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SECTION 100  GENERAL INFORMATION

101  INTRODUCTION

101.1  GENERAL CONSIDERATIONS

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 is shown in Figure 101. See Page 1-2

102  PREPARATION OF PLANS

Drawings should be so planned that all details will fall within the prescribed border lines. 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.

When describing directions or locations of various elements of a highway project, the centerline of 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]
OFFICE OF STRUCTURAL ENGINEERING

STRUCTURES ADMINISTRATOR

BRIDGE MAINTENANCE

ASST STRUCT. ADMIN. REVIEW

PRELIMINARY DESIGN

HYdraulics

FOUNDATIONS

DETAIL DESIGN

ASST STRUCT. ADMIN. CONSTRUCTION

CONSTRUCTION ENGINEER

STRUCTURAL STEEL SECTION

COMPOSITE RESEARCH ENGINEER

BRIDGE RESEARCH ENGINEER

ASST STRUCT. ADMIN. BRIDGE MGMT.

BRIDGE MGMT. SYSTEM

BRIDGE RATING

BRIDGE INVENTORY

ASST STRUCT. ADMIN. TRAINING/STANDARDS

TRAINING SECTION

BRIDGE STANDARDS

TABLE OF ORGANIZATION
Sheets in the bridge plans should be numbered in accordance with Figure 105. See Page 1-10.

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.

Metric plan sheet size to be used is 559 mm x 864 mm. Margins shall be 50 mm on the left edge and 15 mm 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 PLAN 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.

102.2 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 work for the Department shall be pre-qualified according to the requirements contained in the Departmental documental "Bridge Design Pre-qualification Procedure" which is on file with the Office of Contracts. Work shall be completed by those individuals upon whose experience the classification level of the design agency is based. 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 hand performed 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.

**102.3 MANUAL DRAFTING STANDARDS**

**102.3.1 GENERAL**

1. 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.
2. Plan sheets submitted to the Department shall be of extremely good quality on reproducible mylar.

102.3.2 LETTERING STANDARDS

1. All lettering shall be Braddock No. 5 size (upper case 4.0 mm in height), or larger.

2. Lettering within lined areas, such as quantity box, should at no time come in contact with any of these lines.

3. Letters should be properly spaced so that a crowded condition does not exist.

102.3.3 MANUAL DRAFTING LINE STANDARDS

1. "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.

2. All other lines and lettering shall be a minimum of "1" (Rapidograph pen size) (decimal width of 0.5 mm).

3. Individual lines shall be of uniform weight and density.

4. 1.5 mm is the minimum distance between two or more adjacent lines, even though an out of scale condition might exist.

102.4 COMPUTER AIDED DRAFTING STANDARDS

The Department has adopted MicroStation as its standard computer aided drafting and design (CADD) software. All bridge plans shall be submitted in MicroStation format. Files submitted shall meet the requirements within this section.

102.4.1 CADD FILE NAMING CONVENTION

MicroStation bridge files shall be designated by the structural file number (SFN) for the bridge

Example: SFN 4705337

Design file name 4705337.dgn

102.4.2 WORKING UNITS

Master Units = Millimeters
Sub Units = 10
Partial Units = 1000

All drawings shall be drawn at an appropriate scale.

102.4.2 TEXT

Text sizes defined below are the height text should be on the 559 mm X 864 mm standard plan sheet.

All Capital Letters
ODOT Font = 85
Normal Text, WT=1, TX=3.2 mm

Subtitles /Descriptions, WT = 1, TX = 3.6 mm
(Example of Subtitle):
SYMMETRICAL ABOUT CENTERLINE

Titles, WT = 2, TX = 4.5 mm
Titles shall be underlined with one line, WT = 3
(Example) PLAN, PROFILE

Main Titles, WT=2, TX=5.0 mm
Main Titles are normally used only on "Standard Drawing" Sheets

Title Block
Main Title WT = 2 TX = 4.5 mm
Sub-Title WT = 1 TX = 3.6 mm

The long dimension of a line terminator should be 5.0 mm on a finished 559 mm X 864 mm standard sheet. See Figure 102 Page 1-7

102.4.3 ELEMENT ATTRIBUTES

Levels allocated to the Office of Structural Engineering. LV = 41 - 50. See Figure 102 - Page 1-7 for example of line types.

The following CADD standards are established for use on bridge structure plans. These standards are for use with MicroStation by Bentley Corporation.

(a) - user defined

(b) - See Figure 103 - page 1-8 & Figure 104 - page 1-9

For legibility and neatness on a reduced set of plans, the minimum distance between two or more adjacent lines shall be 1.5 mm, even though an out of scale condition might exist.
OBJECT - LINE SHOWING THE PERIMETER OF A PROPOSED OBJECT (EX. PIER, ABUTMENT)
LV=42, CO=0, WT=2, LC=0

4 mm (TYP.) -- 2 mm (TYP.)

HIDDEN - LINE REPRESENTING AN OBJECT EDGE NOT SEEN IN THE VIEW. LV=42, CO=8, WT=1, LC=3

4 mm (TYP.) -- 40 mm (TYP.) -- 2 mm (TYP.)

CONSTRUCTION - LINE (LONG) USED TO REPRESENT A CONSTRUCTION JOINT
CONSTRUCTION - LINE (SHORT) SYSTEM DEFAULT LC=6
LV=42, CO=17 (SHORT), CO=1 (LONG), WT=1, LC=6

4 mm (TYP.) -- 40 mm (TYP.) -- 2 mm (TYP.)

CENTER - LINE (LONG) SHOWING THE CENTER OF AN OBJECT
CENTER - LINE (SHORT) SYSTEM DEFAULT LC=7
LV=42, CO=20 (SHORT), CO=4 (LONG), WT=0, LC=7

2 mm (TYP.) -- 2 mm (TYP.)

EXISTING - LINE TYPE USED TO REPRESENT THE EXISTING OBJECTS AT A SITE. LV=43, CO=5, WT=0, LC=2

DIMENSION - LINE TYPE TO BE USED FOR THE DIMENSIONING OF AN OBJECT. LV=44, CO=9, WT=0, LC=0

REBAR - LINE TYPE TO BE USED TO REPRESENT REINFORCING BARS
BREAKS IN LINE ARE TO BE MADE MANUALLY
LV=45, CO=6, WT=1, LC=0

DIA=1.25 mm -- 5 mm

LINE TERMINATOR - TERMINAL END OF A DIMENSION LINE
LV=44, CO=7, WT=0, LC=0

Figure 102
STANDARD TITLE BLOCKS

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

15 mm

25 mm

75 mm

165 mm

65 mm

25 mm

25 mm

50 mm

90 mm

![Example Site Plan Title Block]

Figure 105

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

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

![Example Plan Sheet Title Block]

THIS BLOCK SHOULD BE FILLED IN WITH THE NAME OF THE ACTUAL DESIGN AGENCY.
102.5 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.

102.6 TITLE BLOCK

See Figure 105 - Page 1-10 for example title blocks for 559 mm X 864 mm sheets.

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 SLK of the structure written without the decimal point. (Example: MER-707-27310)

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

Bridge stations shall be shown to the nearest 1/1000 of a meter.(millimeter) (Example: Sta 8+282.273)

The correct Structure File Number (SFN) shall be shown in the title block. Whether the structure is a new or an existing bridge, the Design Agency is responsible for contacting and confirming with the responsible District office the correct SFN to be shown.

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.

102.7 ESTIMATED QUANTITY CALCULATIONS

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. It is required that it be clearly shown that the calculations have been independently checked by someone other than the original designer. 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. Plan quantities should be shown to the nearest m², m³, etc.

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 which are accidentally misplaced are sometimes very difficult to identify if they are unmarked.

102.8 STANDARD DRAWINGS

Current standard drawings should be followed and used whenever practicable. Reference to standard 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 must be familiar with the standards and know if they are adequate for the particular design situation being addressed. If they are not, then standards must 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.

Only metric standard drawings are to be referenced. Any old or not converted standards using English dimensioning are to be converted by the Designer or the actual details drafted onto the bridge plan sheets. 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.

Standard drawings are available for purchase through the Department’s Office of Contracts or can be downloaded through the Office of Structural Engineering’s web page.

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

102.9 SUPPLEMENTAL SPECIFICATIONS

The Department has many Supplemental Specifications which 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 from the internet by accessing the Office of Contracts home page.

http://www.dot.state.oh.us/CONTRACT/SUPPLMNT/

The designer shall not modify supplemental specifications.

Supplemental specifications, like standard
drawings are to be listed on the General Notes plan sheet. (see section 600 Note [1]). The designer shall assure that standard drawings referenced on the General Notes sheet are also transferred to the Project Plans Title Sheet.

102.10 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 be obtained from the Office of Contracts home page.

Http://www.dot.state.oh.us/CONTRACT/PROPOSAL

Proposal notes are referenced in a 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".

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.

103.1 GEOMETRIC PROGRAMS

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

• COGO - Coordinate Geometry (MF)
• GEOPAK

103.2 DESIGN PROGRAMS

• GAD - Girder Automated Design (MF)
• BDS - Bridge Design Systems (MF) & (PC)
• BRASS - Bridge Rating and Analysis of Structural Systems (MF)
• BOXCAR - Box Culvert Structural Analysis (PC)
• MERLIN DASH - Beam and Girder Analysis and Design (PC)
• DESCUS 1 - Curved Girder (PC)
• LEAP - Conspan(PC)
• PCA COLUMN - concrete column design
• SIMON SYSTEMS - Version 7.00(PC)
• RISA3D - Structural Analysis
• VCON - curved steel bridge structures

103.3 HYDRAULIC ENGINEERING
100 PROGRAMS

- HEC-2 or HEC-RAS - Computations of Water Surface Profiles in Open Channels (PC)
- HY7 - (WSPRO) Water Surface Profiles (PC)
- HY8 - Culvert Hydraulics (PC)
- HY9 - Scour Analysis (PC)
- HEC-12 - Pavement Drainage (PC)
- Long Span Culverts (PC)
- Universal Culvert Program (PC)
- Special Culvert Program (PC)
- HYDRA V3.2 (PC)
- Inlet Spacing (PC)

103.4 GEOTECHNICAL ENGINEERING PROGRAMS

- PICAP - Pile Capacity (PC)
- SHAFT - Drilled Shafts (PC)
- COM624P - Lateral Loading of Piles and Drilled Shafts (PC)
- WEAP - Wave Equation Analysis of Pile Driving (PC)
- STABL - Slope Stability Analysis (PC)
- SPW911 - Sheet Pile Design and Analysis (PC)

103.5 BRIDGE RATING PROGRAM

- BARS - Bridge Analysis and Rating System (MF) & (PC)

104 OHIO REVISED CODE

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:

- Site Plan
- General Plan
- Design Data & General Notes
- Estimated Quantities
- Phase Construction Details
- Abutments
- Piers
- Superstructure
- Railing Details
- Expansion device details
- Reinforcing Steel List

The General Plan sheet no longer requires an elevation view. The General Plan sheet is only required for:
<table>
<thead>
<tr>
<th>Deck overlay projects</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck replacement projects where the bridge deck is variable width or curved.</td>
</tr>
<tr>
<td>New bridge of variable width or curved alignment.</td>
</tr>
<tr>
<td>New or rehabilitated structure requiring staged construction</td>
</tr>
</tbody>
</table>

The General Plan sheet not longer requires an elevation view. 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.
SECTION 200     PRELIMINARY DESIGN

201 INTRODUCTION

The structure 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. When a bridge type structure is the most appropriate structure for a particular site, then a preliminary design study needs to be performed to determine the appropriate bridge type.

A cost analysis comparing alternate structures shall be performed, unless the site conditions discourage the use of all but one type of structure. The cost analysis should be based on the initial construction cost with consideration to all future maintenance costs. 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.

A preliminary design submission should include one(1) copy of the following, except for the site plan(s) which require two(2) copies:

a. The approved Line, Grade and Typical Section (LG&T) roadway submission which includes the title sheet, alignment and profile plan and typical sections.

b. A copy of the approval letters for LG&T and environmental clearances. Correspondence from the ODOT District Utilities Coordinator authorizing any proposed utilities on the bridge. LG&T and environmental document approval shall be completed prior to submitting the preliminary design submission. For a railway crossing any correspondence with the Railroad should be submitted.

c. The bridge Site Plan, as described in Section 202. The LG&T review comments should be resolved and all comments incorporated on the Site Plan prior to submittal.

d. 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.

e. Foundation information and recommendations.
f. Hydraulic submittal, including hydraulic and scour calculations. A copy of the scour calculations shall be provided to the ODOT District Engineer responsible for bridge inspection.

g. Preliminary stage construction or temporary structure details, if applicable, as described in Section 208.

h. A Supplemental Site Plan for waterway and railway crossings, see Section 202.6 and 202.7 respectively.

i Preliminary sketches and photos for rehabilitation projects as described in Section 206.

Detail plans shall not be initiated until after preliminary design approval.

202 SITE PLAN INFORMATION

202.1 GENERAL

The Site Plan scale generally should be 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 to 200 scale is too small to clearly show the Site Plan details a 1 to 100 scale may be considered. The following general information should be shown on the Site Plan:

a. The plan view should show the existing structures (use dashed lines), contours at 0.5 meter intervals showing the existing surface of the ground (for steep slopes contours at 2.0 meter or greater intervals may be used), existing utility lines and their disposition, proposed structure, the proposed temporary bridge, the 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 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), AREA Specifications for increased lateral clearances required when tracks are on a horizontal curve.

b. A profile drawn to the same vertical and horizontal scale along the proposed centerline of the road, for the full length of the bridge, should show the existing and proposed profile grade line, the cross-section of channel, an outline of structure, highest known high water marks, normal water elevation, flow line elevation...
(thalweg), design highwater elevation, including backwater, and 100 year highwater elevation. Also the overtopping flood elevation, magnitude and estimated frequency should be shown if less than the 100 year highwater elevation. Note normal water elevation (which defines the low water table) is the elevation at the top of the un-vegetated channel.

c. Horizontal and vertical curve data.

d. Size of drainage area. The elevation, discharge and stream velocity through the structure for both the 100-year frequency base flood and the design year 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, skew angle, original design loading or upgraded loading, type of deck and type of substructure, date when built, structure file number, 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, width of sidewalks, design 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 following earthwork note: "EARTHWORK limits shown are approximate. Actual slopes shall conform to plan cross sections."

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

j. For bridges with guardrail on the bridge, stationing of first guardrail posts off the bridge should be given to the nearest mm. Guardrail station may be changed during the detail design phase and then revised on the Site Plan. For bridges with concrete barrier railing, the first guardrail post from the end of the barrier should be stationed.

k. For each location where a bearing is to be used, the bearing (fixed or expansion) shall be designated in the profile view (FIX or EXP)

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

m. A cross section sketch at the abutments shall be submitted to provide information to help verify bridge limits.
202.2 SITE PLAN DETAILS

The preliminary design of the proposed structure should be shown on the Site Plan in both plan and elevation. The preliminary design should consist of a plan layout of the proposed structure, 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, treatment of slopes around the ends and under the bridge, channel changes, centerline of temporary structure and temporary approach pavement, and stationing of bridge limits, the latter being the bridge ends of approach slabs. Also, a profile or sectional view along the centerline of roadway should be shown, including embankment slopes and top of slope elevations, proposed footing elevations, type of foundations, soil boring locations, highway grade elevations (at 10 meter increments) and gradient percent, vertical clearance (both the minimum required and the actual) over a railway or highway and their location in the plan view.

The spans should be measured from center to center of bearings along survey. For concrete slab bridges, the centerline of the abutment bearings is assumed 190 mm behind the face of abutment substructure or breastwall. On curved bridges a reference line (chord) drawn from centerline to centerline of abutment bearings at the centerline of survey should be shown and the skew angle given between the reference chord and the abutment bearings. The bearings of the tangents of the roadway centerline should be shown. The profile view of the proposed bridge shall be highlighted by shading in the areas that represent the new bridge components. If the bridge crown (superelevation) changes across the structure a superelevation transition table or diagram, similar to figure 208, page 2-40, shall be provided on the Site Plan or in the detail plan set. Reference should be made to the table or diagram when detailing the typical bridge transverse section.

In the plan view on the Site Plan, show the limits of channel excavation by crosshatching the area to be excavated. Provide a legend on the Site Plan explaining the purpose of the cross hatched area. Bench marks should be shown on the Site Plan.

202.3 PROPOSED AND EXISTING STRUCTURE BLOCK

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 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 165 mm wide for 559 x 864 mm sheet size.
202.4 UTILITIES

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

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. The ODOT Utilities Manual should be referred to. Utilities shall be installed in substantial ducts or enclosures adequate to protect the lines from future bridge repair and maintenance operations. Utilities should be precluded from bridges whenever possible. Utilities shall not be placed inside of prestressed concrete box beams. For some specific detail issues with utilities on bridges refer to section 301.6 of this manual.

c. Stream cross-section plots representing the natural stream conditions. The cross-section plots should indicate the Manning’s "n" values chosen for the channel subsections.

d. 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.

e. 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.

f. A plan sheet showing the approximate location of the stream cross-sections.

g. Hydraulic calculations for the computation of backwater and mean velocities at the proposed bridge for both the design year and 100 year frequency discharges. On computer analyses, with more than one sheet of computer input data a floppy disk of the input data should be provided.

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

2. 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 Site Plan.

202.5 HYDRAULIC SUBMITTAL

The following information should be included with the preliminary design submission:

a. Drainage area determination in Km².

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

c. Stream cross-section plots representing the natural stream conditions. The cross-section plots should indicate the Manning’s "n" values chosen for the channel subsections.

d. 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.

e. 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.

f. A plan sheet showing the approximate location of the stream cross-sections.

g. Hydraulic calculations for the computation of backwater and mean velocities at the proposed bridge for both the design year and 100 year frequency discharges. On computer analyses, with more than one sheet of computer input data a floppy disk of the input data should be provided.

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

2. 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 Site Plan.
h. Bridge scour analysis, a narrative of findings and recommended scour countermeasures. See Section 203.3 for additional information.

i. A brief narrative discussion of 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. A flood hazard evaluation, which is a condition statement should be made regarding the nature of the upstream area, the extent of upstream flooding and whether buildings are in the 100 year frequency flood plain.

3. High water data from local residents and observed high water marks including their locations should be included in the narrative.

4. 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.

**202.6 SUPPLEMENTAL SITE PLAN FOR WATERWAY CROSSINGS**

For Waterway Crossings a Supplemental Site Plan is necessary. A Supplemental Site Plan should show information necessary for the determination of the waterway opening, and should accompany the Site Plan when information cannot be shown adequately on the plan and profile. The following information should be shown:

a. A small scale area plan showing an accurate waterway alignment at least 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.

b. A stream profile at least 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.

c. 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 300 meters upstream or downstream from the proposed bridge, show stream cross sections including the structure and roadway profiles of the overflow sections of the structures. These may be used as a guide in establishing the waterway requirements of the proposed structure.
d. 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.

e. 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.

202.7 SUPPLEMENTAL SITE PLAN FOR RAILWAY CROSSINGS

For Railway-Highway grade separation structures a Supplemental Site Plan is required in the Scope of Services. The Supplemental Site Plan should be completed and submitted during the LG&T review process. 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 to 1000 scale plan of the alignment of the railroad and the highway extended at least 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 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.

gh. Railroad right-of-way lines.

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

203 BRIDGE WATERWAY

203.1 HYDROLOGY

a. Discharges shall be estimated by the method described in USGS Water-Resources Investigations Report 89-4126 "Techniques for Estimating Flood-Peak Discharges of Rural Unregulated Streams in Ohio".

For urban drainage areas less than 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 Report 89-4126 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.


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 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 and Freeway Ramps...... 50 years

2. Other Highways (2000 design ADT and over)......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. Ohio Department of Natural Resources may be contacted for a listing of local flood plain coordinators.

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. Specific bridge hydraulics need not be evaluated at the preliminary development phase when alternative highway alignments are studied for the project.

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 National Flood Insurance Program (NFIP) designated flood plain in the floodway fringe, the rise in the water surface is limited to 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 - Page 2-10

Longitudinal encroachments require alternative location studies to be summarized in the environmental documents at the preliminary development phase. 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. 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.
Figure 2-10

FLOODWAY SCHEMATIC

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.
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 or other reasons.

