanized repairs. They must be suitably anchored and reinforced to prevent interfering with normal traffic operations in the open lanes. Payment for the shields must be included as incidental to the applicable field coating operation. Work must be suspended when
damage to adjacent buildings, motor vehicles, boats, or other property is occurring.

When or where any direct or indirect damage or injury is done to public or private property, the Contractor must restore, at his own expense, such property, to a condition similar or equal to that existing before such damage or injury was done.

10.0 POLLUTION CONTROL

The Contractor must take all necessary precautions to comply with pollution control laws, rules or regulations of Federal, State or local agencies.

11.0 METHOD OF MEASUREMENT

The cost of all labor, materials, equipment necessary to metalize, to fabricate the structural steel in accordance with SS863 and perform any necessary field repair shall be included in this SS 863, as per plan item.

12.0 BASIS OF PAYMENT

Payment will be made at contract prices for the applicable 863 as per plan item.
AN-8  GALVANIZED COATING SYSTEM FOR STRUCTURAL STEEL BRIDGES

This note establishes specification requirements for the application of a galvanized coating for new structural steel bridges. To use this note the designer must have designed the steel bridge’s high strength bolted connections for a slip resistance based on the coefficient for galvanized coatings as defined in AASHTO. Also the connections must accounted for a 1/8" [3 mm] greater diameter hole than the bolt rather than the standard 1/16" [1.6 mm] greater diameter hole. The use of standard rolled beam bolted splice details BS-1-93 (BS-1-93M) cannot be used. A completely detailed splice shall be shown in the bridge plan details. Bolted cross frame connection details defined in standard drawing GSD-1-96 may be used and referenced.

As example:

Item 863  Lump Sum  STRUCTURAL STEEL MEMBERS, LEVEL ?, AS PER PLAN

The note AN-7 follows as the AS PER PLAN specifications to have a shop applied metallized coating on the structural steel members.

1.0  DESCRIPTION

In addition to the requirements of Supplemental Specification 863, this item shall consist of furnishing all necessary labor, materials and equipment to clean and galvanize all structural steel surfaces, as specified herein. The galvanized coating system may be applied by a galvanizer not qualified as a fabrication shop under Supplemental Specification 863 (SS 863), but the approved fabricator of the structural steel shall be responsible for the quality of the applied galvanized coating system and any repairs, re-fabricating, additional laydowns required to assure the fabricated steel meets all requirements of this specification. Sections 863.29 and 863.30 shall not apply.

This item shall also include galvanizing, per 711.02, of all nuts, washers, bolts, anchor bolts.

Any shear studs, section 863.24, shall be installed in the fabricator’s shop before galvanizing.

2.0  PRE-FABRICATION MEETING

In addition to the pre-fabrication meeting requirements under SS 863.081, both the fabricator’s Quality Control Specialist, (QCPS) and galvanized coating applicator shall be present and discuss methods of operation, quality control, including repairs, transportation, erection methods to accomplish all phases of the preparation and coating work required by this specification.
3.0 QUALITY CONTROL

3.1 Quality Control Specialist

The QCPS (Quality Control Paint Specialist) required under SS863, is responsible for all quality control requirements of this specification. The QCPS shall have the testing equipment specified in SS863.29.

3.2 Quality Control Points (QCP)

Quality control points (QCP) are points in time when one phase of the work is complete and ready for inspection by the fabricator’s QCPS and the Department’s QA representative. The next operational step must not proceed unless the QCP has been accepted or QA inspection waived by the Department’s QA representative. At these points the Fabricator must afford access to inspect all affected surfaces. If Inspection indicates a deficiency, that phase of the work must be corrected in accordance with these specifications prior to beginning the next phase of work. Discovery of defective work or material after a Quality Control Point is past or failure of the final product before final acceptance, must not in any way prevent rejection or obligate the Department to final acceptance.

<table>
<thead>
<tr>
<th>Quality Control Points (QCP)</th>
<th>PURPOSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Solvent Cleaning</td>
<td>Remove asphaltic cement, oil, grease, salt, dirt, etc.</td>
</tr>
<tr>
<td>B. Grinding Edges</td>
<td>Remove sharp corners per AWS.</td>
</tr>
<tr>
<td>C. Abrasive Blasting</td>
<td>Blast surfaces, including repair fins, tears, slivers or sharp edges.</td>
</tr>
<tr>
<td>D. Galvanizing</td>
<td>Check coating thickness</td>
</tr>
<tr>
<td>E. Faying Surface Cleaning</td>
<td>Check faying surface roughness. Check bolt hole clearance. Check for other field connections uniform coating thickness.</td>
</tr>
<tr>
<td>F. Second lay down</td>
<td>Check sweep and camber tolerances of each Structural Member.</td>
</tr>
<tr>
<td>G. Field Repair of Damage Areas</td>
<td>Check for damage areas after erection of structure. Perform damage repairs.</td>
</tr>
<tr>
<td>H. Final Review</td>
<td>Clean structure as per QCP#1. Visually inspect system for acceptance.</td>
</tr>
</tbody>
</table>
A. Solvent Cleaning (QCP #1)

The steel must be solvent cleaned were necessary to remove all traces of asphaltic cement, oil, grease, diesel fuel deposits, and other soluble contaminants per SSPC-SP 1 Solvent Cleaning. Under no circumstances must any abrasive blasting be done to areas with asphaltic cement, oil, grease, or diesel fuel deposits. Steel must be allowed to dry before blast cleaning begins. The QCPS shall inspect and document that the cleaning conforms to SSPC-SP1 and provide a cover letter listing each main member inspected.

B. Grinding Edges (QCP #2)

All corners of thermally cut or sheared edges must have a 1/16 inch [1.6 mm] radius or equivalent flat surface at a suitable angle. Thermally cut material thicker than 1 ½ inch [40 mm] must have the sides ground to remove the heat affected zone, as necessary to achieve the specified surface cleaning. The QCPS must visually inspect and document that the grinding conforms to this specification and provide a cover letter listing each main member inspected.

C. Abrasive Blasting (QCP #3)

Beams and Girders must be prepared by the fabricator to Steel Structures Painting Council (SSPC) grade six(6) Commercial blast cleaning prior to galvanizing. All material must be free of paint marks. Secondary angle, plates, bars and shapes need not be blast cleaned.

Abrasives must also be checked for oil contamination before use. A small sample of abrasives must be added to ordinary tap water. Any detection of a oil film on the surface of the water must be cause for rejection. The QCPS must perform and record this test at the start of each shift.

All fins, tears, slivers and burred or sharp edges that are present on any steel member or that appear after the blasting operation must be conditioned per ASTM A6. Welding repairs must only be performed by the SS863 Fabricator

The QCPS must visually inspect and document that the blast conforms to SSPC-SP6, that all conditioning is performed per ASTM A6, and provide a cover letter listing each main member inspected.

D. Galvanizing (QCP #4)

Galvanized per 711.02 and this specification. Coating thickness must be a minimum of 4 mils [100 µm] measured as specified.

Material must be free of imperfections or depressions caused by material handling. The fabricator, galvanizer and erector must use lifting clamps or softeners for handling. Prior to galvanizing, surface imperfections may be repaired by the fabricator in conformance with ASTM A6. Imperfections greater than the limits allowed by ASTM A6 must be documented. Repair or replacement of this member will be at the discretion of the Department.
All damaged galvanizing must be repaired in accordance with ASTM A780, method A1 or A3.

Documentation of coating thickness must be performed by the QCPS. The QCPS must record the gage readings and provide a cover letter listing each main member inspected.

**E. Faying Surface Cleaning (QCP #5)**

Areas of field connections must have a uniform galvanized coating thickness free of local excessive roughness which would prevent splice plates, bearings or other field connections from making intimate contact.

Faying surfaces of the bolted splices must be roughened in the shop after galvanizing by hand wire brushing. Power wire brushing is not permitted. All field splice bolt holes must be free of zinc build up. After galvanizing, each hole must be checked in the shop by using a drift pin with a diameter 1/16 inch [1.6 mm] greater than the diameter of the bolt to be used in that hole. Consideration will be given to other methods of treating the faying surfaces if a written request is submitted to the Office of Structural Engineering (OSE) in accordance with CMS 108.05.

Inspection of the roughening of the faying surfaces and checking of holes with drift pins must be performed by the QCPS. Acceptance of the faying surfaces and holes shall be documented by the QCPS.

**F. Second lay down (QCP # 6)**

After galvanizing, materials must be placed in a second shop assembly per CMS section 863.26 to check alignment of holes, sweep and camber against the fabricators original recorded lay down dimensions. This shop assembly may be performed at the galvanizers facility, by the fabricators personnel, if approved by the OSE. The second lay down may be waived by the OSE if the fabricator records individual beam cambers and sweeps during the first lay down, and the new individual beam cambers and sweeps, after galvanizing, compared to the first lay down are within the following tolerances:

- Bearing points after galvanizing, must be within +/- 1/8 inch [3.2 mm] of the approved shop drawing lay down.
- Camber points after galvanizing must be + 1/4 inch [6 mm] or - 0 inch from the first lay down.
- Sweep points after galvanizing must be +/- 3/8 inch [9 mm] from the first lay down.

Individual Beams that exceed the listed tolerances must be placed with at least two adjacent beams in Lay down for checking against the recorded shop assembly records per 863.07. Documentation of the second lay down or individual member cambers must be recorded by the QCPS per 863.26.
G. Field Repair of Damaged Areas (QCP #7)

Material must be free of imperfections or depressions caused by material handling. The Contractor must use lifting clamps or softeners for handling. Imperfections may be repaired by grinding as allowed by ASTM A6 by the Contractor. Imperfections that are greater than the grinding limits allowed by ASTM A6, must be documented. Repair or replacement of this member will be at the discretion of the OSE.

All damaged galvanizing must be repaired in accordance with ASTM A780, method A1 or A3.

Damaged galvanizing which will be inaccessible for repair after erection must be repaired prior to erection.

In order to minimize damage to the galvanized steel, concrete splatter and form leakage must be washed from the surface of the steel shortly after the concrete is placed and before it is dry. If the concrete dries, it must be removed.

Temporary attachments, supports for scaffolding and finishing machine or forms must not damage the coating system. In particular, sufficient size support pads must be used on the fascias where bracing is used.

Documentation of galvanizing repairs must be performed by the QCPS by a cover letter listing each main member inspected.

H. Final Review (QCP #8)

After the erection work has been completed, including all connections and the approved repair of any damaged beams, girders or other steel members, and the deck has been placed, the Contractor and Engineer must inspect the structure for damaged coating. (QCP #8). Damaged areas must be repaired by QCPS #7. At the completion of construction, the galvanizing must be undamaged and the surfaces free from grease, oil, chalk marks, paint, concrete splatter or other silage. Such silage will be removed by solvent cleaning per SSPC-SP1(QCP #1)

Documentation of Final review must be performed by the QCPS by a cover letter listing each main member inspected.