On wide flood plains, the roadway profile should be adjusted to allow overflow over the approach roadways for floods which exceed the design year discharge. The bridge profile should be set to allow free board above the overtopping flood.

k. Spill-thru type structures are generally preferred for cost effectiveness and hydraulic efficiency.

### 203.3 SCOUR

For all bridges over waterways, the entire spill-thru slope, including the corner cones in front of the abutments, should have a minimum protection of CMS 601.08 Rock Channel Protection, Type C, 600 mm thick with filter. A filter fabric, which is allowed as an alternate in CMS 601.08, may be specified instead of the 150 mm bed of No. 3 or 4 crushed gravel under water placement. When the intent is to use filter fabric, the pay item description should read "...with fabric filter", and not "... with 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 thicknesses.

<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 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 will be supported by piling or drilled shaft foundations unless the footings can be founded on bedrock.

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 204.4, 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. Scour calculations only need to be made for bridges not founded on shale or bedrock. All major rehabilitation work requires a scour analysis. The scour analysis may consist only of determining what the bridge is founded on. Probe existing footings founded on shale to determine weathering of the shale and 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- IP-90-017), "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 2 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.

204 FOUNDATION INFORMATION

Foundation information to be obtained by the Design Agency should conform to the current ODOT Specifications for Subsurface Investigations. The Design Agency’s analysis of the foundation investigation, the recommendation for foundation types, design parameters and the Subsurface Investigation Report prepared on full size plan sheets shall be submitted for the Department’s approval.

When evaluating scour, use the information from borings normally required for a bridge foundation investigation except for special situations which may justify obtaining an extra boring.

Probable 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.

204.1 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, \( q_u \).

Elevations for the bottom of the footing shall be shown on the Site Plan. Preliminary design loads, the estimated size of the footing and the allowable bearing pressure shall be provided for review with the preliminary design submission. 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.

### 204.2 PILE TYPE, SIZE AND ESTIMATED LENGTH

The type, size and estimated length of the piles for each substructure unit shall be shown on the site plan. Preliminary pile design loads and approximate pile spacings shall be provided. This information will be furnished by the design agency preparing the plans. The estimated pay length(s) for the piling should be given to the nearest meter. Procedures for computing estimated pay length of the piles are given in the FHWA's "Manual on Design and Construction of Driven Piles", FHWA-DP-66-1. Minimum pile tip elevations for friction designed piles may be required and should be shown on the Site Plan.

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

#### 204.2.1 STEEL 'H' PILES

When piles are driven to refusal on the bedrock, steel 'H' piles are generally used. The commonly used pile sizes are:

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2).

<table>
<thead>
<tr>
<th>H Pile Size</th>
<th>Design Load</th>
<th>Ultimate bearing Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP250X62</td>
<td>500 kN</td>
<td>1000 kN</td>
</tr>
<tr>
<td>HP310X79</td>
<td>650 kN</td>
<td>1300 kN</td>
</tr>
<tr>
<td>HP360X108</td>
<td>850 kN</td>
<td>1700 kN</td>
</tr>
</tbody>
</table>

Design load values for H piles are based on a maximum service load stress of 62 MPa.
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, HP250X62 should be used if the calculated design load is less than 500 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 hard bedrock. When the depth of overburden is more than 15 meters and the soils are cohesive in nature, piles driven to hard 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.

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

204.2.2 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.

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2).

<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>

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, 400 mm diameter piles shall be used. 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 - 19M epoxy coated reinforcing bars with 13M spiral at 300 mm pitch should be provided for 400 mm diameter piles. Reinforcing steel shall be detailed on the plans, included in the reinforcing steel list, and paid for payment under item as Item 507, 400 mm Cast in place piles furnished, As Per Plan. The reinforcing steel cage should extend 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 6 meters, 450 mm diameter piles can be used. The Office of Structural Engineering (Attn: Foundation Engineer) shall be consulted before recommending the use of 450 mm diameter piles.

**204.2.3 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 for note.

**204.2.4 PILE WALL THICKNESS**

Minimum pipe pile wall thicknesses are specified by formula in CMS 507.

**204.2.5 PILE HAMMER SIZE**

Pipe pile hammer sizes are specified in CMS 507, based on Ultimate Bearing Load and the required blow count range.

**204.2.6 CONSTRUCTION CONSTRAINTS**

For construction constraints regarding pile installation and embankment construction, see Section 600, TYPICAL GENERAL NOTES.

**204.3 DRILLED SHAFTS**

The diameter of the drilled shafts for the abutments and piers shall be shown on the 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 Site Plan. This information will be furnished by the design agency preparing the plans.


Drilled shafts should be considered when their provision would:

1. preclude the need of cofferdams
2. become economically viable due to high design loads (eliminates the need of large quantities of pile)
3. provide protection against scour
4. provide resistance against lateral and uplift loads
5. accommodate sites where the depth to bedrock is too short for adequate pile embedment but too deep for spread footings
6 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. Such design shall be submitted to the Office of Structural Engineering (Attn: Foundation Engineer) for review.

Drill shaft specifications are defined in CMS 524.

The Design Agency should have the Department review any proposed drilled shaft plan notes during the preliminary design submission. The drilled shaft sequence of construction note has often been misused. This note is only appropriate for closely spaced drilled shafts used as a retaining wall.

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 undisturbed shale is bedrock) the minimum depth of the bottom of the footing below the stream bed, D, in meters, shall be as computed by the following:

\[ D = T + 0.50Y \]

\[ T = \text{Thickness of footing (in meters)} \]
\[ Y = \text{distance from bottom of stream bed to surface of bedrock (in meters)} \]

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

### 204.4 FOOTING ELEVATIONS

Substructure footing elevations should be shown on the Site Plan. The top of footing should be a minimum of 0.3 meters below the finished ground line. The top of footing should be at least 0.3 meters below the bottom of any adjacent drainage ditch. The bottom of footing shall not be less than 1.2 meters below and measured normal to the finished groundline.

### 204.5 EARTH BENCHES AND SLOPES

Generally, a bench at the face of abutment should not be used.

Generally, spill thru slopes should be 2:1, except where soil analysis or existing slopes dictates flatter slopes.
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 sightly structure.

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

### 204.6 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 1200 mm. For stub abutments on spread footing on soil the minimum dimension shall be 1650 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.

### 204.7 MECHANICALLY STABILIZED EARTH WALLS AT ABUTMENTS

When MSE walls are proposed, approval will be required by the Department. Approval shall be obtained prior to contacting the proprietary wall companies and requesting MSE project plans.

Mechanically stabilized earth (MSE) walls with stub type abutments on piling or spread footings may be considered when appropriate. All geotechnical and structural concerns must be studied prior to recommending the use of MSE wall designs. If stub abutments are to be supported by piling, the piling shall extend thru the MSE embankment. Use of stub abutments on MSE walls, whatever foundation type used, requires approval of the Foundation Engineer of the Office of Structural Engineering.

### 204.8 PIER TYPES

For highway separations generally the pier type should be cap-and-column piers. 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 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 6 meters. For heights greater than 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 the use of T-type piers 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 is limited T-type piers 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.

### 204.9 RETAINING WALLS

This section defines the process that must be followed to develop project plans for various types of retaining walls. Listed below are the retaining wall types that can be used on ODOT projects.

**Non-Proprietary Systems**

a. Cast-In-Place Reinforced Concrete.

b. Tied-Back Wall.

c. Drilled-In Reinforced Concrete Piers.

d. Sheet Piling.

e. H-Piling with Lagging.

f. Cellular Retaining Wall (Bin or Crib)

**Proprietary Systems**

a. Doublewal
   Doublewal Corporation
   59 East Main Street
   Plainville, CT 06062
   (203) 793-0295

b. Genesis Walls
   TENSAR Earth Tech., Inc.
   3000 Corporate Ctr. Dr.
   Suite 370
   Morrow, Georgia 30260
   (404) 968-3255

c. Tensar Ares Retaining Wall System
   TENSAR Earth Tech., Inc.
   5775-B Glenridge Drive.
   Suite 450
   Atlanta, Georgia 30328
   (404) 250-1290

d. Hilfiker Retaining Walls
   T & B Structural Systems
   637 West Hurst Blvd.
   Hurst, Texas 76053
   (817) 280-9858

e. Reinforced Earth Walls
   The Reinforced Earth Co.
   760 Pasquinelli Drive
   Suite 344
   Westmont, Illinois 60559
   (708) 655-0044

e. Retained Earth
   The VSL Corporation
   P.O. Box 866
   8006 Haute Court
   Springfield, VA 22105
   (703) 451-4300
f. T-Wall
   Hydro-Conduit
   620 Liberty Road
   Delaware, Ohio 43015
   (800) 395-8383

The use of Genesis and T-Wall type retaining walls is limited to a wall design height not more than 7.6 meters and non-critical locations. The proprietary wall types that are approved for supporting abutments on piles or spread footings are Reinforced Earth, Retained Earth. Tensar Ares may be used at abutment locations where the abutments are supported on piles and the wall height is limited to 9.15 meters.

Note that Hilfiker company furnishes a variety of wall types. These guidelines apply only to their wall design referred to as "Reinforced Soil Embankment" with precast reinforced concrete face panels.

204.9.1 PRELIMINARY DESIGN

When a justification study or an early determination has found that a retaining wall is required, generally the wall will be (1) a cast-in-place reinforced concrete wall or (2) a proprietary wall system. All other wall systems are used for unique situations that have, for example, right-of-way and/or excavation limitations. These unique situations will require that only one type of retaining wall design be prepared.

During the preliminary design stage, Office of Structural Engineering and/or Office of Materials Management, Geotechnical Section shall determine the appropriate type of wall(s) for each project. If proprietary walls are chosen, the Office of Structural Engineering and/or Office of Materials Management, Geotechnical Section will select the wall companies eligible for each wall site. The selected wall companies shall be invited to prepare retaining wall plans. The proprietary wall companies may prepare detail plans or decline to participate in the project. When more than one wall company furnishes detail plans, a cast-in-place reinforced concrete wall generally will not be designed.

Generally the use of proprietary retaining walls is to be considered when the wall quantity for the project exceeds 450 square meters.

204.9.2 DESIGN CONSTRAINTS

Below are some design constraints the Office of Structural Engineering and/or the Office of Materials Management, Geotechnical Section consider when establishing acceptable wall types:

a. Future use of the site (note that future excavations can not be made in Mechanically Stabilized Embankments),

b. Deflection and/or differential settlement,

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) and,

i. Railroad policies.

204.9.3 PRELIMINARY DESIGN APPROVAL

The Office of Structural Engineering and/or the Office of Materials Management, Geotechnical Section will review the Consultant's proposed foundation design and furnish the design agency final footing elevations and an allowable bearing pressure for each wall type to be used on the project. For spread footing abutment alternatives design pressures for the abutment footing shall be furnished by the design agency. Upon ODOT's review and selection of the wall types at the preliminary design stage, authorization for the detail design of specific wall types will be given and the design agency shall proceed to Detail Design.

204.9.4 CAST-IN-PLACE REINFORCED CONCRETE WALLS

When a cast-in-place reinforced concrete wall is to be designed, the design agency shall prepare the complete wall design. The Design Standard Drawing RW-1-98M should be followed.

204.9.5 DETAIL DESIGN OF PROPRIETARY WALLS

The design of the wall shall be in conformance with the latest edition of "AASHTO Standard Specifications for Highway Bridges" except as listed below:

a. The design shall meet all plan requirements for the project.

b. Wall sections intersect creating an included angle of 130 degrees or less, a vertical corner element, separate from the standard panel face, shall abut and interact with the opposing standard panels. The corner element shall have mesh reinforcement connected specifically to that plane and shall be designed to preclude lateral spread of the intersecting panels.

c. The corrosion rate for the all soil reinforcements and connections shall be as per "AASHTO Section 5.8.6.1", based on a 100 year design life.

Steel reinforcements shall comply with AASHTO 5.8.6.1.1

Geosynthetic reinforcements shall comply with AASHTO 5.8.6.1.2 and:

Tensar Ares geogrid

\[ T_{\text{ULT}} = 131.3 \text{ (kN/m)} \text{ UX1600HS} \]
\[ T_{\text{ULT}} = 157.6 \text{ (kN/m)} \text{ UX1700HS} \]

\[ RF_{\text{cr}} = 3.1 \]
\[ RF_{\text{D}} = 1.1 \]
\[ RF_{\text{ID}} = 1.25 \]
d. The height of the wall for design purposes shall be measured from the top of the leveling pad to the top of the coping. When the retaining wall is supporting a sloping embankment or a surcharge load, the "equivalent design height" (h) as shown in AASHTO Figure 5.8.2B.

e. The thickness of the unreinforced concrete leveling pad shall not exceed 150 mm.

f. The minimum thickness of the precast reinforced concrete face panels shall be 140 mm.

g. The yield stress for the metallic soil reinforcement shall not exceed 450 MPa. For polymeric reinforcement the maximum stress shall not exceed LTDS, using AASHTO criteria.

h. The wall system (regardless of the size of panels) shall accommodate up to one (1) percent differential settlement in the longitudinal direction.

i. When the retaining wall is supporting a sloping embankment or a surcharge load, the minimum length of the soil reinforcement shall be 70 percent of the "equivalent design height" (h) as in AASHTO Figure 5.8.2B.

j. The vertical stress at each reinforcement level shall be computed by considering local equilibrium of all the forces acting above the level under investigation. The vertical stress (bearing pressure) at each reinforcement level shall be computed using the Meyerhof method in the same manner as the bearing pressure computed for the base of the wall. This applies to both extensible (polymeric) and inextensible (metallic) reinforcements.

k. For a Doublewal design, the lateral pressure is computed on a vertical plane passing through the back of the heel of the lowest module.

l. The following factors of safety shall be satisfied for all proprietary walls:

<table>
<thead>
<tr>
<th>Factor of Safety</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding</td>
<td>&gt; 1.25</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;1.25</td>
</tr>
</tbody>
</table>

Factor of safety for settlement will be site specific.

m. 100% of the ground reinforcement designed and placed in the reinforced earth volume shall extend to and be connected to the facing element. No field cutting of reinforcement systems to avoid piles or other obstacles is acceptable.
n. Under service loads, the factor of safety at the connection between the face panel and the ground reinforcement shall be 1.5 at a 13 mm horizontal movement of the facing element.

o. The coefficient of lateral earth pressure $k_a$ shall be computed using Coulomb equation.

p. For select granular material the value for the angle of internal friction for design purposes shall not exceed 34 degrees.

q. Each wall panel shall have at least two connections to soil reinforcements and the connections shall not be vertically separated by more than 760 mm.

Special design features such as parapets and drainage provisions, must be comparable for all wall designs.

204.9.6 INFORMATION PACKAGE FOR PROPRIETARY WALLS

When one or more of the proprietary wall types have been approved as a result of the preliminary design review, the design agency shall prepare an information package containing the following information:

a. Description of the site,

b. Wall location with stations and offsets,

c. Top of wall elevations,

d. Roadway cross-sections,

e. Safety-barrier or railing requirements,

f. Special coping requirements,

g. Utility accommodations,

h. Drainage requirements,

i. All special design features,

j. Subsurface information (The proprietary wall companies must be informed of all geotechnical stability and settlement concerns),

k. Wall surface textures,

l. Wall design criteria, and

m. Minimum footing depth (frost penetration depth).

A copy of the information package shall be sent to each selected wall company and one copy must be sent to the Office of Structural Engineering (Attn: Foundation Engineer). The proprietary wall companies shall be given three weeks to review the project and inform the design agency in writing of their decision to participate in preparation of detail plans for the project. The design agency must furnish ODOT a copy of each wall company's response or a statement that the wall company did not respond.
204.9.7 WORK PERFORMED BY THE PROPRIETARY WALL COMPANIES

The proprietary wall companies will design their retaining walls and furnish detail plan tracings and design computations to the design agency. If computer programs are used to prepare the design, the output and an example hand calculation shall be furnished. All required reinforcing steel for cast-in-place concrete shall be shown on the plans.

Proprietary wall manufacturer’s detail tracings shall include all dimensional and reinforcing details for each panel, whether standard or special. Elevation sheets of the walls shall define the individual panel to be used in each specific location. Plans shall not only include details to construct the wall but also fabricate all individual components of the wall.

c. Prepare the reinforcing steel bar list and bar marks as per ODOT standards.

d. Analyze the retaining wall designs for:

1. Bearing capacity of the foundation
2. Sliding
3. Overturning
4. Total and differential settlement
5. Overall slope failure.

e. Ensure that all Mechanically Stabilized Embankment (MSE) and Prefabricated Modular Wall designs are in strict conformance with the latest "AASHTO Standard Specifications for Highway Bridges" except as noted in Section 204.9.5 of this Manual.

f. Insert the wall tracings in the project plans and provide all applicable plan notes necessary to coordinate the wall designs with the total project.

2-23
WALL QUANTITIES

TOP OF PAVEMENT

LIMITS OF ITEM 203, SELECT GRANULAR EMBANKMENT

ITEM 203, EMBANKMENT

EXISTING GROUND

ITEM 503, UNCLASSIFIED EXCAVATION, AS PER PLAN

EXCAVATION OR SHEETING BEYOND THIS LIMIT FOR THE CONSTRUCTION OF THE RETAINING WALL SHALL BE PAID FOR UNDER ITEM 503, COFFERDAMS, CRIBS AND SHEETING AND SHALL BE INCLUDED WITH THE WALL QUANTITIES.

150mm THICK UNREINFORCED CONCRETE LEVELING PAD

ITEM 703.33 150mm PERFORATED POLYETHYLENE DRAINAGE PIPE

STRAIGHT LENGTH

THE WALL QUANTITIES BOUNDARY IS SET BASED ON THE LONGEST STRAP LENGTH OF THE VARIOUS PROPRIETARY COMPANIES SPECIFIED PLUS APPROXIMATELY 600 mm

APPROXIMATELY 600mm FOR PROPRIETARY WALL COMPANY WITH THE LONGEST STRAP LENGTH
g. Indicate all required modifications to the Special Provisions for each wall design. (See appendix A of this manual for Special Provisions) A master copy of the Special Provisions for each wall design can be obtained from the Office of Structural Engineering (Attn. Foundation Engineer). Generally, the sections titled "Method of Measurement", "Basis of Payment" and "Coping" require modifications pertinent to the specifics of the project. The final copy of the Special Provisions for each design shall be the designer’s responsibility to furnish the Department similar to tracings and proposal notes.

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 other types of superstructures may be considered. Contact the Office of Structural Engineering prior to initiating any other 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.

When a multiple span arrangement (4 spans or more) is required, preliminary cost estimates 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.

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 drawings are available for the design of single span and three span continuous concrete slabs. The Standard Drawing for single span concrete slab bridges is SB-6-94M. The span ranges from 3350 to 11 580 mm and have a maximum skew angle of 35 degrees. The Standard Drawing for three span continuous concrete slabs is CS-1-93M. The spans range from 4260 mm - 5334 mm - 4260 mm to 13 360 mm - 16 700 mm - 13 360 mm and have a maximum skew angle of 30 degrees. Both standard drawings reinforcing will need to be re-evaluated as the current design is based on a liveload factor of 1.67 and does not include the 25% increase required by this manual. (1.25 x 1.67 = 2.09)

205.4 PRESTRESSED CONCRETE BOX BEAMS

The span limits for prestressed concrete box beams generally range from 9 to 27 meters. The maximum allowable skew is 30 degrees. For rare occasions where the skew is between 30 and 45 degrees box beams may be considered, but approval shall be obtained from the Office of Structural Engineering. For all four lane divided highways or where the design ADTT 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 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 200 mm. For multiple span bridges, individual span lengths may vary but the proposed box beam depth should be constant.

Designs should consider the site limitations for practical hauling. While weight of a precast bridge member may be a limiting factor, length and capability to reach the jobsite with the members may also be a restriction. Gross vehicle weights (vehicle plus load) exceeding 54 400 kg require special hauling permits. The vehicle available for highway transportation of heavy members generally weigh between 16 000 and 18 000 kilograms.