4.0 TESTING EQUIPMENT

The Fabricator must provide the QCPS inspector the following testing equipment in good working order for the duration of the project.

One (Positector 2000 or 6000, Quanix 2200, or Elcometer A345FB11) and the calibration plates, 38-200 mm and 250-625 mm [1.5 -8 mils and 10-25 mils] as per the NBS calibration standards in accordance with ASTM D-1186.
5.0 COATING THICKNESS

Galvanized thickness must be determined by use of type 2 magnetic gage in accordance with the following:

Five separate spot measurements must be made, spaced evenly over one (1) randomly selected, 100 square feet [9 square meters] of surface area on each structural member. Three gage readings must be made for each spot measurement. The probe must be moved a distance of 1 to 3 inches [25 to 75 mm] for each new gage reading. Any unusually high or low gage reading that cannot be repeated consistently must be discarded. The average (mean) of the 3 gage readings must be used as the spot measurement. The average of five spot measurements for each such 100 square foot [9 square meter] area must not be less than the specified thickness. No single spot measurement in any 100 square foot [9 square meter] area must be less than 80% of the specified minimum thickness. Any one of 3 readings which are averaged to produce each spot measurement, may under-run or over-run by a greater amount. The 5 spot measurements must be made for one (1) randomly selected, 100 square feet [9 square meter] of area on each structural member. All Splice material and secondary members must have at least one spot measured on each piece. The probe must be moved so that one reading is taken at each end and middle of the piece for a total of three readings.

The QCPS must inspect and provide documentation of actual data, the galvanized thickness checks were performed per specification, and the coating thickness meets specification requirements.

6.0 HANDLING AND SHIPPING

Reasonable care must be exercised in handling the galvanized steel during shipping, erection, and subsequent construction of the bridge. The steel must be insulated from the binding chains by softeners. Hooks and slings used to hoist steel must be padded. Diaphragms and similar pieces must be spaced in such a way that no rubbing will occur during shipment that may damage the galvanizing. The steel must be stored on pallets at the job site, or by other means, so that it does not rest on the ground or so that components do not fall or rest on each other.

7.0 SAFETY REQUIREMENTS AND PRECAUTIONS

The contractor must meet the safety requirements of the Ohio Industrial Commission and the Occupational Safety and Health Administration (OSHA), in addition to the scaffolding requirements below.

The Contractor is required to meet the applicable safety requirements of the Ohio Industrial Commission in addition to the scaffolding requirements specified below.
8.0 SCAFFOLDING

Rubber rollers, or other protective devices meeting the approval of the Engineer, must be used on scaffold fastenings. Metal rollers or clamps and other types of fastenings which will mar or damage coated surfaces must not be used.

9.0 INSPECTION ACCESS FOR FIELD REPAIR

In addition to the requirement of 105.11, the contractor must furnish, erect, and move scaffolding and other appropriate equipment, to permit the Inspector the opportunity to inspect closely (observe), all affected surfaces. This opportunity must be provided to the Inspector during all phases of the work and continue for a period of at least ten (10) working days after the touch-up work has been completed. When scaffolding is used, it must be provided in accordance with the following requirements. When scaffolding, or the hangers attached to the scaffolding are supported by horizontal wire ropes, or when scaffolding is placed directly under the surface to be painted, the following requirements must be complied with:

When scaffolding is suspended 43" [1100 mm] or more below the coated surface to be repaired, two rows of guardrail must be placed on all sides of the scaffolding. One row of guardrail must be placed at 42" [1050 mm] above the scaffolding and the other row at 20" [500 mm] above the scaffolding.

When the scaffolding is suspended at least 21" [530 mm], but less than 43" [1100 mm] below the coated surface to be repaired, a row of guardrail must be placed on all sides of the scaffolding at 20" [500 mm] above the scaffolding.

Two rows of guardrail must be placed on all sides of scaffolding not previously mentioned. The rows of guardrail must be placed at 42" [1050 mm] and 20" [500 mm] above scaffolding, as previously mentioned.

All scaffolding must be at least 24" [610 mm] wide when guardrail is used and 28" [710 mm] wide when the scaffolding is suspended less than 21" [530 mm] below the coated surface to be repaired and guardrail is not used. If two or more scaffolding are laid parallel to achieve the proper width, they must be rigidly attached to each other to preclude any differential movement.

All guardrail must be constructed as a substantial barrier which is securely fastened in place and is free from protruding objects such as nails, screws and bolts. There must be an opening in the guardrail, properly located, to allow the Inspector access onto the scaffolding.

The rails and uprights must be either metal or wood. If pipe railing is used, the railing must have a nominal diameter of no less than one and one half inches. If structural steel railing is used, the rails must be 2 X 2 X 3/8 inch [50 x 50 x 10 mm] steel angles or other metal shapes of equal or greater strength. If wood railing is used, the railing must be 2 X 4 inch [50 x 100 mm] (nominal) stock. All uprights must be spaced at no more than 8 feet [2.4 m] on center. If wood uprights are used, the uprights must be 2 X 4 inches [50 x 100 mm] (nominal) stock.
When the surface to be inspected is more than 15 feet [4.6 m] above the ground or water, and the scaffolding is supported from the structure being painted, the Contractor must provide the Inspector with a safety belt and lifeline. The lifeline must not allow a fall greater than 6 feet [2 m]. The Contractor must provide a method of attaching the lifeline to the structure independent of the scaffolding, cables, or brackets supporting the scaffolding.

When scaffolding is more than two and one half feet [0.75 m] above the ground, the Contractor must provide a ladder for access onto the scaffolding. The ladder and any equipment used to attach the ladder to the structure must be capable of supporting 250 pounds [115 kg] with a safety factor of at least four (4). All rungs, steps, cleats, or treads must have uniform spacing and must not exceed 12” [305 mm] on center. At least one side rail must extend at least 36” [915 mm] above the landing near the top of the ladder.

An additional landing must be required when the distance from the ladder to the point where the scaffolding may be accessed, exceeds 12” [305 mm]. The landing must be a minimum of at least 24” [610 mm] wide and 24” [610 mm] long. It must also be of adequate size and shape so that the distance from the landing to the point where the scaffolding is accessed does not exceed 12” [305 mm]. The landing must be rigid and firmly attached to the ladder; however, it must not be supported by the ladder. The scaffolding must be capable of supporting a minimum of 1000 lbs [455 kg].

In addition to the aforementioned requirements, the Contractor is still responsible to observe and comply with all Federal, State and local laws, ordinances, regulations, orders and decrees.

The Contractor must furnish all necessary traffic control to permit inspection during and after all phases of the project.

### 10.0 PROTECTION OF PERSONS AND PROPERTY

The Contractor must install and maintain suitable shields or enclosures to prevent damage to adjacent buildings, parked cars, trucks, boats, or vehicles traveling on, over, or under structures having galvanized repairs. They must be suitably anchored and reinforced to prevent interfering with normal traffic operations in the open lanes. Payment for the shields must be included as incidental to the applicable field coating operation. Work must be suspended when damage to adjacent buildings, motor vehicles, boats, or other property is occurring.

When or where any direct or indirect damage or injury is done to public or private property, the Contractor must restore, at his own expense, such property, to a condition similar or equal to that existing before such damage or injury was done.
11.0 POLLUTION CONTROL

The Contractor must take all necessary precautions to comply with pollution control laws, rules or regulations of Federal, State or local agencies.

12.0 METHOD OF MEASUREMENT

The cost of all labor, materials, equipment necessary to galvanize and to fabricate the structural steel in accordance with SS863 and perform any necessary field repair shall be included in this SS 863, as per plan item.

13.0 BASIS OF PAYMENT

Payment will be made at contract prices for the applicable 863 as per plan item.
AN-9 FINGER JOINTS FOR BRIDGES

This note establishes specification requirements for the fabrication and installation of a finger type expansion joint system for bridges. This note specifies either a galvanized or metallized coating for the final joint. This note does not eliminate the need for the designer to have complete plan details of the joint, the trough configuration, and connections to the superstructure and abutment backwall. If requested, the Office of Structural Engineering can furnish the designer details from previous projects.

ITEM 863 STRUCTURAL STEEL MEMBER, MISCELLANEOUS LEVEL FABRICATION, AS PER PLAN

1.0 DESCRIPTION

In addition to the requirements of Supplemental Specification 863, this item shall consist of furnishing all necessary labor, materials, equipment and incidentals necessary to fabricate, test and install a Finger joint expansion system with nylon reinforced neoprene drain troughs in accordance with the plans and this specification. The fabricator must be pre-qualified as a Miscellaneous Level Fabricator.

2.0 MATERIALS

A. Steel plates shall be ASTM A709 grade 50 or 50W.
B. High Strength bolts 711.09
C. Drain trough shall be nylon reinforced neoprene sheeting (NRNS) as specified.
D. Anchor bolts 711.10

3.0 FABRICATION

All complete penetration welds made in the fabrication shop must be 100 % ultrasonically tested and evaluated based on tension criteria of 863.27.

The finger joint system must be shop assembled with all components except drain trough, in a unit(s) no smaller than each construction phase. The shop assembly must be set to required dimensions per 863.26.

The shop assembly check must include a procedure to open and close the finger joint system thorough the full range of the design movement to check for alignment, clearance and fit of the fingers.

Temporary shipping devices must be used to secure the expansion device as a single unit and provide for field adjustment. Temporary/shipping connections must not damage the coatings.
4.0 COATINGS

All steel components shall be either galvanized or metalized. All steel fasteners and anchors shall be galvanized.

The galvanized coating shall meet the requirements of 711.02.

The metalized coating option must have a minimum thickness of 8 mils (200 µm). The metallizing wire must be 100% zinc. Surface preparation and application must conform to SSPC coating system guide No. 23. “Guide For Thermal Spray Metallic Coating Systems”. The areas of structural steel that are in contact with cast-in-place concrete must be sealed. The sealer must be an air dried clear phenolic, either “Metallizing Sealer #9876” or “9127”, manufactured by Keeler and Long (PPG) P.O. Box 460, Watertown CT 06795, or “METCO AP” manufactured by METCO, or equal to METCO AP manufactured by Akron Paint & Varnish.

Field repairs to both coating systems, including field welded splices, shall comply with ASTM A780 method A1 or A3.