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:

\[
Cg = \sqrt{\text{transverse deck grade}^2 + \text{roadway grade}^2}
\]

For a normal transverse deck grade horizontal to vertical of 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. composite design should be used.
205.5 PRESTRESSED CONCRETE I BEAMS

The span limits for prestressed concrete I beams (AASHTO Type II, III and IV, Modified IV) generally range from 18 to 39 meters. The shapes are to conform to Standard Drawing PSID-1-95M. Generally the minimum economical beam spacings will be between 2100 and 2450 mm. 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 drawing PSID-1-95M is based on straight strand, debonded designs to allow for maximum bidding competitiveness between prestress fabricators. The use of draped strands, beam sections other than shown in Standard Drawing PSID-1-95M or 28 day concrete design strengths above 38 MPa are not allowed without authorization by the Office of Structural Engineering.

205.6 STEEL BEAMS AND GIRDERS

Steel rolled beams, up to and including 1000 mm depth, and welded plate girders generally should be considered for spans greater than 18 meters. 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. For continuous, composite designed rolled beams, generally the maximum intermediate span is limited to around 35 meters. For simple, composite designed rolled beams, generally the maximum span is limited to around 30 meters. In the preliminary design phase haunched girders over the intermediate substructure units should be considered for spans greater than 90 meters.

<table>
<thead>
<tr>
<th>Steel Stringer type</th>
<th>Span length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled Beam</td>
<td>18 - 35 m</td>
</tr>
<tr>
<td>Girder</td>
<td>30 - 90 m</td>
</tr>
<tr>
<td>Haunched Girder</td>
<td>&gt; 90 m</td>
</tr>
</tbody>
</table>

Generally the minimum economical beam spacing for rolled beams is 2450 mm. For plate girders a minimum spacing of 2750 mm is generally recommended.

In order to facilitate forming, deck slab overhang should not exceed 1200 mm. On over the side drainage structures the minimum overhang shall be 700 mm. Where scuppers are required for bridge deck drainage the overhang shall be 450 mm.

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

The type of steel to be specified should be A588M weathering steel over waterways and railroads. A572M painted steel should be used over highway crossings. In addition
A572M painted steel should be used in industrial areas, corrosive areas or where subjected to accumulations of debris and exposed to continuous moisture.

A36M painted steel may be considered if deflection constraints do not justify the use of the higher strength steel or the resulting design is more economical.

For bridges with significant substructure costs, the difference in dead loads between the steel superstructure versus a concrete superstructure should be considered in the preliminary cost estimates for choosing the most economical structure type.

For rehabilitation of steel beam or girder superstructures, the fatigue analysis will need to be completed and the tables of results for Methods A and B, defined in 402.2, included in the preliminary submittal package.

**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). These abutment designs are appropriate for bridge expansion lengths up to 75 meters (125 meters total length, assuming 2/3 movement could occur in one direction) and a maximum skew of 30 degrees. See Figure 203 - Page 2-29 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 symmetrical, uncurved structures. See Figure 203 - Page 2-29 for additional limitations and Figure 204 - Page 2-30 for an example of a integral design. Standard Drawing ICD-1-82M establishes details for integral abutment designs.

**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.

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.

**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 80 meters (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.
SEM I- INTEGRAL/INTEGRAL ABUTMENT TYPE

SKEW VS. BRIDGE LENGTH LIMITATIONS (mm)

Figure 203
2-29
between the end of the approach slab and the roadway. This design should be used for symmetrical, uncurved (straight beams) structures. See Figure 205 - Page 2-32 for example of a semi-integral design.

Spread footings may be appropriate for semi-integral abutments but settlement needs to be evaluated and approval for their use should be received from the foundation engineer, Office of Structural Engineering.

To utilize this 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, reference Section 209.6, Pressure Relief Joints.

Standard Drawing SICD-96M establishes details for semi-integral abutment designs.

206 REHABILITATION PROJECTS

Preliminary design submittals for all bridge rehabilitation projects shall have a General Plan. A Site Plan is required where bridge deck widening is proposed and widening involves the existing substructure or earthwork is to be performed. For rehabilitation projects involving roadway elevation changes a General Plan sheet is required.

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 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 sheet containing the General Plan.

Overlay projects should not require a site plan but the general plan should define all necessary information.

A typical section should be furnished to show proposed design changes. Sketches of any proposed substructure modifications should also be provided. Preliminary design submittals for rehabilitation projects shall 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 deck replacement projects, existing structural steel stringer elevations shall be obtained (field verified) in order to establish proposed deck slab depths and the proposed bridge profiles. Composite design should be used as a method of increasing load carrying
SEMI-INTEGRAL ABUTMENT
(DIMENSIONS IN mm)

150 CLEAR

150

BRIDGE LIMIT

CONCRETE DECK SLAB

ELASTOMERIC BEARING PAD

CONSTR. JOINT

STEEL STRINGER

POLYSTYRENE

SLOPE PROTECTION

1200 MIN.

450 MIN.

150 MIN.

600

SUPPORT POST

900 WIDE NEOPRENE WATERPROOFING MATERIAL, CENTERED ON JOINT

CONSTR. JOINT

POROUS BACKFILL W/FILTER FABRIC

150 PERFORATED CORRUGATED POLYETHYLENE DRAINAGE PIPE, TYPE SP

ø PILES-PLACE PILE WEB PERPENDICULAR TO ø BEARING.

Figure 205
2-32
capacity of existing beams or girders when replacing the concrete deck of a bridge which was originally designed and constructed non-composite.

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

207 BRIDGE GEOMETRICS

207.1 MINIMUM VERTICAL CLEARANCE

The minimum vertical clearances shall be in accordance with ODOT’s Location and Design Manual, Section 302.

Minimum preferred vertical clearance is 5200 mm on interstate highway system.

Minimum preferred vertical clearance is 4600 mm on state highway system.

207.2 BRIDGE SUPERSTRUCTURE

Minimum preferred bridge width is 9600 mm face to face of guardrail or toe to toe of parapet on the state highway system.

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 9000 mm from the edge of traveled lane to the point where the 2:1 backslope intersects the radius at the toe of the 2:1 slope. Refer to the 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 9000 mm from the near edge of traveled lane.

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, soft converted into metric, in millimeters, should be labeled as (+/-) plus or minus.

207.5   BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION

During Line, Grade and Typical (LGT) development the location of the structure should be studied to attempt to eliminate the presence of excessive skew, curves or extreme superelevation transitions within the actual bridge limits.

208   MAINTENANCE OF TRAFFIC

208.1   TEMPORARY STRUCTURES

During the preliminary design stage the profile grade, alignment and width of the temporary structure should be established and shown on the Site Plan. All other preliminary design parameters must be determined in conjunction with the development of the preliminary design. See Section 500, Temporary Structures for details.

208.2   STAGE CONSTRUCTION

A transverse section(s) defining all stages of removal and construction should be provided with the preliminary design submittal. Plan views and preliminary working drawings shall be provided to ensure constructability. The following additional information should also be provided with the preliminary design submittal:

a. The existing superstructure and substructure layout with overall dimensions (field verified) and color photographs.

b. Type of temporary railing or barrier and anchorage (if necessary) to super-structure.

c. Proposed temporary lane widths, measured as the clear distance between temporary barriers, shall be shown. A temporary single lane width of 3350 mm or greater is preferred; 3000 mm is the minimum allowable. Minimum preferred lateral clearance from edge of lane to barrier is 500 mm [L & D manual section 502.14] but Section 502.22, L & D manual, allows this lateral distance to be amended for specific sites and conditions. The designer should assure at the LGT that lane and lateral clearance requirements are evaluated versus effects of phased construction on a bridge structure.

d. Sufficient details should be provided to determine the clearance between existing and proposed construction stages. Check for clearances to temporary sheeting if required for the abutments.
e. 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.

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.

Concrete single span slab bridges should be screened 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.

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.

Existing exposed reinforcing steel may be used to confirm the direction of the reinforcing steel.

208.3 TEMPORARY SHORING

Whenever shoring is required to support a roadway where traffic is being maintained and the height of the exposed shoring will be over 2.5 meters, the Design Agency shall be required to provide a temporary shoring design with details provided in the plans and feasibility studied during the preliminary design stage.

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 3.5 meters in height. Design computations are necessary.

b. For cuts greater than 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 cannot 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 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 of equivalent soil height as a surcharge.

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

- Minimum section modulus
- Top and minimum bottom elevation of shoring
- Limits of shoring
- Sequence of installation and/or operations.
- Method of payment
- If bracing or tiebacks are required, all details, connections and member sizes shall be detailed.
- A general note in plans allowing a Contractor designed alternate for temporary shoring.

209 MISCELLANEOUS

209.1 TRANSVERSE DECK SECTION WITH SUPERELEVATION

If the change in cross slope at the superelevation break point is less than 3 percent then no rounding is required. For changes greater than 3 percent the rounding 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 Figure 206 - Page 2-37

B. When the roadway break point is located at the edge of pavement (adjacent shoulder width of 1.8 meters or less) the bridge cross slope is to be continued past the break point to the toe of deflector parapet. See Case b, Figure 206 - Page 2-37

C. When the roadway break point is located at the edge of pavement (adjacent shoulder width is more than 1.8 meters and less than 3.0 meters) a 1.2 meter rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.0417 meter per meter shoulder cross slope. See Case c, Figure 207 - Page 2-39
CASE a.

CASE b.
D. When the roadway break point is located at the edge of pavement (adjacent shoulder width is 3.0 meters or more) a 1.5 meter rounding distance from the edge of pavement onto the shoulder is used to transition from the bridge cross slope to the 0.0417 m/m shoulder cross slope. See ‘Case d, Figure 207 - Page 2-39

The transition from the roadway approach transverse section to the bridge deck transverse section is to take place within the limits of the approach roadway.

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. See Figure 208 - Page 2-40. A table with the information shown in Figure 208, page 2-40, is also acceptable. Where this is not practicable, then transition’s discontinuities should be smoothed by inserting 15 meter roundings at each discontinuity.

209.2 BRIDGE RAILINGS

As of October 1, 1998 all bridge structures on the National Highway System will require the use of crash tested railing meeting the loading requirements of TL-3 as defined by NCHRP report 350. For structures with over the side drainage on the National Highway System, Twin Steel Tube Bridge Guardrail, Standard Drawing TST-1-98M 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 7.6 meters or more above the lowest groundline or normal water, concrete deflector parapets should be used.

Refer to Section 305 for vandal protection fencing requirements.

209.3 BRIDGE DECK DRAINAGE

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 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 considered during the preliminary design phase.
Figure 208
2-40
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. Where the pavement flow from the deck is less than 0.014 m³/s a sodded flume should be provided. (Roadway Standard Drawing DM4.1M) 2m of excelsior matting shall be placed on each side of the flume. On grade separation structures with 2:1 approach embankment slopes and where the pavement flow from the deck exceeds 0.014 m³/s, integral curb shall be provided on the approach slab with a standard catch basin (Roadway Standard Drawing C3-A) catch basin located off the approach slab in lieu of the sodded flume. A 12 diameter type F pipe shall be used to provide an outlet down the embankment slope and the outlet shall be armored.

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 601.06. 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 insure 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 601.05.

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 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 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. In general the length of the approach slab should be based on various factors such as the extent of the new embankment (limit the length where the bridge replacement will be on existing embankments), the ability to span the embankment, skew of the bridge and design average daily traffic. 9000 mm or longer approach slabs shall have special embankment considerations. For four lane divided highways on new embankment, 7600 mm minimum (normal to the abutment) approach slabs shall be used. Refer to Standard Drawing AS-1-81M for approach slab details.

Roadway plans should provide detail drawings for approach slabs which differ from standard approach slab drawings, such as approach slabs which are tapered, having a non-uniform width, or other such variation. These detail drawings should be submitted to the Department for review during the detail design review stage.

209.6  PRESSURE RELIEF JOINTS

Type A pressure relief joints shall be specified when the approach roadway pavement is rigid concrete. The Type A pressure relief joint shall be placed at the location of the first transverse pavement joint but not greater than 15 meters from the approach slab. When an integral or semi-integral design is used and the approach pavement is rigid concrete a Type A pressure relief joint shall be specified at the end of the approach slab. Where a pressure relief joint is specified at the end of the approach slab, the dowels shown at that location on Standard Drawing AS-1-81M are not required and a plan note should be provided to eliminate the use of those dowel bars.

A new pavement standard for pavement pressure relief joints will be issued in October 1998. The tentative standard can be used as a plan insert sheet for projects until the standard is issued.

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.

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:

1. horizontal and vertical clearances for both the proposed design and during construction,

2. the constructability of the substructure units adjacent to their tracks,

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

4. location of railway utilities, and

5. 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 ODOT concurrent 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 A588M 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 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 3.1 meters above the top of rail, except where a pier is located within 3.6 meters of the centerline of tracks and in that instance the minimum height should be 3.6 meters above the top of rail. The crash wall shall be at least 760 mm thick. For a cap and column pier the face of the wall shall extend 300 mm beyond the face of the columns on the track side. 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 7.0 meters. The point of minimum vertical clearance should be from a point 1.8 meters 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 4.25 meters horizontal measured from centerline of tracks and 6.7 meters vertical, measured 2.0 meters from centerline of tracks, wherever possible.

### 209.9 BICYCLE BRIDGES

Reference should be made to "Policy and Procedures for Bicycle Projects" available thru ODOT's Contract Sales. 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 .021 sloped in one direction should generally be used. Bicycle railings should be a minimum of 1370 mm high. A smooth rub rail should be provided at a height of 1065 mm. For the design of the railing refer to AASHTO Section 2.7.2. 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.

### 209.10 PEDESTRIAN BRIDGES

Pedestrian facilities generally shall provide access by means of ramps with a gradient not to exceed 8.33 percent and with level platforms at 9 meter intervals and at turns for purposes of rest and safety. For pedestrian bridges over highways an additional 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 32 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. The width of the bridge sidewalk is generally the width of the approach sidewalk plus 300 mm, with the widths typically between 1500 and 1800 mm wide.

An 0.021(1 to 48) cross slope should be provided to drain the sidewalk towards the curbline. The maximum sidewalk height shall be 200 mm 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 305 for vandal protection fencing requirements.

209.12 MAINTENANCE AND INSPECTION ACCESS

Consideration should be made during the preliminary design phase for maintenance and inspection access. For multiple span bridges with 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 contact the Office of Structural Engineering before the detail plan stage for details and recommendations. Additional information is provided in "FHWA Guidelines for Providing Access to Bridge", dated November 1985.
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, structural efficiency and reduces long term maintenance to a minimum level.

301.2  DESIGN METHODS

Ohio Department of Transportation bridge designs are to be developed in general conformance with AASHTO’s Standard Specification for the Design of Highway Bridges, latest editions including all interims. Exceptions to AASHTO standards are documented in this Manual.

For design of Temporary Structures see Section 500 of this Manual.

301.3  LOADING REQUIREMENTS

Highway Structures:

MS18 and Alternate Military Loading ** *(Case I or Case II)

* Applies to Steel Structures

** Including liveload factor increase of 25%

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

Pedestrian and Bikeway Bridges:

To be designed in accordance with AASHTO, "ODOT Policy and Procedure for Bicycle Projects", latest edition, and the following:

Bridges which cannot accommodate vehicles because of narrow roadway or walkway widths or other access limitations shall be designed based on the AASHTO Guide Specification for Pedestrian Bridges.

Bridges whose width can accommodate service vehicles shall be designed based on the AASHTO Guide Specification for Pedestrian Bridges and an M13.5 vehicle.

Railroad Bridges:

Bridges are to be designed in accordance with current AREA specifications and the individual railway company’s loading requirements. All other aspects of the structure design shall conform to AASHTO.
301.3.1 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 .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:

- Connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to .20 times the dead load reaction force in the restrained direction. The restrained direction for an expansion bearing is transverse to the structure.

- 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:

\[
N \text{(in mm)} = 203 + 1.67L + 6.67H
\]

Where:

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

For Abutments

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

(single span bridges, \( H = 0 \))

For columns and/or piers

\( H = \text{column or pier height, in meters} \)

For hinges within a span

\( H = \text{Average height of the adjacent two columns or piers, in meters.} \)

301.4 REINFORCING STEEL

Reinforcing steel - ASTM A615M, A616M or A617M, Grade 420, \( F_y = 420 \text{ MPa} \)

All reinforcing steel shall be epoxy coated.

301.4.1 MAXIMUM LENGTH

Generally maximum length of reinforcing steel should be 12 200 mm. This limit is for both transit purposes and construction convenience. The maximum length before a lap splice is required is 18 400 mm.

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 2300 mm, and in no case greater than 2450 mm.
301.4.2 BAR MARKS

Bar marks shall be used on detail plans to identify the bar’s position and to reference the bar to the reinforcing bar list and description. The actual size of reinforcing steel bar shall be part of the bar mark.

Example:
[16M bar - bar mark - A16M01]

Letters should be incorporated into the bar marks to help identify their location in the detail plans: Some examples are "A" for abutments, "P" for piers and "S" for superstructure.

Example:
[A16M01]

Spiral reinforcing should use the letters "SP" to identify its bar mark.

Example:
[SP16M01]

Straight bars used in drilled shafts shall include the letters "DS" in their bar mark.

Example:
[DS16M01]

A note or legend within the bar list sheet in the plans shall describe each bar mark’s meaning.

See Figure 301 - Page 3-4

301.4.3 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 No. 43M and 57M bars.

Splicing of reinforcing by welding is not permitted.
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 15M01 IS A #15 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.

SPACERS
Concrete spacers or other approved noncorrosive spacing devices shall be used at sufficient intervals.THE BOTTOM AND AT INTERVALS NOT EXCEEDING 3050 mm TO INSURE CONCENTRIC SPACING FOR THE ENTIRE CAGE LENGTH. SPACERS SHALL BE CONSTRUCTED OF APPROVED MATERIAL EQUAL IN QUALITY AND DURABILITY TO THE CONCRETE SPECIFIED FOR THE SHAFT. THE SPACERS SHALL HAVE ADEQUATE DIMENSIONS TO ENSURE A MINIMUM 75 mm CLEAR SPACE BETWEEN THE OUTSIDE OF THE REINFORCING CAGE AND THE DESIGN DIMENSION OF THE DRILLED SHAFT OR COLUMN. CYLINDRICAL CONCRETE FEET (BOTTOM SUPPORT) SHALL BE PROVIDED TO ENSURE THAT THE BOTTOM OF THE CAGE IS MAINTAINED AT THE PROPER DISTANCE ABOVE THE BASE.
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 302 - Page 3-6

For Compression Splice lengths See Figure 303 - Page 3-7

For Development Length Requirements for Reinforcing Steel, See Figures 303, 304 & 305 - Pages 3-7, 3-8, 3-9

301.4.4 CALCULATING LENGTHS AND WEIGHTS OF REINFORCING

Reinforcing steel lengths shall be calculated to the nearest 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 spiral finish, the additional 1-1/2 coils of spiral required at the end of the spiral, AASHTO 8.18.2.2.4, should be calculated and included. For a 765 mm diameter spiral with 16M bars the additional weight of 3 coils is 11.0 kg.

See Figure 306 - Page 3-10 for area, weight and diameter of standard reinforcing.

See Figure 307 - Page 3-11 for bar bending data.

See Figure 308 - Page 3-12 for standard bar length deductions of common bends.

301.4.5 BAR LIST

Bar lists should include the following:

- Bar Mark
- Number of bars required
- Overall length required of the bar
- Total Weight for each bar mark
- Column for type of bar:
  "ST" for straight
  "Number" assigned to
  "Numbered Bent Bar Detail"
  "Number" and "Series" for series type 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:

- core diameter
- pitch
- mark
- number
- height
- weight
- plan note for spiral bars

See Figure 301 - Page 3-4
# TENSION SPLICES (mm)

<table>
<thead>
<tr>
<th>BAR SIZE</th>
<th>EPOXY</th>
<th>NON-EPOXY</th>
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</thead>
<tbody>
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<td>LOCATION</td>
<td>TOP</td>
<td>OTHER</td>
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<td>1</td>
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<td>3350</td>
</tr>
<tr>
<td>36</td>
<td>4340</td>
<td>4110</td>
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</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% ($x0.80$) (8.25.3.1), BUT NOT LESS THAN 300 mm PER 8.32.3.1.