5.0 NYLON REINFORCED NEOPRENE SHEET (NRNS) TROUGH MATERIAL

Troughs must be shop fabricated with minimum splices from nylon reinforced neoprene sheet (NRNS). The sheet material must be 3/32 inch [2.5mm] thick, general purpose, heavy-duty neoprene sheet with nylon fabric reinforcement. The NRNS must be Fairprene number NN-0003 as manufactured by Fairprene Inc. or an approved equal. The sheeting must conform to the following:

<table>
<thead>
<tr>
<th>Description of Test</th>
<th>ASTM Method</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>D751</td>
<td>3/32”[2.5mm] ± 10%</td>
</tr>
<tr>
<td>Breaking Strength, Grade WXF, Lbs[N]. Minimum</td>
<td>D751</td>
<td>700 Lbs. X 700 Lbs.[3130 N x 3130 N]</td>
</tr>
<tr>
<td>Adhesive 1&quot;x 2&quot;[25mm x 50 mm] Lbs/Inch[ N/mm]. Minimum.</td>
<td>D751</td>
<td>9 Lbs/Inch[40 N/mm]</td>
</tr>
<tr>
<td>Burst Strength (Mullen) Lbs/SQ.Inch Minimum..</td>
<td>D751</td>
<td>1400psi [9.65 KPa]</td>
</tr>
<tr>
<td>Heating Aging 70 hours at 212 F[100 C], 180 Bend without cracking</td>
<td>D2136</td>
<td>No cracking of coating</td>
</tr>
<tr>
<td>Low Temperature Brittleness 1 hour at -40 F[- 40 C], bend around 1/4”[6 mm] Mandrel</td>
<td>D2136</td>
<td>No cracking of coating</td>
</tr>
</tbody>
</table>
Each lot of NRNS sheeting must be tested by an independent laboratory to ensure compliance with these provisions. One copy of the qualification test data indicating that the tested materials comply with these provisions must be submitted with the material test reports per 863.09.

Shop Drawings for the NRNS troughs will be required per 863.08. Drawings, in addition to details required to dimensionally define the complete trough, shall also include a full length layout of the trough showing splice locations, hole patterns, splice details and connections to drainage outlet systems.

Holes in NRNS troughs for trough fasteners must be drilled or punched after the trough has been match marked with holes or fasteners that secure the trough to the finger joint. The match marking shall be performed by either assembling the trough and finger joint or by use of 1/8" thick steel templates matched against the finger joint and then used to drill the holes in the trough. Alternate match drilling procedures may be proposed by the Contractor.

All elements of the NRNS trough including but not limited to splices, fills, edge reinforcement, etc. must be vulcanized bonded in the shop under heat and pressure. Field splices are not allowed. The method of vulcanizing to be used shall develop a completed joint with a breaking strength equal to the breaking strength of the NRNS. A sample test of the proposed method and joint shall be performed to verify the minimum required strength before the trough is constructed.

6.0 FINGER JOINT INSTALLATION

The Engineer shall verify that alignment, elevations and finger clearances of the finger joint are acceptable prior to permanent connection to the superstructure and between construction phases. All complete penetration field welds must be 100% ultrasonically tested per AWS D1.5-95 Bridge Welding Code, with tension acceptance criteria.

Where a finger joint is installed at an abutment, backwall concrete shall not be placed until the concrete deck pour is complete. Before the backwall concrete is placed, the Engineer must re-check alignment, elevations, finger clearances and temperature adjustment. All temporary bracing that will prevent temperature movement must be loosened or removed as soon as back wall concrete is capable of supporting the finger joint.

The NRNS trough must be furnished and installed for the full joint width after all phases of expansion joint armor installation are complete.

7.0 METHOD OF MEASUREMENT

The lump sum price bid must include the cost of all labor, materials and equipment necessary to supply, install and test a Finger Joint Expansion Joint System according to these specifications.
8.0 BASIS OF PAYMENT

Payment will be made at contract prices for:

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>863</td>
<td>Lump Sum</td>
<td>Structural Steel Member, Miscellaneous Level Fabrication, As Per Plan</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Finger Joint Expansion Device)</td>
</tr>
</tbody>
</table>
AN-10 HIGH PERFORMANCE STEEL (GRADE 70)

The following plan note establishes specification requirements for the fabrication of hybrid girders using high performance, grade 70, steel. Provide this note in the structures General Notes.

Item 513, Structural Steel Members, Hybrid Girder, Level Six Fabrication, As Per Plan:

1. DESCRIPTION:
   1.01 This work consist of furnishing all necessary labor, materials and equipment to furnish and erect structural steel members, designed as a hybrid/ mix of steel materials consisting of: ASTM A709, High Performance grade HSP70W in combination with Grade 50W steel.

   1.02 This work shall be performed per Item 513 Structural Steel Member, level six(6) except as modified by the June, 2003 2nd edition of the “Guide for Highway Bridge Fabrication with HPS70W Steel (HPS485W), A supplement to ANSI/AASHTO AWS D1.5” and as modified by these plan notes.

2. MATERIALS:
   2.01 Steel for girder webs and flanges shall be a combination of ASTM A709 grade HPS70W manufactured by the Thermo-Mechanical Controlled Processing (TMCP) or Quenched and Tempered Heat Treatment Processing along with ASTM A588/709 Grade 50W. All other steel shall be ASTM A709 grade 50W.

   2.02 Steel designated CVN shall be impact tested to exceed the test values of ASTM A709 table S1.2 “Non-Fracture Critical Impact Test Requirements” for zone 2, temperature range.

3. ADDITIONAL FABRICATION RESTRICTIONS / WARNINGS:
   3.01 Application of heat for curving and straightening applications, camber and sweep adjustment, or other reason heating is limited to 1100º F/590 ºC maximum, and must be done by procedures approved by the director or his authorized representative.

   3.02 The matching submerged arc welding consumables ESAB ENI4 electrode in combination with Lincoln MIL800H, recommended in appendix A of the AASHTO guide for highway bridge fabrication with HPS70W steel, has produced weldment containing unacceptable discontinuities in a substantial number of complete penetration groove welds in one structure, based on the parameters used and experience of one fabricator. Extreme caution should be exercised when using this electrode/flux combination.

   3.03 Consideration will be given to other welding processes if a written request is submitted to the Office of Materials Management in accordance with CMS
108.05. Other welding processes must be qualified and tested as required by the referenced specifications and these notes.

3.04 In addition to the requirements of ANSI/AASHTO/AWS D1.5 section 5.17. All procedure qualification tests must be ultrasonically tested in conformance with the requirements of AWS D1.5, section 6, part c. Evaluation must be in accordance with AWS D1.5, Table 6.3, ultrasonic acceptance – rejection criteria – tensile stress. Indications found at the interface of the backing bar may be disregarded, regardless of the defect rating.

3.05 Whenever magnetic particle testing is done, only the yoke technique will be allowed, as described in section 6.7.6.2 of the ANSI/AASHTO/ AWS D1.5 Bridge Welding Code, modified to test using alternating current only. The prod technique will not be allowed.

4. BASIS OF PAYMENT:
Payment for the above completed and accepted quantities will be made at the contract bid price for:

<table>
<thead>
<tr>
<th>Item</th>
<th>Ext</th>
<th>Units</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>513</td>
<td>10151</td>
<td>LUMP</td>
<td>Structural Steel Members, Hybrid Girder, Level Six Fabrication, As Per Plan:</td>
</tr>
<tr>
<td>513</td>
<td>10401</td>
<td>POUND</td>
<td>Structural Steel Members, Hybrid Girder, Level Six Fabrication, As Per Plan:</td>
</tr>
</tbody>
</table>

In addition to the above General Plan note, include the following provisions:

1. Include the following note on the detail plan sheet showing girder elevations:

Where a Shape or plate is designated (CVN) the material shall meet requirements as specified in the general notes on sheet __/__.

2. Provide a note on the detail plan sheets showing girder elevations indicating where undermatched fillet welding is acceptable. See section 3.3 of Guide Specification for Highway Bridge Fabrication with HPS70W Steel.
ARN-1 CORRUGATED STEEL BRIDGE FLOORING

Retired proposal note. Not recommended for use on any state project. Has been used for county, township or temporary bridge decking.

ITEM SPECIAL - CORRUGATED STEEL BRIDGE FLOORING

1.0 DESCRIPTION

This item shall consist of furnishing and installing corrugated galvanized steel bridge flooring of the thickness and to the dimensions shown on the plans.

2.0 MATERIALS

The steel shall conform to ASTM A527/A527M, Grade D, Yield Stress = 40,000 psi [280 MPa], Tensile Stress = 55,000 psi [386 MPa], or an approved equivalent. The carbon content shall not exceed 0.3%.

3.0 FABRICATION

The steel sheets shall be fabricated into corrugated plates with a minimum width of 9 inches [225 mm] and of a length as indicated on the plans. The corrugations shall be not less than 3 inches [75 mm] deep and spaced not more than 9 inches [225 mm] center to center. The fabricated plates shall have a minimum section modulus equal to the following:

<table>
<thead>
<tr>
<th>THICKNESS</th>
<th>SECTION MODULUS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.179&quot; [4.554 mm]</td>
<td>0.217</td>
</tr>
<tr>
<td>0.209&quot;[5.314 mm]</td>
<td>0.251</td>
</tr>
<tr>
<td>0.251&quot; [6.373 mm]</td>
<td>0.281</td>
</tr>
<tr>
<td>0.3125&quot; [7.9375 mm]</td>
<td>0.350</td>
</tr>
<tr>
<td>0.375&quot; [9.525 mm]</td>
<td>0.403</td>
</tr>
</tbody>
</table>

The plates may be fabricated from two or more sheets buff-spliced end to end, with 12 inches [300 mm] minimum joint stagger, by a continuous shop butt weld on both sides of the plate. When splices occur over stringers the joints need not be staggered and the welds that bear on the stringers shall be ground flush.

Holes shall be provided in every corrugation valley on both sides of the stringer for attaching the plates to the stringer by a 3/4" bolt [19 mm] and approved clip at 9" [225 mm] centers on alternate sides of the stringer. Holes shall be provided along both 9" [225 mm] edges of the plate at the midpoint between stringers for attaching the plates together with a 3/4" [19 mm] bolt. Holes shall be provided in each end of the plate at the center for attaching a L2 x 2 x 1/4" [L50 x 50 x 6 mm] with a 3/4" [19 mm] bolt to retain a 3-1/2" [90 mm] x 0.179"[4.554 mm] [7 gauge] end dam.
4.0 GALVANIZING

The plates, hardware and accessories shall be galvanized after fabrication in accordance with ASTM A 123 or ASTM A 153. Galvanizing damage during erection shall be repaired in a manner acceptable to the Engineer.

5.0 METHOD OF MEASUREMENT

The quality of flooring paid for shall be the actual square feet [square meter] of flooring completed in place and accepted, including all necessary labor, equipment, hardware and incidentals required to complete the item.

6.0 BASIS OF PAYMENT

Payment shall be made at contract price for:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNIT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Square Feet [Square Meter]</td>
<td>Corrugated Steel Bridge Flooring</td>
</tr>
</tbody>
</table>
ARN-2  CLASS S CONCRETE USING SHRINKAGE COMPENSATING CEMENT

Originally a proposal note for use of shrinkage compensating cement in lieu of Type 1 cement to help eliminate drying shrinkage cracking of bridge decks. FHWA will not participate in the use of this special concrete. Field investigation found higher permeability and more variable permeability in bridge decks with type K cement concrete compared to type 1 cement concretes.

ITEM 842 - CLASS S CONCRETE, USING SHRINKAGE COMPENSATING CEMENT, AS PER PLAN

1.0 DESCRIPTION

This item shall consist of furnishing and placing Portland cement concrete using shrinkage compensating cement for the bridge deck and all parapets, in accordance with these Specifications, and in reasonably close conformity with the lines, grades, and dimensions shown on the Plans. All applicable provisions of SS 842 shall apply except as modified herein.

2.0 MATERIALS

The cement shall be expansive hydraulic cement conforming to 701.08.

Admixtures used in the concrete mixture must be compatible and shall be dispensed in accordance with the manufacturer's recommendations. Compatibility statement must be in writing from admixture supplier prior to the concrete placement. Admixtures shall be obtained from only one source for each specific bridge deck.

Coarse aggregate stockpiles shall be saturated. Saturation shall be completed a minimum of 24 hours prior to use; however, the application of water by sprinkling shall continue as directed by the Engineer.

3.0 PROPORTIONS

The maximum water/cement ratio given for Class S concrete in SS 899.03 shall not be changed from 0.44. The Engineer shall make appropriate adjustments in the aggregate batch weights to maintain the yield in accordance with SS 899.03.

SS 899.031 Proportioning Options, shall not apply to this item.

4.0 SLUMP

Slump at the time and point of concrete placement shall be 3 to 6" [75 TO 150mm], except for concrete used to slipform bridge medians and parapets. The use of superplastizers may be required to achieve a placeable concrete within the specified slump range and to meet the maximum water/cement ratio.
5.0  ENTRAINED AIR

Concrete shall contain 6 plus or minus 2 percent of entrained air at the time and place of concrete placement.

6.0  MIXING CONCRETE

The last sentence of the third paragraph of 899.06. Mixing Concrete, shall be revised to read:

If an approved set-retarding (705.12, Type B) or a water-reducing and set-retarding (705.12, Type D) admixture is used, discharge shall be completed within 75 minutes after the combining of the water and the cement.

7.0  CONCRETE DELIVERY

When supplying concrete using Type K shrinkage-compensating cement, ready-mix plants identified as "Truck mix" shall proportion the concrete such that the volume placed into the truck is no more than 3/4 of its rated capacity, unless a larger size is approved by the Engineer. The cost for complying with the requirement shall be included in the appropriate concrete bid item.

8.0  PLACING CONCRETE

Maximum ambient temperature at the time of placement of concrete shall be 80F [27C].

The deck formwork beam flanges and reinforcing shall be thoroughly sprinkled with water prior to placement of the concrete. Sprinkled areas shall remain damp until placement of concrete, however, no excess standing water will be allowed.

Concrete shall be placed when the rate of evaporation is less than 0.2 lb./sq. ft./hour [1 kg/sq.m/hour)(see chart in SS 842). The Contractor shall determine and document the atmospheric conditions, subject to verification by the Engineer. Atmospheric conditions shall be monitored throughout the pour. If the evaporation rate is exceeded the pour shall be stopped.

9.0  SLIPFORMING OF PARAPETS AND MEDIANS

The Contractor may elect to Slipform the bridge medians or parapets. The concrete to be slipformed shall meet the requirements of this note and the additional requirements for slipforming in SS 842.

10.0  CURING

As soon as all finishing operations have been completed the finished surface shall be covered with a single layer of clean wet burlap. The fresh concrete surface shall receive a wet burlap cure for 7 days. For the entire curing period, the burlap shall be kept wet by the continuous application of water through soaker hoses. Either a 4 mil [10 micrometer] white opaque
polyethylene film or a wet burlap-white opaque polyethylene sheet shall be used to cover the wet burlap for the entire curing period. Storage tanks for curing water shall be on-site and filled before a pour will be permitted to start. Storage tanks shall remain on-site throughout the entire cure period. They shall be replenished, as required, with a shuttle tanker truck or a local water source such as a fire hydrant.

### 11.0 SURFACE FINISH

All exposed surfaces of the parapet and vertical faces of deck edges shall have a rubbed finish in accordance with SS 842.15. Defects shall be corrected prior to rubbing with a Type K cement mortar of the same proportions as the concrete.

### 12.0 METHOD OF MEASUREMENT

The quantity shall be measured as per SS 842 and shall include all labor, materials, equipment and incidentals necessary to complete this item of work.

### 13.0 BASIS OF PAYMENT

The payment will be made at the contract unit price bid for the following:

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>842</td>
<td>Cubic Yd</td>
<td>Class S Concrete, Superstructure, Using Shrinkage</td>
</tr>
<tr>
<td></td>
<td>[Cubic Meter]</td>
<td>Compensating Cement, As Per Plan</td>
</tr>
</tbody>
</table>
ARN-3  RETIRED NOTE 13

[13] ITEM SPECIAL, SEALING OF CONCRETE SURFACES: A concrete sealer shall be applied to the concrete surfaces shown on sheet(s) XX. See proposal for surface preparation requirements, applications rates, material requirements and application procedures.


ARN-4  RETIRED NOTE 32

The following note shall be used where a minimum pile wall thickness is required, see Section 2.4.2.3. The monotube wall thickness to be provided must provide comparable strength to that provided by the pipe pile. Note that the steel for pipe piles is Grade 36 and the monotube piles are Grade 50.

[32] ITEM 507_______ PILES, AS PER PLAN PILE WALL THICKNESS: The responsibility of selecting a satisfactory pile wall thickness for this project shall be borne by the Contractor except that the pile wall thickness shall not be less than_____ inch [mm]. If a pile wall thickness greater than _____ inches [mm] is necessary to resist the pile installation driving stresses, the Contractor shall make this determination and shall furnish a pile with an greater wall thickness. If monotube piles are used, the minimum wall thickness shall be ___ inch [mm].

HISTORY: The 1997 CMS specifications established minimum pile wall thickness by formula thereby eliminating the need for this note. Note [32] was retired by the April 2000 Edition of the Bridge Design Manual.

ARN-5  RETIRED NOTE 33

The following note shall be used where a minimum pile hammer size is required, see Section 2.4.2.4. This note is not required when piles are driven to refusal on bedrock and the estimated pile length is less than 50 feet.

[33] ITEM 507_______ PILES, AS PER PLAN PILE HAMMER: The pile hammer used to install the_____ piles shall have a State's Energy Rating of not less than _____ foot-pounds [joules]. This requirement does not relieve the Contractor from 108.05 which states that the Contractor is to provide sufficient equipment for prosecuting the required work. Refer to "ODOT"s Manual of Procedures for Structures" to obtain the State's Energy Rating.

HISTORY: Item 507.04 of the 1997 CMS specifications established a required hammer rating based on Ultimate design load. Note [33] was retired by the April 2000 Edition of the Bridge Design Manual.
Design Manual.

**ARN-6 RETIRED NOTE 40A**

Supplemental specification 863 covers requirements for steel structures used as highway bridges. As railroads have special requirements for their bridges, SS 863 requires as per plan modifications. The following note should be included for all railroad bridges. The bid item shall also be modified to match the plan note.

[40A] 863 STRUCTURAL STEEL MEMBERS, FRACTURE CRITICAL, LEVEL (6), AS PER PLAN

The requirements of Supplemental Specification 863 shall apply except as modified below:

In addition to the requirements of 863.08, Shop Drawing and Submittal Process, railroads require approval of shop drawings prior to fabrication. The Contractor shall submit three (3) sets of the prepared shop drawings to the railroad(s) listed below for review and approval. Any railroad comments shall be resolved prior to the Contractor’s final acceptance per 863.08. If the Contractor judges the railroad’s comments as outside the Contract, resolution shall be completed before the Contractor makes the acceptance submission, 863.08, to the Office of Structural Engineering. The acceptance submission shall include one set of each railroad’s accepted shop drawings; copies of any documentation between the railroad and the Contractor; and one additional set of the Contractor’s accepted shop drawings for each railroad.

Shapes or plates designated (FCM) shall be subject to the Fracture Control Plan specified by the AASHTO/AWS D1.5, Bridge Welding Code section 12.

Any riveted construction shall be in conformance with the requirements of the American Railway Engineering and Maintenance of Way Association (AREMA) "Manual for Railway Engineering" Chapter 15, Steel Structures, section 3.2.1 Rivets and Riveting.

The Contractor shall make submittals to the following Railroad(s):

* *

* *

* *

* *

* * The designer shall complete the note to include the railroad’s name, address,
responsible person for shop drawing approval and phone number.

**HISTORY:** Note [40A] was retired by the October 16, 2002 Edition of the Bridge Design Manual. In order to cover both structural steel and prestressed concrete members, a note was added to the Boiler Plate proposal note requiring the railroad’s review and approval of shop drawings prior to the Contractor’s acceptance.

**ARN-7 RETIRED NOTE 44A**

[44A] ITEM 863 STRUCTURAL STEEL MEMBERS MISCELLANEOUS FABRICATION, AS PER PLAN, (OTHER DESCRIPTION). All sections of SS 863 apply except as revised herein. The Engineer is responsible for ensuring any shop or field fabricated steel supplied under this bid item is acceptable. The requirements for submittal of shop drawings to The Office of Structural Engineering is waived. At the Engineer’s option, the Contractor shall either supply the Engineer with shop drawings, required in section 863.08, prior to any incorporation of shop fabricated steel at the project, or supply the Engineer with “as fabricated” drawings, meeting 863.08, after completion of field fabrication. The Engineer shall assure the submitted drawings match the final as built steel incorporated into the work. If the Engineer is satisfied with the drawings and the delivered materials, the Contractor shall supply a copy set, stamped and dated as per 863.08, to the Office of Structural Engineering for record purposes. Submittal requirements under 863.09, Materials, shall be made to the Engineer. The Contractor shall furnish a copy of the written letter of acceptance, 863.09, to the Office of Structural Engineering written letter of acceptance.

The Engineer, at or before the pre-construction meeting, may choose to request assistance, as required, from the Office of Structural Engineering.