4. VALUES SHOWN ARE FOR CLASS “C” LAP WITH $f'c = 28$ MPa. AND $f_y = 420$ MPa (8.32.3.2)

* BAR DIAMETER
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 conform to 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 8.32.4.i, but in no case less than 300 mm.
### Development Length for Reinforcing Steel (mm)

<table>
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<tr>
<th>BAR NO.</th>
<th>BAR TYPE</th>
<th>TENSION REINFORCEMENT (mm)</th>
<th>COMPRESSION MIN. LENGTH</th>
<th>WITHIN SPIRAL</th>
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<td>MOD. FACTOR LENGTH</td>
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<tr>
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<td>510</td>
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</tr>
<tr>
<td>16</td>
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*(SEE NOTES FIG. 3.05)*
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 DEVELOPEMENT LENGTH SHALL BE $\geq 200$mm (8.26)

6. VALUES SHOWN ARE FOR CLASS "C" LAP WITH $f'_c = 28$ MPa AND $f_y=420$ MPa (8.32.3.2)
<table>
<thead>
<tr>
<th>BAR SIZE DESIGNATION</th>
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<th>NOMINAL WEIGHT kg/m</th>
<th>NOMINAL DIAMETER (mm)</th>
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BENDING TOLERANCES: Refer to Section CMS 509
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<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
301.4.6 USE OF EPOXY COATED REINFORCING STEEL

All reinforcing steel shall be epoxy coated except as noted for prestressed beams in Sections 302.5.1.7 and 302.5.2.8.

All approach slabs shall have epoxy coated reinforcing steel.

301.4.7 MINIMUM CONCRETE COVER FOR REINFORCING

The clearances of reinforcing steel from the face of the concrete shall be as follows:

- Top reinforcing steel in bridge decks and sidewalks - 65 mm - (including 25 mm monolithic wearing surface)
- Bottom reinforcing steel in bridge decks - 40 mm
- Bottom steel of footings - 75 mm
- Column steel or spirals - 75 mm
- All other concrete - 50 mm

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

301.4.8 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 265 mm$^2$ per meter in each direction.

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

301.5 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.

301.6 UTILITIES

Utilities should not be supported on the facia 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.

Utility conduits embedded in concrete should be shown and dimensioned so as to clear construction joints by a minimum of 25 mm and other conduits by a minimum of 50 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", Sections 8.10, page 14, and 8.20, page 23.

301.6.1 UTILITIES ATTACHED TO BEAMS AND GIRDER

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

Critical utility lines (gas, etc.) which 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.

301.7 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

302.1.1 CONCRETE SUPERSTRUCTURE

Concrete Design Allowables

Superstructure Concrete - Class S
Load Factor Design - 31.0 MPa
### Service Load Design - .33 x 31.0 MPa = 10.3 MPa

Substructure Concrete - Class C
Load Factor Design - 27.5 MPa

Service Load Design - .33 x 27.5 MPa = 9.2 MPa

### 302.1.1.1 SUPERSTRUCTURE CONCRETE REQUIREMENTS

#### 302.1.1.1.a CLASS S SUPERSTRUCTURE CONCRETE

Class S Concrete is the Department’s standard concrete mix design for superstructure concrete. The mix design, curing and placing requirements are defined in the C & MS.

#### 302.1.1.1.b HIGH PERFORMANCE CONCRETE

High performance concrete, may be specified for use on bridge structures.

High performance concrete mix designs are intended to give a highly dense, very impermeable concrete mix resulting in a longer structure life.

High performance concrete is defined in Supplemental Specification 844 which specifies the construction, materials, mix designs, placement, curing and testing requirements for this type concrete.

### The high performance concrete Supplemental Specification 844 incorporates concrete mix designs for both superstructure and substructure concrete.

### The bid item for testing of high performance concrete shall be included in the plans only when required by the District.

### High performance concrete should not be used as a replacement for the drilled shaft concrete as specified in 524.10.

#### 302.1.1.2 WEARING SURFACE

#### 302.1.1.2.a TYPES

- 25 mm monolithic concrete - defined as the top 25 mm of a concrete deck slab. This 25 mm thickness shall not be considered in the structural design of the deck slab or as part of the composite section.
• 64 mm - minimum asphaltic concrete wearing surface to be used only on non-composite prestressed box beams.

32 mm minimum of Item 448 Asphalt Concrete Surface Course, Type 1H

32 mm thickness of Item 448 Asphalt Concrete Intermediate Course, Type 1, PG64-28

• Cast in place deck thickness of composite prestressed box beams shall be a minimum of 155 mm. Also see Section 302.5.1.2

302.1.1.2.b FUTURE WEARING SURFACE

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

No future wearing surfaces shall be considered as included in the design deck thickness in the analysis and design of the superstructure.

302.1.3 CONCRETE DECK PROTECTION

3.2.1.3.a TYPES

• Epoxy Coated Reinforcing Steel - CMS 709.00

• Minimum concrete cover of 65 mm

• Class S concrete

• High Performance Concrete

• Cathodic protection systems

• Drip Strips

• CMS 512, Type D, Membrane waterproofing or CMS 512 Sheet type waterproofing, Type 3

• Asphaltic concrete wearing surface.

• Liquid coatings, sealants and treatments (for vertical deck surfaces subjected to roadway drainage).

302.1.1.3.b WHEN TO USE

• All reinforcing steel shall be epoxy coated.

• All cast in place concrete decks shall have minimum concrete top cover of 65 mm.

• A drip strip shall be used on any decks with over the side drainage. See Bridge Standard Drawing DS-1-92M.

• Cathodic protection systems are considered to be experimental and are to be approved by the Office of Structural Engineering. Approval will be limited to special cases as conditions warrant.
• Non-composite box beam bridges, with over the side drainage, shall have an asphalt concrete overlay. The overlay shall be placed over either a waterproofing membrane, Type D, CMS 512 or sheet type waterproofing, Type 3, CMS 512. Minimum thickness of overlay is 64 mm - See section 302.1.1.2.a

302.1.1.3.c SEALING OF CONCRETE SURFACES SUPERSTRUCTURE

Specifications for sealing material are defined in a Departmental proposal note. Concrete surfaces shall be sealed with an approved concrete sealer as follows:

For bare concrete decks with over-the-side drainage; the exterior 230 mm width on the top of the deck, the deck facia and a 150 mm (minimum) width under the deck shall be sealed with either an epoxy-urethane or non-epoxy sealer.

On decks with curbs and/or sidewalks and/or parapets a 230 mm width of the roadway along the curbline, the vertical face of curb, the top of the curb and/or sidewalk, the inside face, top an outside face of the parapet, the deck facia, and a 150 mm (minimum) width under the deck shall be sealed with either an epoxy-urethane or non-epoxy sealer.

For decks with concrete deflector parapets a 230 mm width of the roadway along the face of parapet, the inside face, top and outside face of parapet, the deck facia and a 150 mm (minimum) width under the deck shall be sealed with either an epoxy-urethane, or non-epoxy sealer.

For non-composite prestressed concrete box beam decks with over-the-side drainage, the facia of the outside beams and a minimum 150 mm width under the beam shall be sealed with a non-epoxy sealer.

See Figures:

309 - Page 3-18
310 - Page 3-19

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 also has the alternative to just use a bid item for sealer, with no preference, and allow the contractor to choose based on cost.
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 309
3-18
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
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 not less than the larger of \( \frac{S + 5200}{36} \) or 215 mm. \( S \) being the effective span length in millimeters.

The 25 mm wearing thickness, section 3.2.1.2.a, 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 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.

For continuous slabs on three or more supports a continuity factor of .80 shall be applied to the simple span bending moments for both live load and dead load.

See Figures 311, 312, 313 & 314 - Pages 3-22, 3-23, 3-24, 3-25 for an illustration of a method of design for a reinforced concrete deck slab.

Upon completing the concrete deck design from the example above, 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 - Pages 3-26 & 3-27

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

302.2.3 SCREED ELEVATIONS

Screed elevations shall be furnished to ensure the bridge deck is completed at its correct elevation, including the gutters or edges of deck on a bridge without curbs.

The detail structure plans shall include a diagram or table showing the elevations at the top of the concrete deck that are required before the concrete is placed. Elevations should be shown for both curblines and crown of the roadway and above all steel beam or girder or prestressed I beam lines for the full length of the bridge, at all bearings and at a maximum of 10,000 mm intervals. Elevations
at mid span are optional and need be shown only for short spans where the nearest 10 000 mm point might be some distance from the point of maximum deflection. Elevations at splice points will be required.

Cases of special geometry, i.e. spirals, horizontal or vertical curves, superelevation transitions, etc., will require additional elevation points to define the concrete deck screed elevations. A sufficient number of screed elevations must be provided so the contractor is not forced to interpolate or make assumptions in the field.

The designer shall furnish all elevation points to allow the proper construction and finishing of the deck.

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 box beam bridges.

302.2.4 REINFORCEMENT, i.e. SIZE, LENGTH, SPACING

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 #13M. The only exception to this requirement is if the bar spacing becomes less than 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). The additional reinforcement shall be uniformly spaced and furnished in length equal to 40 percent of the length of the longer adjacent stringer span. This reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 1000 mm longitudinally.

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, the top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.
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 MS-18 loading.

\[ S = 2900 \text{ mm minus } 150 \text{ mm} = 2750 \text{ mm} \]

\[ T_{\text{min}} = \frac{(5182 \times S)}{36} = 220.3 \text{, use } 220 \text{ mm} \]

\[ f'_c = 31 \text{ MPa} \]

\[ f_y = 420 \text{ MPa} \]

\[ \phi = 0.9 (8.16.1.2.2) \]

\[ Z = 23,000 \text{ kN/m (top steel)}, \]

\[ 30,000 \text{ kN/m (bottom)} (8.16.8.4) \]

\[ n = 8 \]

\[ \text{Slab} = \left( \frac{T_{\text{min}}}{1000} \right)^2 \frac{(2403)(9.81)}{5.186 \text{ kN/m}} = 8.056 \text{ kN/m} \]

\[ \text{FWS} = 2.870 \text{ kN/m} \]

**Design Moments:**

\[ \text{DLM} = (0.125)(W)(S^2)(x)(0.8) = (0.125)(8.056)(2750^2)(8)/(1000^2) = 6.092 \text{ kN/m (3.24.3.1) } \]

\[ \text{LLM} = (S+610)(72)(1.3)(0.8)/(9.74)(1000) = (3360)(72)(1.3)(0.8)/9.74(1000) \]

\[ = 25.83 \text{ kN-m (3.24.3.1) } \]

\[ M_u = 1.3[Dl+1.67(LL)] = 1.3(6.092+1.67(25.83)) = 63.999 \text{ kN-m (3.22) } \]

\[ M_w = \text{Service load moment} = \text{DLM} + \text{LLM} = 31.923 \text{ kN-m} \]

### Top Reinforcement

<table>
<thead>
<tr>
<th>R = Mu/(\phi bd^2)</th>
<th>Bottom Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>= (63.999)(1000)/(0.9)(1)(134)^2</td>
<td>R = (63.999)(1000)/(0.9)(1)(147)^2</td>
</tr>
<tr>
<td>= 3.96 MPa</td>
<td>= 3.291 MPa</td>
</tr>
<tr>
<td>(\rho = 0.001022, \text{ from chart} )</td>
<td>(\rho = 0.00837, \text{ from chart} )</td>
</tr>
<tr>
<td>As = (0.001022)(1000)(134) = 1370 \text{ mm}^2/m</td>
<td>As = (0.00837)(1000)(147) = 1230 \text{ mm}^2/m</td>
</tr>
</tbody>
</table>

Try 16M bars at 145 mm in (As=1379)

### Check steel spacing (8.16.8.4)

\[ d_c = 50 + 13.8 = 71 \]

\[ A = 71 \times 2 \times 145 \]

\[ f_s (\text{all.}) = \frac{z}{(d_c A)^{1/3}} \]

\[ f_s (\text{all.}) = 23,000/[7(11)(2)(145)]^{1/3} = 202.7 \text{ OR } < 0.6(420) = 252 \text{ kN/m}^2 \]

\[ f_s (\text{act.}) = \frac{M_w}{As} \text{ Jd} \]

\[ f_s (\text{act.}) = (31.923)(1000)/[(1379)(0.90)(134)] = 191.95 \text{ kN/m}^2 (O.K.) \]

**Spaced @ 145 mm**

**16M bars @ 145 mm c/c (As=1379)**

**Top and bottom bars shall coincide based on B.M.D. Section 305.15.**
EXAMPLE: Stringer spacing of 2900 mm

S = 2900 mm minus 150 mm = 2750 mm

From chart: T = 220 mm, As (top) = 1379, As (bott.) = 1250

PRIMARY REINFORCEMENT: (See Design Regulations)

Use #16M bars (top and bott.), both at 145 mm c/c, As=1379

DISTRIBUTIONAL REINFORCEMENT:

As (top): (0.33)(1379) = 455 mm²/m

Use #13M bars at 10 equal spaces (As = 469*)

As (bott.): \( \frac{121}{\sqrt{57000}} = 73.06\% \), use 67% max. \( (3.24.10.2) \)

= (0.67)(1379) = 924 mm²/m in mid-half of span

= (0.50)(924) = 462 mm²/m in each outer quarter \( (3.24.10.3) \)

Use 7 #16M bars at 240 mm c/c in mid-half of span and 2 #16M bars at 360 mm c/c in each outer quarter.

* AASHTO

* By load factor procedures. For design data and sample problem, see Fig. 311.
Slab Thickness "T" (mm)

Area of Steel "As" (Sq. mm per m)

Effective Span "S" (m)

As, Top of slab
(d=T-91 mm)

As, Bottom of slab
(d=T-48 mm)
STRENGTH DESIGN OF CONCRETE FLEXURAL MEMBERS
SINGLY REINFORCED

\[ f'c = 31 \text{ MPa} \]
\[ f_y = 420 \text{ MPa} \]
\[ R = f_y \left( 1 - 0.5 \frac{f_y}{0.85f'c} \right) \]

Max. = (75) \% (P_b) = 0.0233

NOTE:
AASHTO minimum reinforcement requirements as per art. 8.17.1
(and art. 8.20 if applicable) must be checked.

SAMPLE PROBLEM (See Figure 311)

\[ M_u = 63.999 \text{ kN-m} \]
\[ \phi = 0.90 \]
\[ b = 1000 \text{ mm (1 m)} \]
\[ d = 131 \text{ mm} \]
\[ R = \frac{M_u}{b d^2} = \frac{(63.999)(1000)/(0.9)(1)(131)}{2} \]
\[ R = 4.144 \text{ MPa} \]
\[ p_s = (0.01022), \text{from chart} \]
\[ A = p b d = (0.01022)(1000)(134) \sim 1369 \text{ mm}^2/\text{m} \]
CANTILEVER SLAB DESIGN

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-18 loading.

\[ P_1 = 72 \text{kN} \quad (3.24.3) \dagger \]
\[ P_2 = 45 \text{kN} \quad (2.7.1.3) \dagger \]

**Truck Load Distribution Factor:**
\[ E_1 = 0.8 \times x_1 + 1.143 \quad (3.24.5.1.1) \dagger \]
\[ E_1 = 0.8 \times (0.15) + 1.143 = 1.263 \]

**Railing Load Distribution Factor:**
\[ E_2 = 0.8 \times x_2 + 1.524 \quad (3.24.5.2) \dagger \]
\[ E_2 = 0.8 \times (0.765) + 1.524 = 2.136 \]

\[ C = 1 + \frac{h - 838}{457} \quad (2.7.1.3) \dagger \]
\[ C = 1 + \frac{915 - 838}{457} = 1.17 \]

**Uniform Dead Load:** (per m of length)
- Slab: \( w_1 = (0.255)(23.56) = 6.01 \text{kN/m} \)
- F.W.S.: \( w_2 = 2.87 \text{kN/m} \)

**Concentrated Dead Load:** (per m of length)
- Parapet: \( P = 6.86 \text{kN} \) (located @ CG)

**Dead Load Moment:**
\[ DLM = \frac{1}{2} w_1 L^2 + \frac{1}{2} w_2 (L - 0.46)^2 + P (L - 0.15) \]
\[ DLM = \frac{1}{2} (6.01)(23.56)^2 + \frac{1}{2} (2.87)(0.915 - 0.46)^2 + 6.86(0.915 - 0.15) = 8.06 \text{kN-m} \]

**Live Load Moment:**
- Truck Load Moment: \( TLM = 1.3 \times x_1 \frac{P_1}{E_1} \)
- Railing Load Moment: \( RLM = h \frac{C P_2}{E_2} \)

**Check Moment:**
- \( M_u = 1.3 \left[ DLM + 1.67 \text{LLM} \right] \)
- \( M_u = 1.3 \left[ 8.06 + 1.67 (22.55) \right] = 59.43 \text{kN-m} \)

**Check Moment:**
- \( M_w = \text{Service Load Moment} - \text{DLR} + \text{LLM} \)
- \( M_w = 8.06 + 22.55 = 30.61 \text{kN-m} \)

CHECK TOP REINFORCEMENT \( \dagger \) AASHTO
f′c = 31 MPa
f_y = 420 MPa
φ = 0.9 \text{ (8.16.1.2.2)†}
Z = 23.0 \text{ MN/m (top steel) (8.16.8.4)†}

R = M_u / φbd^2 = 59.43 / 0.9 (1) (0.166)^2 = 2.396 MPa
ρ = 0.0063 \text{ (from chart)}
A_s = ρbd = 0.0063(1)(0.166)(1 \times 10^6) = 1044 \text{ mm}^2/\text{m} < 1481 \text{ mm}^2/\text{m} \star \text{ ok}

Check Steel Spacing (8.16.8.4)†

\[ f_s (\text{ALL}) = \frac{Z}{(d_c A)^{1/3}} \]
\[ f_s (\text{ALL}) = \frac{23.0}{((0.071)(2)(0.071)(0.145))^{1/3}} = 202.7 \text{ MPa} \]

\[ f_s (\text{ACT}) = \frac{M_u}{A_s \bar{d} d_t} \]
\[ f_s (\text{ACT}) = \frac{30.61}{0.001379 (0.91)(0.166)(1000)} = 147 \text{ MPa} < 202.7 \text{ MPa \ ok} \]

★ - Steel reinforcing ratio for top steel taken from Transverse Slab Design example (#16M bars @ 145mm).

† AASHTO
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.

For prestressed I-beams or composite box beam bridge decks transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

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.

### 302.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam or girder or prestressed I beam structures shall have a design concrete haunch of 50 mm. The minimum 50 mm haunch depth shall be measured from the highest top flange point and the haunch shall be tapered back to the original concrete deck thickness in a 225 mm length. The concrete haunch shall encase the edges of the top flange.

See Figures 317 & 318 - Page 3-29 & 3-30

### 302.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, are not recommended for use.

### 302.7 CONCRETE PLACEMENT SEQUENCE

Placement sequences are not generally detailed for standard steel beam or girder bridges but are left to the contractor. The designer should recognize the need for a pour sequence is not limited to long structures with an intermediate expansion device. Other possible structure types are bridges with end spans less than 70 percent of internal spans, two span structures, structures whose size eliminates one continuous pour, etc.

Placement sequences are required for continuous deck prestressed I beam bridges. Standard Drawing PSID-1-95M defines a sequence of deck placement for a prestressed I beam bridge.

### 302.8 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.
TYPICAL CONCRETE DECK HAUNCH DETAIL
TYPICAL CONCRETE DECK HAUNCH DETAIL

Figure 318
3-30
An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

302.2.9 STAGED CONSTRUCTION

For steel beam and girder bridges and prestressed I-beam bridges where the dead load deflection is greater than 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 800 mm but should be wide enough to accommodate the required reinforcing steel lap splices.

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) in accordance with Supplemental Specification 846. The sealing width should be 600 mm, centered on the construction joints.

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 the Strength Design Method, AASHTO’s Table 3.22.1A L+I (Live load and Impact) beta factors shall be increased by a factor of 1.25.

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 two-thirds (2/3) the slab bearing width but not more than the clear span plus the slab thickness.

Simple span reinforced concrete slab bridge superstructures should conform to Standard Drawing SB-6-94M. The reinforcing steel required will need to be changed by the designer to comply with the change in Strength Design beta factors listed in the paragraph above.

Multi-span reinforced concrete slab bridge superstructures should conform to Standard Drawing CS-1-93M. The reinforcing steel required will need to be changed by the designer to comply with the change in Strength Design beta factors listed in the paragraph above.
302.4 STRUCTURAL STEEL

302.4.1 GENERAL

Composite or non-composite steel beam and girder superstructures shall be designed using Strength Design Method (Load Factor Design). 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.

For the Strength Design Method, AASHTO’s Table 3.22.1A L+I (Live load and Impact) beta factors shall be increased by a factor of 1.25. This increase in beta factors is not to be used in applying the overload provisions of AASHTO Section 10.57.

All curved beams or girders shall be designed in accordance with AASHTO, this Manual and the 1993 AASHTO Guide Specifications for Horizontally Curved Highway Bridges, including Addenda.

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.

While the final design of a steel stringer bridge with top flanges embedded in a concrete deck assumes the laterally unsupported length to be considered zero, the designer shall investigate whether, during construction and erection, the supported length will be exceeded by non-composite erection and deck placement stresses.

302.4.1.1 MATERIAL REQUIREMENTS

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

- High strength, low alloy, (A572M) steel, painted, shall be the primary selected steel.

- Unpainted A588M weathering steel should generally be used for stream crossings and roadways over railroads.

- If unpainted A588M weathering steel is used, a 3000 mm length of the beam or girder adjacent to abutments with expansion joints and on both sides of intermediate expansion joints shall be painted with the IZEU paint system. All cross frames, end frames or any other steel in these 3000 mm sections shall also be painted. The top coat shall be tinted to a color to closely match Federal Standard No 595a - 20045 or 20059 (the color of weathering steel).
• Painted A572M steel should be used in industrial areas, corrosive areas, in overpasses or where subjected to accumulations of dirt or debris or exposed to continuous moisture.

• A36M painted steel may be considered if deflection constraints or strength controls do not require the use of the higher strength steels.

• For galvanized structures A-572M steel should be used.

For A-588M steel, CMS 513 and 863 requires outside and bottom surfaces of facia beams or girders to be blast cleaned to grade Sa 2 after concrete has been placed. This is to help eliminate major blast cleaning problems on the construction site. It does not alleviate the final requirements of CMS 513.221. The pay item will become an "As per plan" pay item.

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 CERTIFICATION

CMS 501.04 requires structural steel for bridges, exclusive of expansion joints, bearing devices and secondary and detail material, be fabricated by AISC (American Institute of Steel Construction) certified fabricators. Recent changes in AISC have created new levels of fabricators.

AISC Category SBr - fabricators qualified for simple bridge structures

AISC Category Mbr - fabricators qualified for all other bridge structures.

AISC has also established a P and F endorsement for fabricators.

P - Painting of steel structures endorsement

F - Fracture Critical endorsement.

The Department has established a new Supplemental Specification 863 for structural steel. This specification establishes seven (7) levels of fabricator qualification using the new AISC fabricator qualification levels.
When modifications or repairs to a steel structure involve only small amounts of main material, (cross frames, brackets, etc.) requiring little or no welding with only a minimal amount of fabrication, such steel, furnished under SS 863, should not require AISC Certification but be defined for the Miscellaneous level. By using this appropriate SS 863 pay item the AISC requirement will be waived.

302.4.1.4 MAXIMUM AVAILABLE LENGTH OF STEEL MEMBER

The rolling mills have established certain maximum lengths of rolled beams. Mills currently rolling beams can supply up to 35,000 mm, but an extra cost is added for lengths over 24,300 mm. The designer should consider this limitation and provide for field splices and allow for optional field splices.

Length of a girder is generally limited by the ability to transport the member from the fabricator’s shop to the jobsite. A length of 36,000 mm for girders is generally the maximum length between splices but girder lengths of 49,000 mm and greater have been transported to project sites.

302.4.1.5 PAINTING OF STRUCTURAL STEEL

As fabrication shops can generally apply a superior paint coat, new structural steel, requiring painting, should have the prime coat shop applied.

This requirement also applies for the applications of A588M weathering steel where only the 3000 mm sections at the end of the structures or at expansion devices are to be painted. (Section 302.4.1.1)

302.4.1.5.a TYPES OF PAINT SYSTEMS

The recommended coating system for new structural steel is the IZEU paint system (Inorganic Zinc, Epoxy, Urethane).

The IZEU paint system should be used for the 3000 mm partial painting requirement at expansion devices of new unpainted A588M steel superstructure applications.

New steel members being added to widen existing structures should be painted with IZEU.

If the total existing structure is to be field painted, the existing members should be painted with the OZEU system (Organic Zinc, Epoxy, Urethane). The OZEU system is better for field application as the prime coat is more forgiving.

Supplemental Specifications 815, for OZEU, and 816, for IZEU, are available.
302.4.1.5b PAINT SYSTEMS, 3 COAT SHOP SYSTEMS

Complete application of all three (3) coats of paint at the fabrication shop has been performed on some projects. Selected projects have been those where field access is limited, such as the waterline close to the bottom of the structure, and large structures. The Office of Structural Engineering has plan note/special proposal note for three (3) shop application of IZEU. (See Appendix)

302.4.1.6 STEEL PIER CAP

Steel pier caps are fracture critical members. If there is no other alternative solution, the preliminary details of the connection should be reviewed and approved by the Office of Structural Engineering before completion of the detail plans. Structure designs which 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 with AASHTO Section 3.23.2.3.1.4.

In order to facilitate forming, deck slab overhang should not exceed 1200 mm. Over the side drainage structures the minimum overhang shall be 700 mm. Where scuppers are required for bridge deck drainage the overhang shall be 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, at a maximum of 10 meter intervals. Points at midspan and at splices shall also be detailed in the deflection and camber table.

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 and horizontal curve adjustment and 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.

302.4.1.9a 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.

For service loadings the number and position of traffic lane units shall conform with the AASHTO specifications, Section 3.11.2.

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. For service loadings the appropriate fractions S/5.5 or S/7 shall be used to determine live load bending moments. (See AASHTO Section 3.23.2)

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.

302.4.1.9b STRESS CATEGORY

In order not to preclude 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: W920 x 223 (CVN) ]

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

Cross frame members and cross frame connection stiffeners 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 1200 mm. Standard expansion joints have designs already established as part of the standard drawings. For suggested details for special conditions review existing expansion joint Standard Drawings.

302.4.1.12 BASELINE REQUIREMENTS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES

CMS 513 & SS 863 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 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 on Standard Drawing BS-1-93M, sheets 1 thru 4. This Standard incorporates Load Factor designed beam splices for A36M, A572M or A588M 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 1.5 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 shall be based on AASHTO's values for Class A, Contact Surface, Standard 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 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 0.8 mm. The designer should recognize this definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

**302.4.14.a BOLTS**

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A-325M.

While the design must specify the size of bolt, the designer should not specify on detail plans the requirements defining which type, I or III, of A325M bolt to use. These requirements are covered in the CMS material specifications for A325M bolts.

If a paint system with a zinc based prime coat, OZEU, IZEU or System A, is to be used, the bolts should be galvanized.
Specifications require the galvanized bolts for IZEU and System A paints. A plan note will be required to call for galvanized bolts if an OZEU paint system is specified for coating new steel. ASTM A325M specifies galvanized A325M bolts to be type I; therefore the note does not need to define type.

If a structure is un-painted A588M, CMS 711.09 requires the use of bolts with weathering steel characteristics. A325M specifies that A325M, type III bolts meet that requirement. No plan note is required.

Generally, bolted splices should be designed using 25 mm diameter bolts.

The use of A490M bolts is not permitted.

302.4.1.14.b EDGE DISTANCES

When 25 mm diameter bolts are to be used splice plates should be detailed to allow for 50 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 an additional 6 mm to the AASHTO minimum edge distance shall be used.

302.4.1.14.c LOCATION OF FIELD SPLICES

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

302.4.1.15 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. 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 50 mm deep haunch over the top flange will have an effect on the length of shear studs.

302.4.2 ROLLED BEAMS

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 13,700 mm in length. Therefore, beam lengths should not be longer than 18,500 mm. Before a design is completed, the designer should confirm with local galvanizers that the structural members detailed can be galvanized by a local plant.

Bolted splice designs will require oversized holes because standard holes partially fill with galvanizing. Bolted crossframes will be required due to field installation issues. Standard Drawing GSD-1-96M has bolted cross frame details that may be specified.

Field welding of cross frames is not an alternative to bolting because welding into galvanizing causes damage to the coating and there is not a quality touch-up system developed to adequately handle the repair of damaged galvanizing for the large quantity of repairs which would be required.

302.4.2.2 STIFFENERS

Intermediate stiffeners shall only be used when required for cross frames. Stiffeners shall be minimum 10 mm thickness and wide enough to make an adequate and easily accessible cross frame connection. Stiffeners 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 25 mm horizontally and 65 mm vertically. Dimensions are shown on STD DRG. GSD-1-96M

Both sides of the stiffener shall be fillet welded to the beam web and both flanges. See STD DRG GSD-1-96M.

302.4.2.3 INTERMEDIATE CROSS FRAMES

Cross frames connections should be to the webs or intermediate web stiffeners.

Generally cross frames shall not be spaced more than 7500 mm.

See Standard Drawings GSD-1-96M for standard cross frame configurations.

Cross frames should be perpendicular to stringers and be in line across the total width of the structure. Horizontal legs of cross frame angles should align on both sides of a beam.

For structures with flared stringers, cross frames should be attached at the same relative position on both sides of the web. Cross frames should also be positioned as follows:

- For flared structures with a differential angle between individual stringers of 5 degrees or less, the cross frames should be perpendicular to one stringer.
- For flared structures with a differential angle between individual stringers of greater than 5 degrees the actual differential angle should be divided evenly between connections to both stringers.
• The designer should show the typical crossframe details and the maximum spacing in the plans. Actual spacing of the crossframes should be left to the structural steel fabricator’s detailer to allow for designer errors and fabrication fit-up. If a design requires specific location of crossframes those crossframe lines should be marked in the plans to not allow adjustment.

A detail showing a completely bolted connection for cross frame to the steel member is shown in Standard Drawing GSD-1-96M.

Holes for erection bolts are normally provided in the connection of cross frames to stiffeners. Provide 20 mm holes in cross frames and 18 mm holes in stiffeners for 16 mm bolts. See GSD-1-96M.

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 cross frames 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" beams, cross frames should be placed near the bend points. The cross frames should be located approximately 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 300 mm clearance dimension and should be connected to the adjacent stringer only on the same side of the centerline of beam 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 - Page 3-42 for plan view layout of cross frames for dog-legged beams.

Cross frames for curved beams may be similar to the above design but the designer is required to confirm that the members and their connections meet the additional loadings developed in a curved member design.

Both doglegged beam cross frames at the dogleg or curved beam cross frames shall be connected to the beam by use of welded stiffeners.

302.4.2.4 WELDS

CMS 513 & SS 863 permits welding by the following processes:

- Shielded Metal Arc Welding (SMAW)
- Flux Cored Arc Welding (FCAW)
- Gas Metal Arc Welding (GMAW)
- 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.
PLAN VIEW
Crossframes for Dog-legged Splices

Figure 319
3-42
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.

302.4.2.4a MINIMUM SIZE OF FILLET WELD

Fillet welds shall be designed for required stresses but should also meet the following size requirements:

- Minimum size of fillet weld is based on the thickness of the thicker steel section in the weld joint. The minimum size of fillet weld is defined by AWS D1.5.
- 6 mm leg for up to 19 mm thick material
- 8 mm leg for greater than 19 mm material

302.4.2.4.b NON-DESTRUCTIVE INSPECTION OF WELDS

Non destructive testing (NDT) of welds is defined in CMS 513 & SS 863. 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 will be required in the plans defining any special requirements.

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. Details for moment plates should be approved by the Office of Structural Engineering.

302.4.3 GIRDER

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 locatized point of failure for the coating system. 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.

In applying the above practices, consideration should also be given to the availability of plate lengths. Plates should not be extended beyond the lengths which 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 which requires fracture critical members.

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 which 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.

A plan note shall be added requiring all FCM’s to be subject to the AASHTO/AWS D1.5, chapter 12, Fracture Control Plan for Fracture Critical Non-redundant Steel Bridge Members, latest edition, and welding requirements of CMS 513 and/or 863.

If a girder is non-redundant, include the entire girder in the fracture critical steel pay quantity. The designer’s required plan note shall state which parts of the girder, and which welds, are actually subject to the fracture control plan.

### 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:

\[
W = \frac{d_w}{6} + 65
\]

Round up to the next 25 mm

\[d_w = \text{web depth, mm}\]

\[W = \text{flange width, mm}\]

But not less than 300 mm width.

The minimum thickness for any girder flange shall be 22 mm.

Generally, selection of flange thicknesses should conform to the following:

- 22 to 76 mm thick - in even 2 mm increments.
- Greater than 76 mm thickness - in 5 mm increments.
Whenever possible use constant flange widths throughout the girder.

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:

- A36M steel
  135 kg + (0.0175 kg X cross sectional area, in mm$^2$, of the lighter flange plate).

- For 345 MPa steels, A588M & A572M, the cutoff point is 85 percent of the value for A36M steel.

302.4.3.3.b WEBS

The minimum web thickness shall be 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 minimum 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 facia girders.

Stiffeners shall be provided for the attachment of cross frames and both ends of the stiffener shall be welded to the flanges to help eliminate cracking of the web due to out of plane bending. Both sides of the stiffener shall be fillet welded to the girder web and compression flange. Cross frame stiffeners shall be welded to the girder web and both flanges. The designer must investigate that fatigue criteria is met in these areas.

Stiffener plates shall have corners in contact with both web and flange clipped. The clip dimensions shall be 25 mm horizontally and 65 mm vertically.
302.4.3.5 INTERMEDIATE CROSS FRAMES

Connection of cross frames should be made to the intermediate web stiffeners. This is normally done by field welding.

Generally cross frames shall not be spaced at more than 7500 mm.

See Standard Drawing GSD-1-96M for standard cross frame 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. Then the cross frames can be permanently attached and a deck closure pour can be completed to finish the superstructure. See Section 302.2.9.

On skewed structures, when the differential dead load deflection of adjacent girders at any intermediate cross frame location is 13 mm or more, the erection bolt holes in the cross frame members should be detailed as 40 x 18 mm long slotted holes for 16 mm diameter bolts. The slotted holes shall require a 8 mm thick structural plate washer placed over them. Final bolting, and the welded connection, shall not be completed until after the deck concrete has been placed.

For curved or flared bridges with "dog-legged" girders, cross frames should be placed near the bend points. The cross frames should be located approximately 300 mm from the bend point so as not to interfere with the splice material. The cross frame should be placed normal to the girder used to set the 300 mm clearance dimension and should be connected only to the stringer on the same side of the centerline of girder splice. They should be similar to standard cross frames with an additional horizontal angle near the top flange of the stringers.

For curved or flared bridges with "dog-legged" girders, cross frames should be placed near the bend points. The cross frames should be located approximately 300 mm from the bend point so as not to interfere with the splice material. The cross frame should be placed normal to the girder used to set the 300 mm clearance dimension and should be connected only to the stringer on the same side of the centerline of girder splice. They should be similar to standard cross frames with an additional horizontal angle near the top flange of the stringers.

See Figure 319 - Page 3-42 for plan view layout of cross frames for dog-legged members.

Cross frames for curved girders may be similar to the above design but the designer is required to confirm the members and their connections meet the additional loadings added due to curved member designs.

302.4.3.5.a ERECTION BOLTS

For plate girder bridges, erection bolts shall be provided in the connections of cross frames to girder stiffeners. All bolt holes in stiffener and cross frames should be detailed as 4 mm larger than the erection bolts. Erection bolts are normally 16 mm diameter.

302.4.3.6 WELDS

CMS 513 & SS 863 permits welding by the following processes:
• Shielded Metal Arc Welding (SMAW)

• Flux Cored Arc Welding (FCAW)

• Gas Metal Arc Welding (GMAW)

• 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.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:

- Minimum size of fillet weld is based on the thickness of the thicker steel section in the weld joint. The minimum size of weld is defined in AASHTO/AWS D1.5.
  - 6 mm leg for up to 19 mm thick material
  - 8 mm leg for greater than 19 mm thick 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 which 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:

\text{CS} \quad \text{- indicates butt weld subject to compressive stresses only.}

302.4.3.6.c \quad \text{INSPECTION OF WELDS, WHAT TO SHOW ON PLANS}

Non destructive testing (NDT) of welds is defined in CMS 513 and/or 863. 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 50 percent ultrasonic inspection. (The designer should check the AREA specifications and with the actual railroad to confirm the individual railroad’s requirements for NDT of welds.)

302.5 \quad \text{PRESTRESSED CONCRETE BEAMS}

302.5.1 \quad \text{BOX BEAMS}

Physical dimensions and section properties of box beam cross sections shall be as shown on Standard Drawing PSBD-1-93M.

Box beams should be limited to a maximum skew of 30 degrees.

Box beam sizes should be limited so the total weight of transport vehicle and member do not exceed a maximum weight of 54 500 kg.

Multiple span box beam bridges shall be joined over the piers with a T-joint per Standard Drawing PSBD-1-93M. Structurally the beams shall be designed as simple spans.

Expansion at the piers shall be accommodated by elastomeric expansion bearings or by flexibility of the piers if integral design.

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 12 mm per joint between beams.

In order to keep the beam seat from extending beyond the facia 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 beam of 6 mm per joint.
For box beam bridges which 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 geometrics cause 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 25 mm joint filler, CMS 705.03. The designer should show both requirements in the plans. Not casting the backwall and wingwalls until after the box beams are erected allows the actual box beam fit-up height and width variances to not create installation problems with the joint filler and also help prevent spalling of the concrete backwall and wingwall due to movements of the elastomeric bearings.

For box beam bridges with steel railing, the post spacing and position of post anchorage shall be detailed on the plans. 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.

302.5.1.1 DESIGN LOADS AND LOAD DISTRIBUTION FACTORS

For both Service Load and Strength Design Methods, AASHTO’s Table 3.22.1A (Live load and Impact) beta factors shall be increased by a factor of 1.25.

For box beam members, the live load distribution factors of AASHTO Section 3.23.4.3 shall be used.

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.

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.

Prestressed box beam design data sheets for PSBD-1-93M are based on the use of low lax 12.7 mm (99 mm$^2$) strand. These design data sheets do not reflect the increase in beta factors under section 302.5.1.1.

Strands extended from a beam to develop positive moment resistance at pier locations shall not be debonded strands.

### 302.5.1.2.a TYPE, SIZE OF STRANDS

- Low-relaxation 12.7 mm diameter ($A_s = 99$ mm$^2$) seven wire uncoated strands, ASTM A416, Grade 270.
- Low-relaxation 12.7 mm diameter ($A_s = 108$ mm$^2$) seven wire uncoated strands, ASTM A416, Grade 270.

### 302.5.1.2.b SPACING

Strands shall be spaced at increments or multiples of 50 mm.

Location of the centerline of the first row of strands shall be 50 mm from the bottom of the beam. Rows of strands should also have a minimum clearance of 50 mm from the side of the beam and preferably 100 mm, if possible.

### 302.5.1.2.c STRESSES

Initial prestressing loads for low-relaxation strand shall be as per AASHTO requirements and shall be detailed on the plans.

Initial stress

$$\text{.75 } f_s = 1400 \text{ MPa}$$

Initial tension load

137 800 N/strand

Stress at release

$$\text{.69 } f_s = 1285 \text{ MPa}$$

Total losses expressed by

$$F_s = SH + ES + CRc + CRs$$

$$F_s = 77.07 + 18.0 f_{cfr} - 6.65 f_{cds}$$

### 302.5.1.3 COMPOSITE

Composite reinforced deck slabs on prestressed box beams shall be a minimum of 155 mm thick and shall be reinforced with No. 19M bars, spaced 450 mm longitudinally and 225 mm transversely.

On multiple span composite box beam bridges additional No. 19M longitudinal reinforcing steel over the piers shall be required. The No. 19M bars shall be alternately spaced with the standard longitudinal reinforcement and the pier bar's length shall be 40 percent of the longest adjacent span. The pier bars should be placed longitudinally and approximately centered on the pier but with a 1000 mm stagger.
For box beam structures with composite decks and concrete parapet and/or sidewalks with concrete raling the finished roadway width should not incorporate fit up tolerances. 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 50 mm to 200 mm each side with the fit-up being absorbed in the overhang. A mixture of 1220 mm and 915 mm boxbeam units may be necessary to meet this requirement.

Combining grades greater than 4 percent require a composite deck design.