Steel members included in this item include _____, _____, and _____.

**HISTORY:** Note [44A] was retired by the October 18, 2002 Edition of the Bridge Design Manual because the 2002 CMS incorporated the SS863 pay items into Item 513.

**ARN-8 RETIRED NOTE 49**

The below note should only be used where Dowel holes are to be drilled and there is no new concrete item SS 842 for which the dowel holes would be incorporated.

[49] ITEM SPECIAL DOWEL HOLES

This item shall include the drilling or forming of holes into concrete or masonry and the furnishing and placing of grout into holes. Non-shrink epoxy grout shall be used in accordance with CMS 510 and CMS 705.20. Depth of holes shall be as shown in plans.
Payment for drilling or forming holes and furnishing and placing materials shall be included in the contract prices for Item Special - Dowel Holes

**HISTORY:** Note [49] was retired by the October 18, 2002 Edition of the Bridge Design Manual because the 2002 CMS reinstated pay items for Dowel Holes.

**ARN-9 RETIRED NOTE 50**

The following note shall be included in the General Notes of the roadway plans for ITEM 611 Reinforced Concrete Approach Slab (T=___”), As Per Plan:

[50] ITEM 611 REINFORCED CONCRETE APPROACH SLAB(T=___mm), AS PER PLAN: The reinforcing steel for the approach slabs of this structure shall be epoxy coated in conformance with 509.

** Two separate thicknesses of clear or opaque polyethylene film, 705.06, shall be placed on the prepared subbase and where the approach slab is to be constructed. The polyethylene films shall completely cover the full length and width of the subbase between the sidewall forms for the approach slab.

Materials, labor and installation shall be included for with approach slabs for payment.

** This paragraph only required if abutment designs are semi-integral or integral type.

**HISTORY:** Note [50] was retired by the April 2000 Edition of the Bridge Design Manual for two reasons. First, the 1997 CMS incorporated the use of epoxy coated reinforcing steel and second, the use of polyethylene film was not considered to be essential.

**ARN-10 RETIRED NOTE 50A**

The following note should be added for approach slabs to change the concrete from CLASS C to match the superstructure concrete of the bridge. The designer shall specify the concrete to match the deck superstructure concrete being used. Either SS 842 Class S or SS 844 High Performance Concrete. This note should only be used if the approach slab standard drawing being used does not incorporate this modification.

[50A] ITEM 611 REINFORCED CONCRETE APPROACH SLAB, T = ??, AS PER PLAN Concrete for this item shall be CLASS S, SS 899 or SS 844, High Performance Concrete, Mix 3 or 4.

* The designer shall specify the concrete being used in the deck. If no deck concrete is being placed the required concrete shall be CLASS S, SS 899.
HISTORY: Note [50A] was retired by the October 18, 2002 Edition of the Bridge Design Manual because the 2002 CMS moved the pay item for Approach Slabs to Item 526 and incorporated the requirement for the concrete to match the superstructure.

ARN-11 RETIRED NOTE 52

The following notes shall be included in all bridge plans where a pipe drainage system behind the abutment backwall is being installed.

For perforated corrugated plastic pipe:

[52] ITEM 518, 150 mm PERFORATED CORRUGATED PLASTIC PIPE, AS PER PLAN: Corrugated pipe used in abutment drainage shall be 150 mm diameter plastic corrugated as per 707.33

HISTORY: Note [52] was retired by the April 2000 Edition of the Bridge Design Manual because the 1997 CMS incorporated the plastic pipe requirements above.

ARN-12 RETIRED NOTE 53

For non-perforated corrugated plastic pipe:

[53] ITEM 518, 150 mm NON-PERFORATED CORRUGATED PLASTIC PIPE, INCLUDING SPECIALS, AS PER PLAN: Corrugated pipe used in abutment drainage shall be 150 mm diameter, plastic corrugated as per 707.33 This item shall include all elbows, tees and end caps required to complete the abutment drainage system.

HISTORY: Note [53] was retired by the April 2000 Edition of the Bridge Design Manual because the 1997 CMS incorporated the plastic pipe requirements above.

ARN-13 RETIRED NOTE 67

For all painted steel structures that require a paint system other than 514 System A or SS 816, IZEU, the following note should be placed on the structural steel detail sheet:

[67] HIGH STRENGTH BOLTS shall be ___________ diameter A325/A325M, galvanized, unless otherwise noted. If A-588 unpainted steel is used the following note should be placed on the structural steel detail sheet:

HISTORY: Note [67] was retired by the April 2000 Edition of the Bridge Design Manual because the above requirements for galvanizing were incorporated into the materials section of 1995 CMS, 711.09.
ARN-14 RETIRED NOTE 68

The following note shall be provided if reference is made to Standard Drawing SD-1-69 for scupper details:

[68] SCUPPERS shall be in accordance with Standard Drawing SD-1-69 except that scupper pipes shall extend 200 mm below the bottom of the beams instead of 50 mm.

HISTORY: Note [68] was retired by the April 2000 Edition of the Bridge Design Manual because a new scupper design was provided on Standard Bridge Drawing GSD-1-96 and SD-1-69 was retired.

ARN-15 RETIRED NOTE 73

For a steel plate girder bridge with a concrete deck the distance from the top of deck slab to bottom of top flange shall be made a constant (required slab thickness and haunch plus the thickness of the thickest flange plate) for the full length of the girder, except as may be modified for a curved deck on straight girders. This dimension shall be shown with an asterisk * and the following note provided:

[73] *DECK SLAB DEPTH: The distance shown from the top of the deck slab to the bottom of the top flange is the theoretical design dimension including the design haunch thickness of ____ mm. The quantity of deck concrete to be paid for shall be based upon this dimension, minus the design haunch thickness, even though deviation from it may be necessary because the top flange of the girder may not have the exact camber or conformation required to place it parallel to the finished grade. Deduction shall be made for volume of encased steel plates as per 511.18.

HISTORY: Note [73] was originally retired by the April 2000 Edition of the Bridge Design Manual because, at that time, the haunch concrete was deemed to be incidental to the deck concrete. However, this became in conflict with the Item 511 Method of Measurement and caused some confusion during construction. The October 18, 2002 Edition of the Bridge Design Manual featured a revised note [72] for both rolled beams and girders to pay for the haunch concrete, and note [73] was again no longer necessary.

ARN-16 RETIRED NOTE 74

For a steel beam or plate girder bridge with concrete deck, the deck concrete shall be haunched down to the bottom of the top flange and the following note provided. For vertical sided haunches this note does not apply.

[74] A HAUNCH WIDTH of 9 inches [225 mm] shall be used. However, the haunch width may vary between 6 inches [150 mm] and 12 inches [300 mm].
For prestressed I-beam members a vertical haunch is used and no note is required.

**HISTORY:** Note [74] was retired by the October 18, 2002 Edition of the Bridge Design Manual because the content of the original note was incorporated into the revision of note [72].

---

**ARN-17 RETIRED NOTE 78**

The following note should be provided for bridge superstructures using A588 steel which is to be left unpainted:

[78] A588 STEEL is to be left unpainted. The outside surfaces and bottom surfaces of the bottom flanges of fascia beams shall be abrasively blast cleaned to grade Sa2 in the fabrication shop. See CMS 513.221 for final field cleaning requirements. Payment shall be included in item 513.

**HISTORY:** Note [78] was retired by the April 2000 Edition of the Bridge Design Manual because the shop cleaning requirements for A588 fascia beams were revised in SS 863.30.

---

**ARN-18 RETIRED NOTE 84**

Use the following note for non-composite box beams. A detail will be required in the plans showing the drip strip.

[84] STAINLESS STEEL DRIP STRIP: Prior to applying waterproofing, a bent drip strip shall be installed along the edges of the deck. An additional 1'-0" long drip strip shall also be installed centered on each post.

The strips shall be fastened at 1'-6" C/C maximum with 1 1/4" x 3/32" galvanized or stainless button head spikes (length x shank diameter) or No. 10 galvanized screws and expansion anchors, subject to approval of the Engineer. The strips shall be placed the full length of the deck, ending at the abutments. The strips shall be 8" wide x 0.029" thick. Where splices are required the individual pieces shall be butted together. Stainless steel shall be ASTM A167, Type 304, mill finish.

The final pay quantity shall be the actual overall length of the drip strip. Additional strips at posts shall not be measured for payment.

Payment shall be at the contract price bid for Item Special, linear feet, Steel Drip Strip, which shall include all materials, labor, tools, and incidentals necessary to complete the item.

**HISTORY:** Note [84] was retired by the April 2000 Edition of the Bridge Design Manual
because of the availability of Standard Bridge Drawing DS-1-92.

**ARN-19 RETIRED NOTE 85**

Use the following note for composite deck prestressed box beams, prestressed I-beams, concrete deck on steel stringers and reinforced concrete slab with over the side drainage. A detail will be required in the plans showing the drip strip.

[85] STAINLESS STEEL DRIP STRIP: Prior to the concrete deck placement a bent drip strip shall be installed along the edges of the deck by anchoring to the top layer of reinforcing steel and being butted, with a 90 degree bend, against the formwork. An additional 1'-0" long drip strip shall also be installed centered on each post.

The strips shall be placed the full length of the deck, ending at the abutments. Where splices are required the individual pieces shall be butted together. Stainless steel shall be 22 gauge ASTM A167, Type 304, mill finish.

The final pay quantity shall be the actual overall length of the drip strip. Additional strips at posts shall not be measured for payment.

Payment shall be at the contract price bid for Item Special, linear feet, Steel Drip Strip, which shall include all materials, labor, tools, and incidentals necessary to complete the item.

**HISTORY:** Note [85] was retired by the April 2000 Edition of the Bridge Design Manual because of the availability of Standard Bridge Drawing DS-1-92.

**ARN-20 RETIRED NOTE 86**

On rehabilitation plans where new reinforcing steel may require field bending and cutting, the following note can be used. Generally, all reinforcing steel shall be dimensioned to fit and shown on the plans.

[86] REINFORCING STEEL: New reinforcing steel may require field cutting or bending to be properly fitted. Payment shall be included in the applicable concrete item.

**HISTORY:** Note [86] was retired by the October 18, 2002 Edition of the Bridge Design Manual because a new general note [43a] was added that makes field cutting or bending an “As Per Plan” pay item.
ARN-21    RETIRED NOTE 28

Provide the following note when there is a pay item for Item 503:

[28] ITEM 503, UNCLASSIFIED EXCAVATION ***, AS PER PLAN: The backfill material behind the abutments shall be Type B granular material, 703.16.C, placed and compacted in 6 inch [150 mm] lifts.