### 302.5.1.4 NON-COMPOSITE WEARING SURFACE

Non-composite box beam bridges with asphalt overlays shall have either a waterproofing membrane as per CMS 512, type D, or a sheet waterproofing membrane, type 3, placed on the boxes before the 32 mm minimum layers of CMS type 448 asphaltic concrete is applied. (See section 302.1.1.2.a)

Non-composite box beam bridges with asphalt overlays shall be limited to a 4 percent combined grade. Combined grade includes both the longitudinal and transverse structure grades.

Combined Grade
\[ C_g = \left( \left( \text{deck slope} \right)^2 + \left( \text{transverse grade} \right)^2 \right)^{\frac{1}{2}} \]

The designer shall show a longitudinal superstructure cross section in the plans detailing the required thicknesses of the deck, deck components, or slab at centerline of spans, piers and abutments.
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.
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 Standard Drawing PSBD-1-93M.

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 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

BOX BEAMS

Design Concrete Strength of shop fabricated prestressed members = 38.0 MPa at 28 days.

Cast-in-place concrete, (composite decks, pier "T" sections, I- beam diaphragms, etc.) Class S superstructure concrete - 31.0 MPa at 28 days.

For concrete in composite decks see Section 302.1.1.1.

302.5.1.8 REINFORCING

Epoxy coated reinforcing steel shall be used in composite deck slabs and shall be Grade 420, Fy = 420 MPa.

Reinforcing steel used in the standard design box beams is Grade 420 Fy = 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.

Stirrups in boxbeams should completely enclose the majority of the prestressing strands.

302.5.9 TIE RODS

Tie rods shall be provided and installed as per Standard Drawing PSBD-1-93M.

Diaphragms and transverse tie rods for prestressed concrete box beam spans shall be provided at midspan for spans up to 15 000 mm, at third points for spans from 15 000 mm to 23 000 mm and at quarter points for spans greater than 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 drawing PSID-1-95M, shall be used.

Modified cross sections other than those on standard drawing PSID-1-95M should only be used after reviewing this option with the Ohio/Indiana/Kentucky Prestressed Concrete Institute and with approval of the Office of Structural Engineering.
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.

302.5.2.1 DESIGN LOADS AND LOAD DISTRIBUTION FACTORS

For both Service Load and Strength Design Methods, AASHTO’s Table 3.22.1A (Live load and Impact) beta factors shall be increased by a factor of 1.25.

Prestressed I-beam load distribution factors shall conform to AASHTO.

302.5.2.2 STRANDS

Preferably, straight strands should be used. Debonding of a maximum of 25% the number of strands in the I-beam, with an approved plastic sheath, may be done to relieve excessive stresses. More than 25% of the strands can be debonded if it can be shown that the applied moment is less than the cracking moment in the transition zone at ultimate load. The number of debonded strands in any horizontal row should not exceed 40% of the strands in that row.

Any use of draped strands shall first be approved by the Office of Structural Engineering.

The designer shall show on the detail plans the number, spacing and the length of required debonding per strand, if any.

Strands extended from a beam to develop positive moment resistance at pier locations shall not be deboned strands.

302.5.2.2.a TYPE, SIZE

- Low-relaxation 12.7 mm diameter \( A_s = 99 \text{ mm}^2 \) seven wire uncoated strands, ASTM A416, Grade 270, shall be used.

- Low-relaxation 12.7 mm diameter \( A_s = 108 \text{ mm}^2 \) seven wire uncoated strands, ASTM A416, Grade 270.

Other, larger diameter, sizes of strands may be specified with approval of the Office of Structural Engineering.

302.5.2.2.b SPACING

Strands shall be spaced at increments of 50 mm.

A minimum 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
\[0.75 f'_s = 1400 \text{ MPa}\]

Initial tension load
\[137 \text{ 800 N/strand}\]

Stress at release
\[0.69 f'_s = 1285 \text{ MPa}\]

Total losses expressed by
\[F_s = SH + ES + CR_c + CR_s\]
\[F_s = 77.07 + 18.0 f'_c + 6.65 f'_d\]

C = Adjustment for vertical curve. Positive for crest Vertical curve
D = Deflection due to dead load of slab, diaphragms, curbs, sidewalks and parapets.
E = Design Haunch Depth (minimum 50 mm)
F = Slab depth at beam bearings = \[A + B - C - D + E\]
If crest vertical curve correction [C] is greater than \([B-D]\) design slab thickness \([A+E]\) will control.
G = Anticipated slab depth at mid-span = \[A + E\]
If crest vertical curve correction [C] is \((B - D)\) Then \[G = A - B + C + D + E\]

302.5.2.3 CAMBER

A haunch shall be required in the deck to account for the I-beam's maximum camber due to design prestressing.

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:

\[A = \text{Design slab thickness}\]

\[B = \text{Anticipated total camber in the beam = established by design}\]

The designer shall show a longitudinal superstructure cross section in the plans detailing the required thicknesses of the deck, deck components, or slab at centerline of spans, piers and abutments.

302.5.2.4 ANCHORAGE

25M dowel anchors shall be provided at each fixed pier as per STD DRG PSID-95M

Minimum number of anchors shall be 2 for each beam line.
The anchors shall be a minimum of 600 mm long. Anchors shall be embedded a minimum 300 mm into the pier cap and 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 High Performance 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 12 000 mm.

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 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. Refer to standard drawing PSID-1-95M for typical diaphragm and reinforcing details.

Intermediate diaphragms of galvanized steel may be used if approved by the Department.

The designer shall add a note to the plans for prestressed I-beam designs requiring the intermediate diaphragms to be placed and cured at least 48 hours before deck placement.

I-beam designs should include threaded insert details for dowels/reinforcement to allow easier installation of cast-in-place diaphragms, to allow transfer of loads and to help protect the diaphragms against cracking. The threaded inserts and the threaded rods shall be galvanized as per 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 drawing PSID-1-95M 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 drawing PSID-1-95M establishes one sequence. The designer should either accept the standard drawing sequence or detail an alternative.

302.5.2.8 CONCRETE MATERIALS I-BEAMS

Design Concrete Strength of shop fabricated prestressed members = 38.0 MPa at 28 days.

Cast-in-place concrete, (composite decks, pier "T" sections, I-beam diaphragms, etc.) Class S superstructure concrete - 31.0 MPa at 28 days.

Higher concrete strengths may be specified with approval of the Office of Structural Engineering.

302.5.2.9 REINFORCING

Unless otherwise specified all reinforcing steel used shall be epoxy coated, Grade 420 Fy=420 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.

303 SUBSTRUCTURE

303.1 GENERAL

If a pier column, wall or other structural member is located in the sloped portion of an embankment, it shall be assumed that in addition to the earth pressure due to the embankment directly in back of the member, there also will be earth pressure due to the adjacent embankment on each side.

The minimum design earth pressure shall be 2.0 kPa unless granular backfill is provided.

303.1.1 SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Specifications for the sealer can be found in a proposal note. Concrete surfaces shall be sealed with a concrete sealer as follows:

The front face of non-integral 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.)
The exposed surfaces of all wingwalls and retaining walls, exclusive of abutment type, that are within 10 000 mm of any pavement edge shall be sealed with an epoxy-urethane sealer.

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.

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 of A-588M 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 allow the contractor to choose based on cost.

See Figures 321, 322 & 323 - Pages 3-59, 3-60 & 3-61

303.2 ABUTMENTS

303.2.1 GENERAL

Abutments should be provided with backwalls to protect the superstructure from contact with the approach earth fill and to assist in preventing water from reaching the bridge seat.

For members designed to retain earth embankments and are restrained from deflecting freely at their tops as in a rigid frame bridge, abutment walls keyed to the superstructure, and some types of U-abutments, the computed backfill pressure shall be determined by using at rest pressure.

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 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-2.3M.
SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 321
3-59
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
3-60
CROSSHATCHED AREA REPRESENTS THE CONCRETE SEALING LIMITS.

SEAL END OF WINGWALL ABOVE GROUND LINE.

SEAL ENTIRE SURFACE AREA

GROUND LINE

ABUTMENT SEALING LIMITS
(FOR INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

CROSSHATCHED AREA REPRESENTS THE CONCRETE SEALING LIMITS.

SEAL END OF WINGWALL ABOVE GROUND LINE.

SEAL ENTIRE SURFACE AREA

GROUND LINE

ABUTMENT SEALING LIMITS
(FOR SEMI-INTEGRAL ABUTMENT STEEL BEAM BRIDGE)

SEALING OF CONCRETE SURFACES, SUBSTRUCTURE

Figure 323
3-61
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 600 mm. Centerline of bearing should be 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 400 mm with centerline of bearing 225 mm from face of breast wall. The seat width for prestressed box beams with expansion devices is increased with centerline of bearing still 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 301.3.1 of this Manual.

### 303.2.1.2.a 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 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.
303.2.2 TYPES OF ABUTMENTS

303.2.2.1 ABUTMENTS, FULL HEIGHT

If the computed horizontal forces at the bottom of the full height abutment footing 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 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 No. 19M reinforcing bars attached near the top of each sheet pile and included with the sheet piling for payment. The No. 19M bars shall be long enough to be fully developed in bond.

If a 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 375 000 mm$^3$ 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 rustications may be provided at 1200 mm centers to fit the width of standard plywood forms or liners.

An alternate to vertical rustications is the use of form liners to provide the wall surface with an aesthetic appearance. While a variety of formliners are available the following criteria should be met:

- 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.
- 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.
- The cost of formliners selected should add only minimal additional cost to the overall cost of the concrete (1 to 3 percent per meter of the abutment, pier or wall)
- 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, FULL HEIGHT ABUTMENTS

For full height abutments exceeding 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 150 mm clearance between rows. Reinforcing steel splices should be staggered a minimum of 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, 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 Sheet Type 2 waterproofing, 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 with 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 1200 mm. If concrete slope protection is being provided the perpendicular distance required can be reduced to a minimum of 1000 mm.

The maximum spacing of piles in a single row or in the front row of a double row shall be 2500 mm.

For phased construction projects no abutment phase shall be designed to be 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 which 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 no abutment phase shall be designed to be supported on less than three (3) piles or two (2) drilled shafts.

303.2.2.5 ABUTMENTS, SPREAD FOOTING TYPE

Where foundation conditions warrant the use of an abutment on a spread footing, the bottom of the footing should be at least 1200 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 1200 mm. If concrete slope protection is being provided the perpendicular distance required can be reduced to a minimum of 1000 mm.

In no case shall the top of the footing be less than 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 which 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 - Page 3-66

The horizontal and vertical joint shall be sealed at the back face of the backwall by use of a 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. The sheeting shall be secured by the use of 32 x 3 mm (length x shank diameter) galvanized button head spikes at 225 mm C/C max. through a 25 mm outside diameter, 3 mm thick galvanized washer. A note for the neoprene sheeting is available in the section 600 and the requirements are also shown on standard Drawing ICD-1-82M.

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.
INTEGRAL ABUTMENT
(DIMENSIONS IN mm)

- CONCRETE DECK SLAB
- BRIDGE LIMIT
- 300 300 300
- 150 CLEAR
- 150 CM BEARING
- APPROACH SLAB
- 750 600
- CONSTR. JOINT
- PEJF
- 450 M/N.
- 300 M/N.
- LEVELING BOLTS
- 900 WIDE NEOPRENE WATERPROOFING MATERIAL, CENTERED ON JOINT
- CONSTR. JOINT
- POROUS BACKFILL W/FILTER FABRIC
- 150 PERFORATED CORRUGATED Polyethylene DRAINAGE PIPE, TYPE SP
- SLOPE PROTECTION
- 1200 M/N.
- 1
- 2
- 460 460
- 920
- Q PILES-PLACE PILE WEB PARALLEL TO Q BEARING.

Figure 324
3-66
For phased construction projects no abutment phase shall be designed 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.

Standard Drawing ICD-1-82M shows details for integral abutments with a steel beam or girder superstructure. Cantilevered or turnback wingwalls shall not be used with integral abutments.

303.2.2.7 SEMI-INTEGRAL ABUTMENTS

This abutment design is appropriate for bridge expansion lengths up to 80 meters or 125 meters total length of structure. 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 which have bend points in any stringer line.

The expansion length for a semi-integral structure is considered to be (2/3) two thirds 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 50 mm of 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 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.

Semi-integral abutment details are shown in standard drawing SICD-1-96M

See Figure 325 - Page 3-69

For phased construction projects no abutment phase shall be designed 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 ABUTMENT BACKWALL DRAINAGE

The porous backfill immediately behind abutments and retaining walls should be provided as per 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 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 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 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 600 mm thick for its full height behind the abutment and wingwalls except where the vertical backface of the abutment footing extends out 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). Segments should be connected by overlapping bands. Ends of runs, unless intended to function as outlets, should have end caps. While galvanized corrugated pipe
Figure 325
3-69
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 150 mm to 300 mm above normal water or ground line. The porous backfill with filter fabric behind the walls should be shown as extending at least 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 30 000 mm with the following exceptions:

- When the total length of wingwalls and breastwall exceeds 30 000 mm in length, vertical expansion joints should be provided just beyond each side of the superstructure.
When the length of a breastwall exceeds 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 REINFORCING STEEL

303.2.6.1 REINFORCEMENT, "U" AND CANTILEVER WINGS

The minimum amount of reinforcing in the wings, their junctions with the backwall and their supports shall be No. 16M bars on 450 mm centers, both horizontally and vertically, in both faces.

If a secondary member, such as a short cantilevered turnback 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 not attempt to add these requirements to a specific structure without confirmation from the Office of Structural Engineering's foundation section.

303.3 PIERS

303.3.1 GENERAL

A "free-standing" pier is defined as one which 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.

Height of pier is the distance from bottom of footing to 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.

For phased construction projects no pier phase shall be designed to be supported on less than three (3) piles, if a cap pile pier or two (2) columns if a cap and column type.

For a new or replacement structure, individual free standing columns without a cap are not permitted.

303.3.1 BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform with Standard Drawing CPP-2-94M. Also see Section 303.3.2.4 of this Manual.

Pier caps on piles, drilled shafts or on columns are normally a minimum of 915 mm wide. This is the standard width used for continuous span prestressed box beams and I-beams. Bearing seat widths of 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.

303.3.2 PIER PROTECTION IN WATERWAYS

See Section 200 of this Manual for piling projection requirements and Section 600 & 700 for plan notes to be added to design drawings.

303.3.2 TYPES OF PIERS

303.3.2.1 CAP AND COLUMN PIERS, CAP & COLUMN REINFORCEMENT

The cantilever arms of cap and column piers shall be designed for the same impact fraction as the superstructure. (See section 303.3.2.7)
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 915 mm diameter. Cap dimensions should be selected to meet strength requirements and to provide necessary bridge seat widths. Ends of caps shall be squared and cantilevered beyond the face of the end column to provide approximately balanced moments in the cap. Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap.

Minimum column diameters of 915 mm are generally used with spiral reinforcing. Spirals are made up of 16M diameter bars at 115 mm c/c pitch with a 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 with 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.

At least 3 spacers, devices to position the actual spiral to the required pitch, should be used for 765 mm diameter spirals. Spacers shall be equally spaced along the periphery of the spiral.

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 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 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 0.3 meter below ground level or 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 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 frostline.

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 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.

Exposed H piles and unreinforced concrete piles shall have pile protection. See description in Standard Drawing CPP-2-94M or a plan note is 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 distance from the edge of a concrete pier cap to the side of a pile shall be not less than 200 mm.

The diameter of the exposed portions of cast-in-place reinforced concrete piles generally should be 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-#19M reinforcing bars with a 300 mm outside diameter, #13M spiral, with a 300 mm pitch. The cage length should extend from the finished top of the pile to 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. This will require an as per plan note. The use of cast-in-place piles greater than 400 mm in diameter will require an increase in the width of the cap of Standard Drawing CPP-2-94M. See Section 303.3.3.
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 no pier or abutment phase shall be designed 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 (see sections in steel superstructure) which has historically shown both steel member and weld metal cracking problems.

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 box shall not be physically smaller that would limit access to the interior of the box. 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 assure that all governmental agency regulations as 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 which 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. If there is no other alternative solution, the preliminary details of the connection should be reviewed by the Office of Stuctural Engineering before completing the plans.

Designers should review all weld details for possible fatigue problems. Contact the Office of Structural Engineering for assistance in this area.

**303.3.2.7 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.
T type or wall type piers of large cross-sectional area and low unit stresses which are reinforced with a small percentage of longitudinal bars shall be provided with a nominal amount of lateral ties or hoops. In no case shall the concrete column contain less reinforcing than necessary to satisfy shrinkage, temperature and column reinforcement requirements of AASHTO.

303.3.2.8 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.

303.3.2.9 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.

303.3.2.10 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 450 mm into the concrete cap. Other substructure units on a single row of piles should have the piles embedded 600 mm into the concrete. A 300 mm embedment depth into the concrete footing is required for all other cases. In every case, there shall be at least 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 450 mm. The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 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:

For grade separation structures the top of footing shall be a minimum of 300 mm below the finished ground line. The top of footing should be at least 300 mm below the bottom of any adjacent drainage ditch.
The bottom of footing shall not be less than 1200 mm below, measured normal to, the finished groundline.

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 meters, shall be as computed by the following:

\[ D = T + 0.50Y \]

\[ T = \text{Thickness of footing (in meters)} \]

\[ Y = \text{distance from bottom of stream bed to surface of bedrock (in meters)} \]

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

Adjustment may be made to the minimum depth of the bottom of a footing due to actual frostline 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" may be taken as 0.55.

For coarse-grained soil with silt, "f" may be taken as 0.45.

For silt, "f" may be taken as 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 shall 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 which may constitute planes of weakness such as laminations or interbedding. 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, additional resistance shall be provided by one or more of the following means:
(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:

\[
\frac{A - C}{B - C} \geq 1.5 \text{ where,}
\]

\[
A = \text{available resistance to movement}
\]

\[
B = \text{force tending to cause movement}
\]

\[
C = \text{horizontal component of the axial load in battered piles}
\]

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 \( W_p \) (kN/m\(^3\)) for the undisturbed material encountered or anticipated. The equivalent passive fluid weight may be based on the following equation:

\[
W_p = W \tan^2 (45 + \phi/2) \text{ kN/m}^3
\]

where

\[
W = \text{unit soil weight, kN/m}^3
\]

\[
\phi = \text{angle of internal friction, in degrees.}
\]

For soft clays to coarse compact sand and gravels, \( W_p \) may vary from 15.7 to 125.7 kN/m\(^3\), respectively. For firm soils \( W_p \) may be taken as equal to 47.1 kN/m\(^3\). The total passive resistance \( P_p \) may therefore be based on the following equation:

\[
P_p = 0.5 W_p (H1^2 - h1^2), \text{ kN/m}
\]

where,

"H1" and "h1" are the effective depth and the surcharge depth respectively, both in meters. For structures without piles the effective depth "H1" 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 1.5 meters. The surcharge depth "h1" is the depth below ground surface affected by
seasonal changes or the depth of uncompacted backfill, whichever is larger. 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 No. 25M @ 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 No. 19M @ 300 mm centers.
At locations where the concrete unit tensile stress approaches the allowable for unreinforced 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, each pile for a structure shall be individually identified by a unique number. The designer may choose to number each pile on the individual substructure plan sheet or on a separate pile layout sheet.

303.4.2.2 PILES, NUMBER & SPACING

The designer shall comply with the following maximum center to center spacing of piles:

- In capped pile piers, 2300 mm.
- In capped pile abutments, 2500 mm.
- In stub abutments, front row, 2500 mm.
- In wall type abutments and retaining walls, front row, 2300 mm.
- 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.

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 shall 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 table below 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>HP250X62</td>
<td>500 kN</td>
<td>1000 kN</td>
</tr>
<tr>
<td>HP310X79</td>
<td>650 kN</td>
<td>1300 kN</td>
</tr>
<tr>
<td>HP360X108</td>
<td>850 kN</td>
<td>1700 kN</td>
</tr>
</tbody>
</table>

Design load values for H piles are based on a maximum service load stress of 62 MPa.

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>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>

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

A static load test item should be included in the structure’s estimated quantities if the actual calculated Ultimate Bearing load for the piles is 800 kN or more, except as follows:

- A static load test is not necessary if piles are driven to refusal on bedrock.
- A static load test is not necessary if the estimated linear meters of piles are less than 3000 meters.

Structures that require a static load test item may also require subsequent static load tests as defined in the following table:
When more than one type or size of pile is specified for a structure, the above criteria shall be applied independently for each pile type or size. If a project includes more than one structure, consideration should be given to reducing the number of static load test items, as generally many of the static load test items are non-performed.