*** Use of excavation 503 items as defined above.

HISTORY: Note [28] was retired by the January 2005 revisions to the Bridge Design Manual because the 2005 CMS revised 503.08 to include the material previously defined by note [28].
APPENDIX – 109.2

ARN-22  6 mm (1/4") EPOXY WATERPROOFING OVERLAY FOR BRIDGE DECKS

This proposal note was retired in April 2005.

6mm (1/4 INCH) EPOXY WATERPROOFING OVERLAY FOR BRIDGE DECKS - 04/19/02

GENERAL

This specification describes a two component 100% solid, flexible epoxy system and a special aggregate designed to provide a minimum 6mm (1/4") thick bridge deck overlay for the purpose of providing a waterproofing system which remains flexible at all operating temperatures and a non-skid surface.

MATERIALS

Formulation

The material shall be a two component epoxy or an epoxy derived co-polymer system consisting of simple volumetric mixing ratios such as one to one or two to one. For the remainder of this specification, the material systems shall be referred to as "epoxy".

The epoxy system shall be formulated to provide flexibility in the system without any sacrifice of the hardness, chemical resistance or strength of the epoxy system.

Properties

Adhesion to Concrete: When the finished system (including aggregate) has been applied as per manufacturers recommendation and tested according to ACI Method 503R-30, it shall have 100% failure in concrete. The prepared specimens, (minimum of 3) shall be conditioned for 7 days at 24 +/- 1 °C (75 +/- 2 °F) prior to testing.

Hardness: The epoxy material, when tested according to ASTM D 2240, shall have a Shore D Hardness between 50 to 75. Three samples shall be prepared on a structurally sound surface with a minimum thickness of 3 mm (0.12 inch) and allowed to cure for 7 days at 24 +/- 1 °C (75 +/- 2 °F) prior to performing the indicated tests.

Abrasion Resistance: Abrasion resistance shall be evaluated on Tabor abrader with a 1000 gram load and CS-17 wheel. Duration of the test shall be 1000 cycles. The wear index shall be calculated based on ASTM C 501 and wear index of the catalyzed materials shall be under 80. The test shall be run on cured samples of material which shall be applied at a film thickness of 15 to 20 mil to a stainless steel (316) plate. The film shall be allowed to cure for 7 days at 24 +/- 1 °C (75 +/- 2 °F) prior to performing the indicated test.

Tensile Strength: When tested according to ASTM D 638, the epoxy material shall have a tensile strength not less than 17.2 MPa (2500 psi). The Type IV, semi-rigid specimens, shall be cast in the specified mold and pulled at a rate of 5mm (0.20 inches) per minute by a suitable dynamic testing machine. The samples (minimum of 3) shall be allowed to cure at room temperature for at least 7 days at 24 +/- 1 °C (75 +/- 2 °F) prior to performing the indicated test.

Tensile Elongation: The elongation produced at the break in the tensile strength test must be a minimum of 30 percent.
**Compressive Strength:** When tested according to ASTM C 109, the cured epoxy material (including aggregate) shall have a compressive strength not less than 5000 psi. Three samples shall be cast and tested using the specified size and amount of aggregate. The samples shall be conditioned for 7 days at 24 +/- 1 °C (75 +/- 2 °F) before performing the indicated test. The rate of compression of these samples shall be no more than 12.7mm (0.5 inches) per minute.

**Water Absorption:** When tested as per ASTM D 570 the cured epoxy system shall not exceed the water absorption of 0.5 percent. Sample specimens shall be prepared according to section 4.1 and allowed to cure at 23 +/- 2 °C (73.4 +/- 3.6 °F) and 50 +/- 5% relative humidity. Tests are then to be carried out as per section 6.1.

**Aggregate:**

Aggregate for all layers shall be bauxite, crushed granite, aluminum oxide, flint, Washington Steilacoom, Tuff-Grane Type A (furnished by Emeri-Crete, Inc., Portsmouth, N.H.) or other aggregate as recommended by the Manufacturer of the Overlay System, provided it has MOH scale hardness of 7 or more and meets the following gradation:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>% Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 6</td>
<td>95-100</td>
</tr>
<tr>
<td>No. 10</td>
<td>10-35</td>
</tr>
<tr>
<td>No. 20</td>
<td>0-3</td>
</tr>
</tbody>
</table>

**Repair of Spalled Areas:**

Deck patching shall be performed as outlined in Proposal Note entitled "Patching Concrete Bridge Decks".

The patching material shall be a quick setting cementitious mortar meeting the requirements of 705.21, Type I (Magnesium Phosphate Based Materials will not be allowed).

All patches shall be allowed to cure a minimum of 14 days before the overlay is placed. Traffic shall be allowed to use the bridge during the curing period of the patches.

The bridge shall be open to traffic during all non-working hours, therefore, patching must be scheduled and performed accordingly. Exceptions to this would be other construction activities on the approach roadway which prevent opening the road to traffic.

**Preparation:**

After all patches have cured for a minimum of 14 days, the entire deck shall be cleaned by shotblasting to remove any oil, dirt, rubber or any other potentially detrimental material such as curing compound and laitances which would prevent bonding and curing of the epoxy material.

Only areas that the shotblasting equipment cannot reach (i.e., along curbs and median walls) will the use of sandblasting be permitted to an extent satisfactory to the epoxy manufacturer and Engineer. This should be performed prior to the shotblasting whenever applicable and practical. Abrasives containing more than 1% free silica will not be allowed (No silica sand). If it rains on the prepared surface, the surface shall be reblasted.

Traffic shall not be allowed on any portion of the deck which has been shotblasted or on areas where all coats have not already been placed. The overlay application equipment, however, will be allowed to drive on the deck surface during application, provided planking or similar
precaution is taken to insure that the deck surface will not become contaminated or otherwise damaged.

**Test Patch:**

Prior to placing the first course, the Contractor shall use the test method prescribed in ACI 503R - Appendix A of the ACI Manual of Concrete Practice (modified as per Virginia Transportation Research Council, Michael M. Sprinkel, 804-293-1941) to determine the cleaning practice (size of shot, flow of shot, forward speed of shotblast machine, and number of passes) necessary to provide a tensile bond strength greater than or equal to 1.7 MPa (250 psi) or a failure area, at a depth of 6mm (1/4 inch) or more into the base concrete, greater than 50% of the test area. A test result shall be the average of three tests on a test patch of approximately 0.3m x 0.9m (1 ft x 3 ft), consisting of two courses.

One test result must be obtained from each span or 167 m² (200 square yards), whichever is the smaller area. The Engineer will designate the location of these test patches. To provide assurance that the cleaning procedure, materials, installation procedure, and curing period will provide the desired overlay, test patches shall be installed with the same materials, equipment, personnel, timing, sequence of operations, and curing period prior to opening to traffic, that will be used for the installation of the overlay. The cleaning practice, materials and installation procedure will be approved if one passing test result is obtained from each test area.

If the cleaning practice, materials and installation procedure are not acceptable, the Contractor must remove failed test patches and make the necessary adjustments and re-test all areas at no additional cost to the Department until satisfactory test results are obtained.

**Application of Overlay:**

All surfaces to be overlaid shall be dry at the time of application. Immediately before applying the epoxy material, all prepared surfaces shall be cleaned with compressed air to remove dust and debris.

The application of the epoxy system shall not be made when it has rained or snowed within 24 hours prior to application and rain or snow is forecast within 8 hours after application. If rain occurs during the application, all operations shall cease and the surfaces allowed to dry, before continuing.

The minimum temperature at which the epoxy can be applied is to be 10 °C (50 °F) and rising.

The epoxy manufacturer shall have a representative on the job site at all times who, upon consultation with the Engineer, may suspend any item of work that is suspect and does not meet the requirements of this specification. Resumption of work will occur only after the epoxy manufacturer's representative and the Engineer are satisfied that appropriate remedial action has been taken by the Contractor.

The overlay system shall be applied on all deck areas using metering, mixing and distribution machinery approved by the epoxy manufacturer. The application machine shall feature positive displacement volumetric metering pumps. The resin shall be stored in temperature controlled reservoirs capable of maintaining 38 °C (100 °F) plus/minus 4 °C (10 °F) to insure optimum mixing. Ratio check verification at the pump outlets as well as cycle counting capabilities to monitor output will be standard features.
The number of layers (minimum of two) and the application rates of the epoxy in the various layers shall be as recommended by the manufacturer in order to achieve a minimum overlay thickness of 6mm (1/4 inches) measured from the highest point on the deck surface to the top of epoxy (not the peaks of the aggregate).

Application of the Epoxy: After mixing of the components, the epoxy shall be evenly distributed on the clean, dry deck surface at the rate as recommended by the manufacturer.

Application of Aggregate: After the application of the epoxy, broadcasting of the aggregate on decks shall be by automated dispenser depositing the aggregate onto the deck in a uniform manner as directed by the epoxy manufacturer.

The aggregate shall be broadcast at a rate in excess of that needed to cover the surface so that no wet spots appear. The aggregate must be dropped vertically in such a manner that the level of the epoxy will not be unduly disturbed.

When working within the confines of one lane, (i.e. aggregate truck driving over freshly placed epoxy and aggregate) planking or 3/4" plywood shall be placed under the wheels of the truck or any other equipment to distribute the load and prevent displacement of the fresh epoxy.

Consolidation: (If required by manufacturer) A hand operated roller as approved by the epoxy manufacturer and the Engineer shall be used at surface temperature below 16 ºC (60 ºF) within ten minutes after the application of the aggregate to evenly consolidate the aggregate into the epoxy.

Removal of Excess Aggregate: After the overlay has hardened, removal of all loose and excess aggregate with a power vacuum or other method shall be made prior to the application of subsequent coats.

Application of Additional Layers: May be made immediately after the preceding layer has completely hardened and all excess aggregate has been removed. The time between each coat will vary depending on the temperature and circumstances of the project.

Joints in the Overlay (i.e., between two adjacent lanes) shall be staggered and overlapped 76mm (3 inch) minimum between successive layers so that no ridges appear.

Traffic shall be allowed on the final layer after the system has cured (as determined by the manufacturer) and after removal of all excess, loose aggregate, unless other construction activities prevent the re-opening to traffic.

STORAGE AND HANDLING:

Epoxy Material: Epoxy material shall be transported to the job site in their original containers inside a dry, temperature controlled facility maintained at a minimum temperature 16 ºC (60 ºF) and not to exceed 49 ºC (120 ºF). The containers shall be identified as "Part A - Contains Resin" and "Part B - Contains Curing Agent" and shall be clearly marked with the name and address of the manufacturer, name of the product, mixing proportions and instruction, lot and batch numbers, date of manufacture, and quantity contained therein. Material Safety Data Sheets shall accompany each shipment and the driver must have a ready access to them.

Job Site Storage: The epoxy material shall be stored on the job site in a dry, temperature controlled facility within the temperature range of 16 ºC (60 ºF) to 49 ºC (120 ºF). If the epoxy material is transported or stored on the job in the application machine tank, the material must also be maintained within the above temperature range.
Handling of Epoxy on the Job: Protective gloves, clothing, boots, and goggles shall be provided by the Contractor to workers and inspectors directly exposed to the epoxy material. Material Safety Data Sheets shall be provided to all workers and inspectors as obtained from the manufacturer. Disposal of all material containers shall be the Contractor's responsibility.

Aggregate: All aggregate shall be stored in dry, moisture free atmosphere. The aggregate shall be fully protected from any contaminants on the job site and shall be stored so as not to be exposed to rain or other moisture sources.

SAMPLING AND ACCEPTANCE:
Acceptance: The following are acceptable materials, pending submission of approved, certified independent laboratory results:

FLEXOGRID as manufactured by POLY-CARB, INC.
33095 Bainbridge Road, Solon, Ohio 44060.

FLEXOLITH, as manufactured by TAMMS INDUSTRIES, CO.
7405 Production Drive, Mentor, Ohio 44060.

SIKADUR EPOXY BROADCAST OVERLAY SYSTEM as manufactured by SIKAGET.
201 Polito Avenue, Lyndhurst, NJ 07071.

Upon approval by independent testing laboratory, other products will be considered as equal, providing they:
- meet or exceed the requirements of this specification
- are specifically marketed as bridge deck overlays
- have been successfully used on bridge decks in the continental United States or Canada for at least three years.

Certified Test Data: See 101.061

Laboratory Sample: At the preconstruction conference, the Contractor shall notify the Engineer of the source of material he expects to use. The material manufacturer shall furnish samples of epoxy material as may be required by the Engineer.

Thickness Verification: The Contractor shall establish that the overlay is at least 6mm (1/4 inch) thick (measured from the deck surface to the top of the epoxy) at three random locations selected by the Engineer for every 836 m2 (1,000 square yards) of deck surface. Thin areas shall be recoated as described above by the Contractor and retested at no additional cost to the State. This verification may consist of cores, holes, etc., but in all cases any tested areas shall be repaired by the Contractor.

Epoxy Overlay Guarantee
The Contractor and/or the epoxy Manufacturer shall furnish to the State a written (5) five year guarantee on the completed epoxy overlay. This guarantee shall commence on the date of acceptance by the State and shall cover faulty materials and/or workmanship. Failure is defined as any peeling, cracking or rupture of the overlay and any delamination or disbanding from the deck surface.
The State agrees to notify the Contractor/Manufacturer of the need for any repairs covered by the guarantee promptly upon discovery of same and said repairs shall be commenced within a reasonable period of time after receipt by Contractor/Manufacturer of said notice from the State, subject to delays by strikes, acts of God or other causes beyond the reasonable control of Contractor/Manufacturer.

Manufacturer's/Contractor's sole responsibility to the State of Ohio, Department of Transportation shall be to make the repairs referred to herein.

The guarantee shall be sent in triplicate by the Contractor/Manufacturer to the State for review and filing. Final payment will not be made until the warranty is received.

**BASIS OF PAYMENT**

Payment for completed and accepted quantities as measured above will be made at the contract price bid for:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNIT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Square meter (square yard)</td>
<td>Patching Concrete Bridge Decks</td>
</tr>
<tr>
<td>Special</td>
<td>Square meter (square yard)</td>
<td>Epoxy Waterproofing Overlay 6mm (1/4 inch thick)</td>
</tr>
<tr>
<td>Special</td>
<td>Lump</td>
<td>Test Patch</td>
</tr>
</tbody>
</table>
This proposal note was retired in April 2005.

CONCRETE REPAIR USING PREPLACED AGGREGATE CONCRETE - 04/19/02

Scope of Work:
The work covered by these specifications consists of furnishing all labor, materials and equipment for the removal of unsound concrete, the placing of coarse aggregate and the mixing and pumping of mortar into the voids of the preplaced aggregate for the purpose of making preplaced aggregate concrete.

General:
a. The removal of disintegrated concrete and preparation of the surface shall be as specified in 519.03 and 519.04.
b. Preplaced aggregate concrete shall be composed of graded coarse aggregate solidified with mortar.
c. Mortar shall consist of a mixture of Portland cement, pozzolan, fluidifier, sand and water so proportioned and mixed as to produce a grout capable of maintaining the solids in suspension without appreciable water gain, yet which may be pumped without difficulty and which will penetrate and fill completely the voids in the aggregate mass. Furthermore, these materials shall be so proportioned as to produce a hardened preplaced aggregate concrete of 31 MPa (4,500 psi) minimum in 28 days.
d. The proportioning and placing of the coarse aggregate and mortar shall be performed in accordance with provisions of these specifications. The Contractor shall submit for approval to the Engineer, a description of the type and proportioning of materials to be used; the method of procedure; furnish records of past experience in performing this type of work; and furnish records and data to prove conclusively that the resulting concrete will meet the quality and properties required by these specifications.
e. Two companies which specialize in this type of work are: Anderco, Inc., 7804 Hillside Road, Independence, Ohio 44131 and the Prepakt Concrete Co., The Superior Building, Cleveland, Ohio 44114.

Materials:
a. Portland Cement: Shall conform to 701.01, 701.04.
b. Pozzolan: Shall conform to the requirements of ASTM C 618, Class F.
c. Fluidifier: A water-reducing, set controlling agent which imparts to the mortar the properties of a colloidal suspension to prevent the constituents of the mortar from settling, thereby assuring complete bond on the undersides of the aggregate particles and reinforcement; it shall act as a protective colloid to inhibit the early stiffening of the mortar, thereby facilitating pumping of the mortar through supply lines and voids in the aggregate mass; it shall eliminate the setting shrinkage observed to take place in all straight Portland-cement grout; and it shall produce the effect of air-entraining agents with respect to freezing and thawing resistance. It shall be as
manufactured by Concrete Chemicals of Cleveland, Ohio or equal and shall conform to ASTM C 937.

d. Water: Water for intrusion mortar shall be fresh, clean, and free from injurious amounts of sewage, oil, acid, alkali, salts, or organic matter.

e. Fine Aggregate: Sand shall meet the requirements of ASTM C-33 and 703.03, except that the gradation shall be as hereinafter specified.

The sand shall be well graded from fine to coarse and the gradation shall conform to the following requirements as delivered to the grout mixers:

<table>
<thead>
<tr>
<th>Sieve Designation</th>
<th>Cumulative Passing</th>
<th>Percentages by Weight Retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.36 mm (8)</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>1.18 mm (16)</td>
<td>95-100</td>
<td>0-5</td>
</tr>
<tr>
<td>600 µm (30)</td>
<td>60-85</td>
<td>15-40</td>
</tr>
<tr>
<td>300 µm (50)</td>
<td>20-40</td>
<td>60-80</td>
</tr>
<tr>
<td>150 µm (100)</td>
<td>10-30</td>
<td>70-90</td>
</tr>
<tr>
<td>75 µm (200)</td>
<td>0-10</td>
<td>90-100</td>
</tr>
</tbody>
</table>

The sand shall have a fineness modulus of not less than 1.30 or more than 2.10. During normal operation, the grading of the sand shall be controlled so that the fineness modulus of a single sample taken at the mixer will not vary by more than 0.10 from the average fineness modulus. The fineness modulus is defined as the total divided by 100 of the percentages retained on sieve numbers 16, 30, 50 and 100.

f. Coarse Aggregate: Coarse aggregate shall meet the requirements ASTM C-33 and 703.02, except as to grading.

Coarse aggregate furnished shall be adequately washed and suitably clean for this type of work. It shall subsequently be drained until the residual free moisture is determined to be reasonably uniform and stable.

The coarse aggregate furnished shall be washed as outlined above immediately prior to placing in the forms and shall be in a suitably clean and washed condition at the time of placement in the forms. If necessary to stockpile, the coarse aggregate shall be stockpiled by approved means in such a manner as to prevent objectionable segregation of sizes.

The grading of the coarse aggregate shall be #4's (AASHTO M 43) or as follows:

<table>
<thead>
<tr>
<th>Sieve</th>
<th>Cumulative Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>63mm (1 ½&quot;)</td>
<td>90-100</td>
</tr>
<tr>
<td>50mm (1&quot;)</td>
<td>20-45</td>
</tr>
<tr>
<td>19mm (3/4&quot;)</td>
<td>0-10</td>
</tr>
<tr>
<td>12.5mm (½&quot;)</td>
<td>0-1</td>
</tr>
</tbody>
</table>

g. Variation from the above gradations may be permitted by the Engineer only on the basis of tests submitted by the Contractor which will prove conclusively that the final concrete will meet or exceed the strength and durability requirements of these specifications.

**Placing Coarse Aggregate:** The coarse aggregate shall be handled and deposited in the form in such a manner that the grading of the aggregate in place will be as uniform as practicable. The
aggregate shall be lightly vibrated, tamped or rodded in approximately 200mm (8 inch) lifts during placing operations to reduce the voids to an economic minimum. Care shall be taken that reinforcing steel and embedded items are not displaced from the locations as indicated on the drawings.

**Mixing and Pumping Mortar:** All mixing and pumping equipment used in the preparation and handling of mortar will be subject to approval by the Engineer. All oil or other rust inhibitors shall be removed from the mixing drums, stirring mechanisms, and other portions of the equipment in contact with the mortar before the mixers are used.

All materials shall be accurately measured by volume or weight as they are fed to the mixer. The order of placing the materials in the mixer shall be as follows: (1) water, (2) fluidifier, (3) other solids. The quantity of water used and the time of mixing shall be as to produce homogeneous mortar capable of being pumped without difficulty and which will penetrate and completely fill all voids within the preplaced aggregate. Time of mixing shall be not less than one minute. The maximum time between charging of the mixer and placing of the concrete will depend on the temperature of the grout as follows:

<table>
<thead>
<tr>
<th>Temperature Range</th>
<th>Minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below 15 ºC (60 ºF)</td>
<td>90</td>
</tr>
<tr>
<td>15 to 30 ºC (60 to 85 ºF)</td>
<td>60</td>
</tr>
<tr>
<td>30 ºC (85 ºF) and higher</td>
<td>45</td>
</tr>
</tbody>
</table>

If there is a lapse in the operation of mortar injection, the mortar shall be recirculated through the pump, or through the mixer drum or agitator and pump. A screen no larger than 6mm (1/4 - inch) mesh shall be used between the mixer and pump, or between the mixer and agitator to remove large particles which might clog the smaller voids in the aggregate mass.