### 303.4.2.6 PILES, DYNAMIC LOAD TEST

A dynamic load test item should be included in the structure’s estimated quantities if the actual calculated Ultimate Bearing load on the pile is 800 kN or more, except as follows:

- A dynamic load test is not required if piles are driven to refusal on bedrock.
- A dynamic load test is not necessary if the estimated linear meters of piles is less than 500 meters.

Structures that require a dynamic load test shall have an estimated pay quantity as listed in the following table.

<table>
<thead>
<tr>
<th>Estimated Lineal meters of Pile Type</th>
<th>Number of Subsequent Static Pile Test Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>3000-6000</td>
<td>1</td>
</tr>
<tr>
<td>6000-9000</td>
<td>2</td>
</tr>
<tr>
<td>9000-12 000</td>
<td>3</td>
</tr>
<tr>
<td>etc.</td>
<td>etc.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Estimated linear Meters of Piles</th>
<th>Estimated Pay Quantity in Hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>500-1500</td>
<td>3</td>
</tr>
<tr>
<td>1500-3000</td>
<td>6</td>
</tr>
<tr>
<td>3000-6000</td>
<td>9</td>
</tr>
<tr>
<td>6000-9000</td>
<td>12</td>
</tr>
<tr>
<td>9000-12 000</td>
<td>15</td>
</tr>
<tr>
<td>etc.</td>
<td>etc.</td>
</tr>
</tbody>
</table>

If more than one type or size of pile is specified for a structure, the above criteria shall be applied independently for each pile type and size.

### 303.4.3 DRILLED SHAFTS

1065 mm diameter drilled shafts for piers and 915 mm diameter for abutments are normally used.

The diameter of bedrock sockets of a drilled shaft are generally 150 mm less in diameter than the diameter of the drilled shaft above the bedrock elevation. The 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 #13M diameter bar at 115 mm pitch with spiral diameter of 150 mm less, out to out of spiral cage than the drilled shaft diameter. (Note AASHTO specifications do not recognize 115 mm pitch as meeting spiral requirements definition 8.18.2.2.3) When steel casing is left in place, a pitch of 300 mm should be used for the spiral reinforcing.

Drilled shafts with a diameter of less than 915 mm are not recommended.

The diameter of the drilled shafts should be 150 mm larger than the pier column diameter so that if the drilled shaft is slightly mislocated, the pier column can still be placed at plan location, although the pier column would not be exactly centered on a mislocated 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 misalignment 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 75 mm concrete cover within the pier column.

When the drilled shaft is socketed into the bedrock, the quantity of the reinforcing steel in the drilled shaft should be included with the item special "Drilled Shaft" 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.

A general note as listed in section 600 will be required.

The top of the drilled shaft is defined as 0.3 meter above normal water elevation, for piers in water, and 0.3 meter below the ground surface for piers not in water.
304 RAILING

304.1 GENERAL

All structures funded by FHWA are required to have railing systems which have been approved through actual crash testing of the railing design. Crash testing shall conform to the requirements of NCHRP document 350.

Projects on the National Highway System sold after October 1, 1998 are required to have crash tested permanent railing systems meeting or exceeding NCHRP 350 Testing Level 3 (TL3).

Provision should be made in metal railings for expansion and contraction. If standard railing connections will not allow for needed expansion and contraction movement, sliding devices need to be used.

304.2 TYPES OF RAILING

- Bridge Railing Deflector Parapet Type, New Jersey Shape - Standard Drawing BR-1M
  - NCHRP TL4 (915 mm)
  - NCHRP TL5 (1065 mm)

- Deep Beam Guardrail with Tubular Back-up - Standard Drawing DBR-2-73M (NCHRP TL 2)

- Bridge Retro-Fit Railing, Thrie Beam Bridge Railing for bridges with Safety Curbs - Standard Drawing TBR-91M

- Transition section for Bridge Deflector Parapet Type

- Bridge Sidewalk Railing with Concrete Parapets - Standard Drawing BR-2-82M (not crash tested. No TL level)

- Portable Concrete Bridge Barrier, Standard Drawing PCB-91M

- Twin Steel Tube Bridge Guardrail - Standard Drawing TST-1-98M (NCHRP TL4)

- Bridge Sidewalk Railing with Concrete Parapets - Standard Drawing BR-2-98M (NCHRP TL4)

304.3 WHEN TO USE

304.3.1 BRIDGE RAILING DEFLECTOR PARAPET TYPE

Bridge railing deflector parapet, New Jersey shape, shall be used on all highway & railroad overpass structures with no sidewalks.

Two (2) heights of bridge railing deflector parapet, New Jersey shape, are currently available in Standard Drawing form.
915 mm high deflector parapet. A 915 mm height is the new minimum height deflector parapet for two lane structures with an ADTT in one direction of less than 2500. The height increase over the old 813 mm high parapet is to allow for future overlays. Standard Drawing BR-1M gives details for the deflector parapet reinforcing, end transition section and transition to 813 high parapet.

1065 mm high deflector parapet. A 1065 mm height is the recommended deflector parapet height for Interstate, 4 lane divided or structures with an ADTT, in one direction of 2500. Standard Drawing BR-1M gives details for the deflector parapet reinforcing, end transition section and transition to 813 high parapet.

1270 mm high bridge railing deflector parapet shall be used in median areas where protection against oncoming headlight glare is required or to match roadway parapets. Median parapets should match the height of roadway parapets.

Bridge decks, which have concrete deflector parapets installed, shall be checked to confirm structural adequacy support additional railing and vehicle impact loads.

Concrete parapets, whether New Jersey shape or sidewalk vertical wall type, should be designed and detailed as follows:

- All horizontal reinforcing steel shall be detailed as continuous for the total length of the structure.
- Crack control joints shall be sawed into the concrete parapets. Distance between sawed joints on the structure shall be between 1800 and 3050 mm.
- The sawcut crack control joint should be detailed as 25 mm deep, and the joint filled with a caulking material, federal specification TT-S-00227E. This requirement is already established on many of the standard drawings. For special cases a plan note will be required. See section 600.
- Detail plans for structures with concrete parapets should include detailed locations of the crack control joints and vertical reinforcing bars.

Bridge structures with sidewalks should have one of the following:

- If bridge fencing is required a 813 mm high, vertically straight, 300 mm thick concrete parapet is recommended. If this fence and parapet configuration is used an aluminum railing as per Standard Drawing BR-2-98M is not required. See Figure 326 - Page 3-90.
- If no bridge fencing is required bridge sidewalk railing with concrete parapets, Standard Drawing BR-2-982M, shall be used.
304.3.2 DEEP BEAM RAIL WITH TUBULAR BACK-UP

This railing configuration is not recommended for use as the crash testing does not meet NCHRP TL3.

The designer should carefully review the position of posts that are near an obtuse corner of a skewed structure for possible interference of not just the anchor bolts but the back of the actual installed post with the wingwall.

304.3.3 TWIN STEEL TUBE BRIDGE GUARDRAIL

Rural bridges crossing a stream shall use the twin steel tube bridge guardrail system in accordance with Standard Drawing TST-1-98M.

If a bridge crossing a stream has its deck surface elevation greater than 7.5 meters above normal water elevation a 915 mm high bridge railing deflector parapet or sidewalk and parapet should be used in lieu of TST-1-98M.

The required bridge terminal assembly section between standard roadway single W beam guardrail and TST-1-98M is part of TST-1-98M.

The designer should confirm the stations to the centerline of the first posts off the bridge, shown on the site plan, are correct.

304.3.4 BRIDGE RETRO-FIT RAILING, THRIE BEAM BRIDGE RAILING FOR BRIDGES WITH SAFETY CURBS

Thrie-beam railing as described on Standard Drawing TBR-91M should only be used as a provisional upgrade on structures with safety curb and parapets where a safety upgrade is required but the structure will have a major rehabilitation or will be replaced in the near future.

This alternative is not generally recommended by the Office of Structural Engineering. A more suitable alternative is concrete refacing of existing safety curb and parapets to a New Jersey barrier shape. See Section 400 of this Manual for additional information on refacing of safety curb and parapets.
304.3.5 TRANSITION FOR BRIDGES DEFLECTOR PARAPETS

If deep beam guardrail from the roadway is to connect with bridge deflector parapets a transition section is required. The Department has standard drawings showing the transition section between the roadway deep beam guardrail and 813 mm high bridge railing deflector parapets.

The deflector parapet transition details should be used on both new structures and rehabilitated structures having concrete parapets added. Deflector parapet details can be used on either a structure’s turnback wingwalls, widened approach slabs or directly on the actual structure.

Pay item for barrier shall be item 622 - Portable Concrete Barrier, 813 mm, Bridge Mounted, As Per Plan - meters.

An alternate is the use of temporary twin steel tube bridge guardrail railing with posts anchor systems designed into the structure if the additional lane width gained is justified.

Although temporary railing shall be specified and completely described in the bridge plans, temporary railing is a roadway item and should be included in the roadway quantities.

304.3.6 PORTABLE CONCRETE BRIDGE BARRIER STANDARD DRAWING PCB-91M

All phased construction or rehabilitation which create a temporary no railing condition generally shall require Portable Concrete Bridge Barrier, PCB-91M, to be installed in accordance with the requirements of Design Data sheet PCB-DDM.

This railing configuration is not recommended for use as the railing, STD DRG BR-2-82M, has not been crash tested.

304.3.8 BRIDGE SIDEWALK RAILING WITH CONCRETE PARAPETS - BR-2-98M

Recommended for use on bridge structures with sidewalks of 200 mm height.
305 FENCING

305.1 GENERAL

The primary purposes of protective fencing are (1) to provide for the security of pedestrians and (2) to discourage the throwing or dropping of objects from bridges onto lower roadways, railroads, boat lanes or occupied property.

Fence may be needed on high level bridges where wind may threaten to blow pedestrians or even occasional stranded motorists off the bridge.

Also, on bridges where there is a danger that the outside parapet may be mistaken for a median barrier, persons may 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 Standard Drawing VPF-1-90M. The designer may need to enhance this standard to deal with requirements for the specific structure

305.2 WHEN TO USE

Pedestrian Fencing may be required when a total of 10 points or greater is achieved for a structure due to the following criteria.

The designer should use the following point 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 which qualify according to the total index number is not mandatory if adequate justification for not doing so can be furnished.

305.3 FENCING CONFIGURATIONS

For structures with sidewalks, the top of fence should be a minimum height of 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 2450 mm above the sidewalk before curving inward over the sidewalk. The overhang should be at least 300 mm less than the width of the sidewalk, with a maximum overhang of 1100 mm. The slope of the straight overhanging portion should be 1 vertical to 4 horizontal. The radius of the connecting arc should be 815 mm.

See Figure 326 - Page 3-90

For narrow pedestrian bridges, bent pipe frames are generally used with pipe bend radii of 600 mm at the upper corners and the start of the radii about 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.
### SECTION 300 DETAIL DESIGN-METRIC

**SEPTEMBER 1998**

<table>
<thead>
<tr>
<th>INDEX POINTS</th>
<th>JUSTIFICATION ITEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 a.</td>
<td>Overpass within an urbanized area of 50,000 or more population</td>
</tr>
<tr>
<td>2 b.</td>
<td>Overpass with sidewalks but not in an urbanized area as defined in(a.) (&quot;Sidewalk&quot; does not include safety curbs 685 mm or less in width.)</td>
</tr>
<tr>
<td>2 c.</td>
<td>Overpass which is unlighted.</td>
</tr>
<tr>
<td>2 d.</td>
<td>Overpass not a main thoroughfare, i.e., on collectors or local streets.</td>
</tr>
<tr>
<td>2 e.</td>
<td>Overpass within 0.8 km of another overpass exclusive of pedestrian bridges, having or requiring protection.</td>
</tr>
<tr>
<td>4 f.</td>
<td>Overpass within 0.8 km of another overpass having previous reports of falling objects.</td>
</tr>
<tr>
<td>4 g.</td>
<td>Overpass within 1.6 km of a school, playground or other pedestrian attraction.</td>
</tr>
<tr>
<td>4 h.</td>
<td>Bridges over any feature which has a high count of boat, rail, vehicular or pedestrian traffic, or includes damage-sensitive property.</td>
</tr>
<tr>
<td>6 i.</td>
<td>Overpass which has had prior reported incident of falling objects.</td>
</tr>
<tr>
<td>10 j.</td>
<td>Overpass which is used exclusively by pedestrians.</td>
</tr>
</tbody>
</table>

"OVERPASS" is a bridge over a highway or a railroad.

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 327 - Page 3-91 for an illustration of this configuration. To try to eliminate the adventurous youngster problem, some pedestrian bridges have used a frame design which comes to a peak at the center of the structure, similar to a house roof line.

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 standard drawing VPF-1-90M. The closure plate detail is required for all fence configurations which 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 esthetics 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 Standard Drawing VPF-1-90M may be referred to in the project plans.

3-89
BRIDGE WITH SIDEWALK - VERTICAL FENCE

300mm min. horizontal distance between face of curb and edge of fence.

305mm - 1092mm max. overhang

813mm R

2438mm +

813mm New

305mm

1524mm

BRIDGE WITH SIDEWALK - CURVED FENCE

Figure 326
3-90
PEDESTRIAN FENCING ON STRUCTURES

DEFLECTOR PARAPET WITH FENCING

Figure 327
3-91
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 Drawing VPF-1-90M.

For fence installation projects on new structures, the installation of a traffic railing (aluminum railing) is not required if the top concrete parapet or concrete wall is 813 mm above roadway for structures without sidewalks or 813 mm above the top of sidewalk for structures with sidewalks. See Figure 326 - Page 3-90

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 Drawing VPF-1-90M for development of required notes.

305.5 FENCE DESIGN

GENERAL REQUIREMENTS

Fencing mesh should consist of supported wire mesh of the chain-link variety with 25 mm diamonds. The core wire is to be 3.05 mm with a Polyvinyl chloride coating. (CMS 710.03)

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 pipe’s inside diameter.

305.5.1 LOADS

Wind Loads

High wind velocity: 50 years (1)

"or"

Maximum velocity: 129 km/hour at 9 meters above ground (1)

Wind pressure (kPa)

\[ P = 1.326C_h, \text{ derived from the formula:} \]
\[ P = 0.0471(1.3V)^2C_sC_h/1000 \] (2)

where

\[ V = \text{wind velocity (km/h)} \]

\[ 1.3 = 30\% \text{ gust factor} \]

\[ C_s = 1.0 \text{ (Shape constant for cylindrical shapes)} \]
C_h = height constant

<table>
<thead>
<tr>
<th>C_h</th>
<th>above terrain (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.08</td>
<td>0 - 4500</td>
</tr>
<tr>
<td>1.0</td>
<td>4500 - 9000</td>
</tr>
<tr>
<td>1.1</td>
<td>9000 - 15 000</td>
</tr>
<tr>
<td>1.25</td>
<td>15 000 - 30 000</td>
</tr>
<tr>
<td>1.40</td>
<td>30 000 - 46 000</td>
</tr>
<tr>
<td>1.50</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.

C_i = ice constant which shall be taken as unity.

PROJECTED AREAS for wind forces for polyvinyl chloride coated 3.05 mm core, 25 mm mesh, wire, use 20% of the gross horizontally projected area.

Additional area for posts, rails and other hardware need not be considered.

Ref. (1) Isotach’s of the U.S. The 129 km/h line covers the northwestern portion of Ohio and shall be used herein For all of Ohio.

Ref. (2) Specifications for the Design and Construction of Structural Supports for Highway Signs, AASHTO.

306 EXPANSION DEVICES

306.1 GENERAL

Expansion devices required should provide a total seal against penetration and moisture.

Expansion devices and their support systems such as end floor beams or end cross frames shall be designed for both MS18 Loading and 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 drawing.

To protect steel expansion devices metalizing of the exposed surfaces with a protective zinc coating shall be specified. Standard drawings define the requirements for metallizing. For special expansion devices, plan notes will be required to establish metallizing requirements.

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 513/863 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 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 Drawings EXJ-2-81M, EXJ-3-82M, EXJ-4-87M, EXJ-5-93M and EXJ-6-95M for sidewalk plates.

306.1.3 EXPANSION DEVICES; 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 capability, up to 5 mm. A plan insert sheet, Abutment Joints in Bituminous Concrete Box Beam Bridges, Metric, is available from the Department. This device requires two bid items, an item special and item 516.

306.2.2 ABUTMENT JOINTS AS PER AS-1-81M

A group of no or small movement joints used for sealing and rotational purposes is detailed on STD DRG AS-1-81M.

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 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 50 mm and maximum of 75 mm.
306.2.4 STRIP SEAL EXPANSION DEVICES

The seal size is limited to a 125 mm maximum. Unpainted A588M weathering steel should not be used in the manufacture of this type expansion device as A588M does not perform well in the atmospheric conditions an expansion device is subjected to. Standard Drawings, EXJ-4-87M, EXJ-5-93M and EXJ-6-95M, 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 100 mm. A 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 nor 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 Drawings EXJ-2-81M & EXJ-3-82M 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 100 mm, including movement in both directions. End cross frame support details shown on Retired Standard Drawing SD-1-69 can generally still be used with all steel supported expansion devices.

This is not a metric standard and is not for incorporation into project plans. Details, if used from this old standard should be converted and added to the actual plan sheets.

Sliding plates should be configured to preclude binding and bearing when the super structure is supported on elastomeric bearings.

Unpainted A588M 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. Use of modular devices requires approval of the Office of Structural Engineering.
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 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. Fabricator’s of modular devices shall be certified AISC, category appropriate for the work. 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

This is another alternate type of expansion device for structures where movements exceed the capacity of either strip or compression seal devices. Not generally recommended as this device has sealing, construction, fabrication, support and installation problems. Use of this type of device requires approval by the Office of Structural Engineering.
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.

Vulcanizing for sealing is recommended over adhesive sealing.

Finger devices shall be designed for fatigue and conform to fracture critical requirements if the design has fracture critical components in it.

Fabricator's of finger devices shall be certified AISC, category appropriate for the work. 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 Drawing AS-1-81M, Detail B.

306.3.2 REINFORCED CONCRETE SLAB BRIDGES

The below 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 (mm)</th>
<th>joint required</th>
<th>approach slab joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 12 000</td>
<td>None</td>
<td>AS-1-81M detail B</td>
</tr>
<tr>
<td>12 000 - 60 000</td>
<td>None (1) PM (2)</td>
<td>AS-1-81M detail B</td>
</tr>
<tr>
<td>60 000 +</td>
<td>PM</td>
<td>AS-1-81M</td>
</tr>
</tbody>
</table>

(1) = flexible abutments and piers (CPP-2-94M and CPA-5-94M)

(2) = abutments and/or piers fixed or rigid

PM = Polymer Modified Asphalt Joint

306.3.3 STEEL STRINGER BRIDGES

The below 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 (mm) | Joint required | Approach slab joint | Expansion length (mm) | Joint required | Approach slab joint
--- | --- | --- | --- | --- | ---
0-10 000 | None | AS-1-81M detail B | 0-12 000 | None | AS-1-81M detail B
10 000-38 000 | PM (t) or EXJ-4-87M | AS-1-81M detail B | 12 000-65 000 | PM (t) or EXJ-6-95M | AS-1-81M detail C
38 000-125 000 | EXJ-4-87M | AS-1-81M detail C | 65 000-150 000 | EXJ-6-95M | AS-1-81M detail C
125 000 + | TTED or MED | | 150 000 + | TTED or MED | 

PM = Polymer Modified Asphalt Joint

TTED = Tooth Type expansion device

MED = Modular Expansion Device

(t) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems.
### Expansion Joint Requirements for Composite Prestressed Concrete Box Beam Bridges

<table>
<thead>
<tr>
<th>Expansion length (mm)</th>
<th>Joint required (2)</th>
<th>Approach slab joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-12 000</td>
<td>PM</td>
<td>AS-1-81M detail C</td>
</tr>
<tr>
<td>12 000 - 65 000</td>
<td>PM (1) or EXJ-5-93M</td>
<td>AS-1-81M detail A, C, E</td>
</tr>
<tr>
<td>65 000-150000</td>
<td>EXJ-5-93M</td>
<td>AS-1-81M 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)

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### 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, the Office of Structural Engineering has specific proposal notes available. For specialized bearings the designer's detail plans should allow for possible different bearing heights and seat widths between manufacturer's bearings. The notes will require modification, by the designer, based on the specific structure.