The method of injecting the intrusion mortar into the aggregate shall be submitted to the Engineer/Director for approval. Injection shall start at the lowest point in the form and shall continue thereafter at a point always below the surface of previously injected mortar. All pumping shall be done slowly and at a uniform rate without interruption so that as R rises in the aggregate-filled form, the mortar completely fills all voids in the coarse aggregate. When the aggregate mass is totally enclosed, the pumping shall continue until all excess air and water have been expelled through vents or venting surfaces at the top of the forms.

**Forms:** Forms shall be of wood, steel, or other approved material. Absorptive form lining will not be permitted. Forms shall be true to line and grade, mortar-tight and sufficiently rigid to prevent objectionable deformation under load. Where forms for continuous surfaces are placed in successive unit, care shall be taken to fit the forms over the completed surface so as to obtain accurate alignment of the surface and to prevent leakage of mortar. Responsibility for their adequacy shall rest with the Contractor. The form surfaces shall be smooth, free from irregularities, dents, sags, or holes when used for permanently exposed surfaces. Bolts and rods used for internal ties shall be so arranged that when the forms are removed, all metal will be not less than 50mm (2 inches) from any concrete surface. Wire ties will not be permitted. All forms shall be so constructed and oiled so that they can be removed without hammering or prying against the concrete. All exposed joints shall be chamfered and suitable molding shall be placed to bevel or round exposed edges or corners, including the use of dummy chamfers and false
joints to provide a neat and uniform appearance, unless otherwise indicated on the drawing or directed.

Forms for exposed surfaces shall be coated with nonstaining mineral oil which shall be applied shortly before the coarse aggregate is placed. After oiling, surplus oil on the form surfaces and any oil on the reinforcing steel or other surfaces requiring bond with the concrete shall be removed. Forms for unexposed surfaces may be thoroughly wetted in lieu of oiling immediately before the placing of coarse aggregate, except that in freezing weather, oil shall be used.

When appropriate, during the pumping of the mortar, the forms shall be lightly vibrated on the outside in the vicinity of the mortar surface to remove air bubbles which sometimes adhere to the inside of the sheathing and to insure a continuous film of mortar between the aggregate particles and the forms. The vibrating shall be done on both the sheathing and the studs, using approved equipment.

Form removal shall be accomplished in a manner which will prevent injury to the concrete.

**Curing:**  Preplaced aggregate concrete shall be cured by the application of approved curing compounds or by continuous wetting as required under 511.17.

**Testing:** Prior to the start of the patching operation, the Contractor will be required to perform a 1m x 1m (3 foot x 3 foot) (approximately) test patch at a location selected by the Engineer. The Contractor will use the same procedures, materials, equipment and curing as will be used for the rest of the patching. A core will be obtained from the test patch to determine the filling of all voids, and that the required strength has been obtained. The average 7-day strength of three cores shall be 21MPa (3000 psi). After curing and before final acceptance of the finished project, all patched areas shall be visually inspected and sounded. In addition, the Contractor shall remove a representative number of cores shall be taken from the patched areas (at least one core per pier). The core locations shall be determined by the Engineer and shall extend into the concrete at least 200mm (8 inch). The cores shall be visually inspected for pockets, hollow areas, voids around reinforcing steel or other signs of improper consolidation. The cores shall then be tested by an independent testing laboratory approved by the Department. The average 28-day compressive strength of the cores shall be at least 31MPa (4500 psi). All defective patches as determined by the above methods shall be removed and replaced, followed by further coring. Core holes shall be filled with 705.21 material.

All coring, repair of core holes, independent laboratory testing and replacement of rejected areas shall be the responsibility of the Contractor and included in the unit price bid for this item.

**Method of Measurement:** The quantity shall be the actual area in square feet of the exposed surfaces of all completed patches, irrespective of the depth or thickness of the patch; if a patch includes corners or edges of such members as beams, curbs, columns, etc., all of the exposed surfaces shall be included, or if a patch extends completely through a member or a slab, both exposed surfaces shall be measured.

**Basis for Payment:** Payment shall be made at the contract price bid for:

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Square meter</td>
<td>Preplaced Aggregate Concrete (square feet)</td>
</tr>
</tbody>
</table>

APPENDIX – 109.11
ARN-24  STRUCTURAL SURVEY AND MONITORING OF VIBRATIONS

This proposal note was retired in April 2005.

ITEM SPECIAL - STRUCTURAL SURVEY AND MONITORING OF VIBRATIONS - 04/19/02

1.0 Description. This work consists of conducting a survey of the condition of structures and the monitoring of ground vibrations. The survey work is to be conducted before and after all construction work is performed which could cause undesirable ground vibrations. Ground vibrations and acoustics shall be monitored at the appropriate times during the duration of this project.

2.0 Personnel Qualifications. A Professional Engineer, registered in the State of Ohio, shall be engaged by the Contractor to be in charge of conducting a structural survey and in charge of monitoring vibrations and acoustics. The engineer in charge of performing the required work under this item is herein referred to as the Monitoring Foreman. The Monitoring Foreman and/or his team of experts shall have collectively worked on two similar projects or shall have collectively accrued not less than two years of successful experience in performing the type of work specified by this note. The monitoring foreman and/or his team of experts shall have expertise in (1) conducting structural surveys by video methods, (2) monitoring vibrations with a seismograph or with other appropriate instrumentation, and (3) assessing sites for potential damage that may occur as a result of the proposed construction. Documentation of this experience shall be furnished at the preconstruction meeting.

2.1 The requirement for the Monitoring Foreman to be an engineer can be waived provided that the Monitoring Foreman's experience or the collective experience of the monitoring team shows substantial expertise in performing the required work.

3.0 Structural Survey. The structural survey shall include but not be limited to the following:

3.1 Documentation of the integrity of existing building materials and the general overall condition of the structures recorded by written text, photographs, and VHS video cassette recording.

3.2 The establishment of locations and elevations of reference points, chosen by the Monitoring Foreman, for documentation of measurements.

3.3 A detailed on-site inspection conducted in the presence of the Project Engineer, the Contractor, property owners, property tenants if appropriate, and representatives of any involved utility companies.

3.4 Documentation of all structural deficiencies with regard to location, size, type, etc.

4.0 Monitoring of Vibrations and Acoustics. The monitoring of vibrations and acoustics shall include but not be limited to the following:

4.1 Determination and documentation of existing levels of vibrations and noise.
4.2 Monitoring of all construction operations that significantly contribute to the production of vibrations and noise with a special effort made to document the vibration and sound levels associated with blasting and/or pile installation procedures.

4.3 The development of criteria for controlling construction activities so that the Monitoring Foreman's allowable predetermined vibration levels are not exceeded during construction.

5.0 Water Quality. When appropriate, water samples shall be collected from wells, streams or project runoff areas to document before and after construction site conditions regarding the quality of water available in the vicinity of the project.

6.0 Ground Vibration. Vibration monitoring guidelines can be found in FHWA's May 1985 manual entitled "Rock Blasting" and in various other reports.

The peak particle velocity (PPV) of ground vibrations is generally used to monitor the effect of vibrations on structures. When monitoring vibrations consideration must be given to (1) the type of structure being evaluated and (2) the frequency of the vibrations (low frequency 40 Hz). Generally allowable ground vibration peak particle velocities range from 13mm/second (0.5 inches per second) to 50mm/second (2.0 inches per second) depending on the type of structure under consideration. When an allowable PPV is exceeded, the vibration producing operation shall be suspended and alternative construction procedures should be evaluated. The Director shall be consulted whenever the measured magnitude of the vibration level is considered potentially capable of producing structural damage.

7.0 Method of Measurement. The final twenty percent of the payment for this work shall not be made until the Office of Structural Engineering has received and approved three copies of the Monitoring Foreman's final report. The final report shall be typed and contain all measurements, conclusions, and recommendations which resulted from performing the above required work. Included with the reports shall be one copy of all pictures and video recordings. Interim reports shall be furnished to the Project Engineer during construction thereby keeping the Project Engineer informed of the Monitoring Foreman's progress and findings. The original tapes shall remain in the exclusive possession of the Monitoring Foreman for a period of not less than 10 years.

8.0 Method of Payment. Payment for this item will be made at the contract lump sum price for Item Special - “Structural Survey and Monitoring of Vibrations”.

ARN-25 RETIRED NOTE 17

Include the following note as part of an Item 202, “As Per Plan” note when protection of traffic is required.

[17] PROTECTION OF TRAFFIC: Prior to demolition of any portions of the existing superstructure, submit plans for the protection of traffic (vehicular, pedestrian, boat, etc.) adjacent to and/or under the structure to the Director at least 30 days before construction begins. These plans shall include provisions for any devices and structures that may be necessary to ensure such protection. Maintain the temporary vertical clearances specified on the plans or in the proposal at all times except as otherwise approved by the Director.
All costs associated with this traffic protection will be included with Item 202 for payment.

HISTORY: Note [17] was retired by the release of the 2005 Construction and Material Specifications. The information contained in Note [17] is entirely contained in CMS 501.05.

ARN-26 RETIRED NOTE 81

If the differential dead load deflection at each end of the crossframes is greater than ½" [13 mm], provide the following note. (Note - if part of a structure’s crossframes have a differential deflection of greater than ½" [13 mm] and part of the structure does not, use the following ERECTION BOLT note.)

ERECTION BOLTS AND CROSS FRAME FIELD WELDING: The hole diameter in the girder stiffeners shall be 3/16" [4 mm] larger than the diameter of the erection bolts. The cross frame members shall have slotted holes, 3/4" [19 mm] longer than the bolt diameter and 1/16" [2 mm] wider than the erection bolt diameter. The slot shall be parallel to the longitudinal dimension of the cross frame member. Erection bolts shall be high strength bolts and shall remain in place. Supply two hardened washers with each high strength bolt. Fully torque the bolts or use a lock washer in addition to the two hardened washers. Furnish erection bolts as part of Item 513.

Do not weld the cross frame members to the stiffeners until the concrete deck has been placed.

HISTORY: Note [81] was retired in order to reduce the potential for unanticipated girder deflection during deck placement. All crossframes and lateral bracing shall be permanently fastened before deck placement begins.
MSE WALL MOUNTED DEFLECTOR PARAPET
(Note: All reinforcing steel to be epoxy coated)

MSE WALL COPING
(Note: All reinforcing steel to be epoxy coated)
MSE WALL MOUNTED DEFLECTOR PARAPET
(NOTE: ALL REINFORCING STEEL TO BE EPOXY COATED)

MSE WALL COPING
(NOTE: ALL REINFORCING STEEL TO BE EPOXY COATED)

Figure APP-IM