307.2 BEARING TYPES

307.2.1 ELASTOMERIC BEARINGS

The design of elastomeric bearings shall conform to AASHTO. A design data sheet is available from the Office of Structural Engineering for elastomeric bearings for steel beam and girder bridges. Non laminated elastomeric bearings are only acceptable if actual design calculations support their use.

Elastomeric bearings should generally be limited to a 125 mm maximum elastomeric height excluding internal laminates with a minimum total height of 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. This maximum height may be waived by the Office of Structural Engineering.

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 150°C.

Elastomeric bearings with load plates, 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 38 mm is recommended but not mandatory.

Elastomeric bearings should not bear on unbonded steel surfaces. Therefore all steel plates in contact with an elastomeric bearings shall be vulcanized to the bearing.
Vertical deformation of the bearings greater than 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-1-55M

Generally, 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 Drawing RB-1-55M. The standard drawing also includes material and maximum load capacity requirements for this bearing type.

This bearing is limited to a 50 mm movement in one direction from the vertical.

The assumed rolling and sliding resistance of rockers is $0.25 \times DL \times r/R$, where: $DL$ is the dead load reaction on the rockers, $r$ is the radius of the pin, in mm, and $R$ is the radius of the rocker, in 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.

### 307.2.3 SLIDING BRONZE TYPE & FIXED TYPE STEEL BEARINGS

Generally, 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 type bearing is found on older steel beam or girder structures and is shown on Standard Drawing FSB-1-62. This standard is not currently active but copies are available through the Department. The fixed type bearing shown on Standard Drawing FB-1-82M, originated from the old Standard 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 POT TYPE BEARINGS

Generally pot type bearings are capable of high vertical loads and rotations up to 5 degrees, depending on the design. Included with Teflon sliding surfaces, they are capable of expansion movement.

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.

Justification must be provided for the use of Pot bearings. As a minimum this justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure.

Generic proposal notes for pot bearings are available through the Office of Structural Engineering. The notes will require modification, by the designer, based on the specific structure.

Pot bearings should 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.

Minimum vertical dead load needs to be 20% of total vertical load.

Design plans should take into account the possible different sizes and heights of different manufacturer’s bearings. Abutment and pier designs should accept these variances. This must be shown clearly on the plans so the contractor is informed. Final bearing height of the supplied bearing can be achieved by fabricating additional thickness in the bearing base plate to meet elevation requirements or allowing for adjustment in the bearing seat’s elevation. The plans should show the maximum adjustment allowed in the bearing seat’s elevation if this alternative is selected.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 6 mm.

This type of bearing generally accommodates the required horizontal movements by use of PTFE (Teflon) to stainless steel sliding surfaces. The proposal notes for this type of bearing also include requirements for the sliding surfaces and materials. The designer must be aware that Teflon to stainless steel friction factors vary with the loads applied. The lower the load the higher the friction factor.

307.2.5 DISC TYPE BEARINGS

Disc bearing are a special type "Proprietary" bearing developed for higher loadings, rotations and movements than standard type steel rocker, bolster or elastomeric bearings.
Because Disc bearings are "Proprietary", the designer must design and offer alternate bearings as a bidding substitute to comply with FHWA requirements. Generally the choice is Pot and/or Spherical type bearings. Pot bearings are the recommended alternate.

Justification must be provided for the use of Disc bearings. As a minimum this justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure.

Generic proposal notes for disc bearings are available through the office of Structural Engineering. The notes will require modification, by the designer, based on the specific structure.

Disc bearings should not be used with other bearing types.

Minimum vertical dead load needs to be 20% of total vertical load.

Design plans should take into account the possibility of different sizes and heights of various manufacturer’s bearings. Abutment and pier designs should accept these variances. This must be shown clearly on the plans so the contractor is informed. Final bearing height of the supplied bearing can be achieved by fabricating additional thickness in the bearing base plate to meet elevation requirements or allowing for adjustment in the bearing seat’s elevation. The plans should show the maximum adjustment allowed in the bearing seat’s elevation if this alternative is selected.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 6 mm.

This type of bearing generally accommodates the required horizontal movements by use of PTFE (Teflon) to stainless steel sliding surfaces. The proposal notes for this type bearing also include requirements for the sliding surfaces and materials. The designer must be aware that Teflon to stainless steel friction factors vary with the loads applied. The lower the load the higher the friction factor.

307.2.6 SPHERICAL TYPE BEARINGS

Spherical bearings are a special type of bearing developed for higher loadings, rotations and movements than standard type steel rocker or bolster bearings, elastomeric bearings or pot bearings.

Spherical bearings are not considered a proprietary type bearing.

Justification must be provided for the use of Spherical bearings. As a minimum this justification shall include calculations showing elastomeric bearings, with or without sliding surfaces, will not be adequate for the structure.

Generic proposal notes for spherical bearings are available through the Office of Structural Engineering. The notes will require modification, by the designer, based on the specific structure.
Spherical bearings should not be used with other bearing types.

Design plans should take into account the possibility of different sizes and heights of various manufacturer’s bearings. Abutment and pier designs should accept these variances. This must be shown clearly on the plans so the contractor is informed. Final bearing height of the supplied bearing can be achieved by fabricating additional thickness in the bearing base plate to meet elevation requirements or allowing for adjustment in the bearing seat’s elevation. The plans should show the maximum adjustment allowed in the bearing seat’s elevation if this alternative is selected.

Plans should require anchors for bearings to be set by use of a steel template with a minimum thickness of 6 mm.

This type of bearing generally accommodates the required horizontal movements by use of PTFE (Teflon) to stainless steel sliding surfaces. The proposal notes for this type bearing also include requirements for the sliding surfaces and materials. The designer must be aware that Teflon to stainless steel friction factors vary with the loads applied. 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 STANDARD DRAWINGS RB-1-55M OR FB-1-82M

These types of fixed bearings have been used in the past for steel beam or girder bridges.

Fixed bearings, Standard Drawing FB-1-82M, 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 degree skew and 20 meter deck width.

Bolster type fixed bearings (Standard Drawing RB-1-55M) 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 - STEEL BEAM BRIDGES

Fixed laminated elastomeric bearings are recommended for use on new steel structures, replacement steel structures or total superstructure rehabilitation.
A design data sheet for laminated elastomeric bearings, "Laminated Elastomeric Bearings for Steel Beam and Girder Bridges" is available. This design data sheet does not limit the size of elastomeric bearings.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Other 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 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 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 PRESTRESSED I-BEAM

Laminated elastomeric bearings shall be used for prestressed concrete I-beam bridges.

The Department has no standards for fixed or expansion bearings for prestress I-beam superstructure; 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, would 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
STANDARD DRAWING
RB-1-55M

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 degree skew, 20 meter deck width.

Rocker type expansion bearings 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.

Twin structures, being rehabilitated, which have RB-1-55M type bearings should not be tied together if overall finished width exceeds 20 meters. This bearing is not designed to accept transverse movement.

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 Drawing FSB-1-62 and normally used with FB-1-82M 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 20 meters. This bearing is not designed to accept transverse movement.

307.3.2.3 EXPANSION ELASTOMERIC BEARINGS BEAM AND GIRDER BRIDGES

This bearing type is recommended for use on new structures, replacement structures or total superstructure rehabilitation.

A design data sheet for laminated elastomeric bearings called "Laminated Elastomeric Bearings for Steel Beam and Girder Bridges" is available.

Elastomeric bearings should be designed based on selected durometer of either 50 or 60.

Other 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 20 meters laminated elastomeric bearings shall be required.

**307.3.2.4 EXPANSION ELASTOMERIC BEARINGS PRESTRESSED BOX BEAMS**

Box beam bridges shall have two elastomeric bearing pads at each end of each beam. At least a 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, 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 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, the Office of Structural Engineering has specific proposal notes available. Justification must be provided for the use of Pot bearings. As a minimum this justification shall include calculations showing elastomeric bearings will not be adequate for the structure. The notes will require modification, by the designer, based on the specific structure.

**307.3.3.1 POT BEARINGS**

See Section 307.2.4 for specific requirements on Pot bearings.
307.3.3.2 DISC TYPE BEARINGS

See Section 307.2.5 for specific requirements on Disc bearings.

307.3.3 SPHERICAL BEARINGS

See Section 307.2.6 for specific requirements on Spherical bearings.
 SECTION 400  STRUCTURE REHABILITATION AND 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.

For individual members, it will be necessary to determine whether the best option is to repair or replace. In making this decision, cost must 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. The Federal Highway Administration (FHWA) will only participate in a structure’s funding, whether repair, rehabilitation or replacement, once in any ten year period.

Due to the variation in the types of problems encountered, the designer must 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 must 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 which can be physically seen must 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 must be made to prepare plans which reflect the actual conditions in the field. However, it is recognized that uncertainties may exist. Consequently, note [42] found in Section 600 of the 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.

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, then permission to do so must be obtained from the Office of Structural Engineering. 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. This determination must be made by the designer. 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 ANALYSIS

A fatigue analysis of any existing steel members to be re-used or rehabilitated is required. It shall be performed in accordance with the method presented in the latest edition of the "AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges" (which gives a remaining fatigue life) and the method presented in the current "Standard Specifications for Highway Bridges" (which is based on allowable stress range). Neither of these methods produces absolute results. Rather they are useful as indicators of the relative severity of the fatigue detail. So they should both be evaluated along with any other pertinent information which could help in reaching a conclusion.

A fatigue analysis submittal shall be made to the Department, at the preliminary design stage, for final determination as to whether the members require fatigue related upgrading.
If the submission is made directly to the Department, any involved review consultant shall receive a copy of the transmittal letter. The Department’s comments shall be directed through the review consultant, if any.

The fatigue submittal shall include the following:

Method A  AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges

A table showing:

- Remaining safe and mean fatigue life
- Moments and stress ranges at each detail and location being evaluated.

A list of assumptions and input values used for each detail and location being evaluated including:

- Live load distribution factor
- Wheel and axle spacings of the fatigue truck used as defined in the guide specification.
- Section properties at the detail and location and a narrative stating whether those section properties are composite or non-composite.
- ADTT, "$T_a\$", growth rate "$g\$" and present age of structure in years.
- Impact percentage (10%)

Calculated reliability factor "$R_s\$", basic reliability factor "$R_{s0}\$", "$F_{s1}\$", "$F_{s2}\$", "$F_{s3}\$"

Method B  Current Standard AASHTO specifications for fatigue

A table showing:

- Remaining safe and mean fatigue life
- Moments and stress ranges at each detail and location being evaluated.

A list of assumptions and input values used for each detail and location being evaluated including:

- Live load distribution factor, axle; (S (in mm)/2134 mm)
- Fatigue vehicle used (MS18)
- Section properties at the detail and location and a narrative stating whether those section properties are composite or non-composite.

402.3  FATIGUE RETROFIT

402.3.1  END BOLTED COVER PLATES

When the fatigue category of welded cover plate ends or welded flange and web splices are to be upgraded, bolted splice plates shall be provided. The bolted plates shall be designed to carry the total loads (MS18) at the point of transfer. AASHTO, in the 1997 interim and commentary, presents a design example for end bolted cover plates.
402.3.2 BOX GIRDER PIER CAPS

Often box girders were constructed using non-continuous back-up bars which 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 - Page 4-5 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 RETROITS

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 STRENGTH ANALYSIS

The 125 percent Beta Factor required in Section 302.4.1 of the Manual shall not be applied when analyzing existing members for strength.

In analyzing the strength of existing members which are to receive a new deck, a future wearing surface (2.87 kPa) shall be included in the dead load.

402.5 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 must 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 must be left to the ingenuity of the designer. However, the designer must remember to pay strict attention to practicality as well as strength and fatigue requirements. Review unusual details with the Office of Structural Engineering before proceeding.
Procedure:
1. Drill 40 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.
When retrofitting or repairing truss members, the designer must 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.

### 402.6 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 513/863 "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.

### 402.7 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. An assessment by the District Bridge Engineer or other ODOT representative with experience in heat straightening must be made as to 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. If field welding is required notes should be added detailing the inspection required to assure a quality weld. Plan requirements are to provide a pay quantity and a detail showing the location of the repair.

### 402.8 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. Figure 402 - Page 4-7 and Figure 403 - Page 4-8 show a method for consideration.

Contact the Office of Structural Engineering if a pin and hanger assembly is to be rehabilitated. Lubrication and nondestructive testing will be required.

### 402.9 BOLTS

Bolts should conform in general to Section 300 of the 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 are acceptable when clearance problems arise. Remember that two or more bolt manufacturers must 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 must 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 must 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 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, thepatching should be paid for on a meter basis. This will require an Item Special, Patching Concrete Structure, misc.:........ pay item. A plan note will be required describing the work and tying it to CMS item 519.

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 25 and 150 mm, and where at least 15 m² of repair area is involved.

- For quantities of 15 to 35 m²
  Item 520, Pneumatically Placed Mortar, As Per Plan should be used.
• For quantities greater than 35 m²

A pay item, Item 520, Pneumatically Placed Mortar, as per plan should be used.

The "as per plan" portion of Items 519 & 520, is required as the plans must detail the locations of the 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). Note [55a]

Trowelable mortar should generally be specified when the repair depth is less than 40 mm deep and the repair area is less than 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 75 mm and the quantity is less than 15 m². 75 mm is the maximum depth of patch that should be attempted with this type of mortar.

A pay item, Item Special Patching Concrete Structures with Trowelable Mortar should be used and reference should be made to a proposal note.

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 for which a proposal note is available.

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:

1. 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.

2. 32 mm minimum latex modified concrete (LMC) per supplemental specification 847 or 848.

3. 45 mm minimum superplasticized dense concrete (SDC) per supplemental specification 847 or 848.
4. 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 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 meters of the bridge decks to be overlayed is 400 square meters or greater. The normal depth of uniform removal of the original deck concrete called for shall be 25 mm. Plan removal depths should not be set up to go below the top mat of deck reinforcing.

404.3 UNDERDECK REPAIR

For under deck spalls up to 25 mm deep use trowelable mortar (a proposal note is available). 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 extra 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.

405 BRIDGE DECK REPLACEMENT

Notes similar to those found in Section 600 should be provided for deck removal in order to preclude damage to steel stringers. Refer also to the applicable portions of the Manual, Section 300.

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 which 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 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. 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 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 300 mm. Provide reinforcing steel in any haunch greater than 125 mm. A deep haunch (125 mm or more) shall be made by providing a haunch similar to Figure 317 with the horizontal haunch width limited to 225 mm on either side of the flange.

405.3 CLOSURE POUR

A closure pour is not 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 retro-fit 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, 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 - Page 4-14

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 1000 mm 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 must in fact be established that a severe problem does not exist if coating is the chosen course of action.
TERMS:

LONG CANTILEVER  SHORT CANTILEVER  SPAN CONTIGUOUS WITH SHORT CANTILEVER

EXAMPLE CONCRETE DECK POUR SEQUENCES:

ACCEPTABLE

EXPANSION JOINT

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

UNACCEPTABLE

EXPANSION JOINT

PLAN SHOWING CONCRETE DECK POUR SEQUENCE

DECK CONCRETE, SEQUENCE OF PLACING

Figure 404
4-14
When practical, on overlay projects, a retrofit similar to that shown on Figure 405 - Page 4-17 and Figure 406 - Page 4-18 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. An alternative is the use of the Polymer Modified Asphalt Expansion Joint System, Section 306.2.3. This joint is limited in movement to 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 must 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 must 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 must take care to ensure that they are designed and detailed in a professional manner.

On projects involving stage construction, joints in the seal armor must 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 Figure 407 - Page 4-19 and Figure 408 - Page 4-20. 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 80 meters (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 must 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 205.8.1 of the Manual.

Type A pressure relief joints shall be specified when the approach roadway pavement is rigid concrete. The pressure relief joint shall be placed at the end of the approach slab first transverse pavement joint but not greater than 15 meters from the approach slab. When integral or semi-integral design is used and the approach pavement is rigid concrete, a Type A pressure relief joint shall be specified at the end of the approach slab.

407 RAISING AND JACKING BRIDGES

Thought must be given to any required jacking procedure and constraints. The bridge must be raised uniformly in a transverse direction in order to avoid inducing stresses into the superstructure. Differential movement between stringers shall be limited to 6 mm. Similarly, consideration must 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.

When raising a structure, the adjustment in beam seat elevations shall be accomplished by steel shims if the amount raised is 100 mm or less. If the structure is raised more than 100 mm, the bridge seat should be raised for its entire length by adding a reinforced concrete cap.

408 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 must 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 must 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 200 mm below the bottom flange. Check to see if the bottoms are rusted through before preparing the scupper extension detail.
VERTICAL EXTENSION OF STRUCTURAL EXPANSION JOINTS INCLUDING ELASTOMERIC STRIP SEALS
AS EXAMPLE ONLY. NOT FOR DETAIL
409 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 which 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 Page 4-22.

For widenings (2 beams or more) where the deck’s phase line is not between the new and old superstructure members, figure 409-C, a closure will be still be required. Existing cross frames (figure 409-C1) under the closure pour location need to be released before the phase 2 pour. Cross frames between new and existing members should be installed before the Phase 2 pour. Re-install the released crossframes after phase 2 but before phase 3 pours. Rebar splices should occur within the closure section. The width of the closure section should be at least 800 mm. See figure 409-C page 4-22.

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 (figure 409-B1) 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 800 mm. See figure 409-B Page 4-22.

Closure pours may be eliminated if the differential deflection is expected to be less than 6 mm.

Longitudinal construction joints should be treated with a high molecular weight methacrylate (HMWM) sealer. See section 302.2.9.
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.

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

Differential foundation settlements must 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.

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:

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.
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.

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 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 free standing individual columns without a cap are not permitted.

410 RAILING

Railing not meeting current standards will require upgrading when that structure is included in any construction project.

There are several methods for upgrading existing parapets to the deflector shape.

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 - Page 4-25. Dowels should be at least 150 mm deep and should be spaced at no more than 400 mm c/c. Grout should be epoxy grout per 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, [48], can be found in Section 600 of the Manual. Be aware that this note is general and must 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 150 mm deep and should be spaced at no more than 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
DESIGNER:  
CONTACT THE DISTRICT TO DETERMINE THE DESIRED DISPOSAL OF THE EXISTING RAILING

MODIFIED CURB PLATES

EXISTING CURB PLATES

RAILING FACED, AS PER PLAN, TYPE A

RAILING FACED, AS PER PLAN, TYPE B

ESTIMATING INFORMATION
REINFORCING STEEL........... 0.50 m³/m³
CONCRETE.................... 0.35 m³/m³
DOWEL HOLES................. 3 CYLMERSovo F Joint
CONCRETE REMOVAL........... 0.05 m³/m³

NEW 110 RAILING CONCRETE, AS PER PLAN

COMMON NOTES:
1. EXISTING REBAR LOCATIONS NOT KNOWN
2. ALL REINFORCING STEEL SHALL BE SPACED AS SHOWN VERTICAL STEEL SHALL BE 76 MM MIN.
   SHINKAGE CONTROL JOINTS BY 76 MM MIN.
3. ALL CONDITIONAL STEEL SHALL BE CONTINUOUS WITH MIN. LAP SPACES
4. CONCRETE COVER SHALL BE 50 MM TYPICAL
5. REINFORCING STEEL - ASTM A615, A620 OR
   A992. TE = 400 MPa, TF = 160 MPa
6. FOR ADDITIONAL NOTES AND INFORMATION
   SEE D-1 AND THE GENERAL NOTES.
7. THE ADJACENT DETAIL APPLIES TO BOTH THE SUPERSTRUCTURE AND MINIMAL RAILING
8. CURING SHALL BE AS PER SIJU, METHOD IN
testing of various railing sections with shallow rebar anchorage. Grout should be epoxy per 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, Standard Drawing TBR-91M 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 the Manual) and the actual bearing restoration work into another. The contractor should be given the option of totally replacing the bearings.

If elastomeric bearings are used at abutments where the existing expansion joint is built according to Standard Drawing SD-1-69, the joint must be modified. The old Standard 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.

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 - page 4-27
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 which 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 minimum of two (2) per bridge for bridges with a deck area less than 225 square meter.
- A minimum of three (3) per bridge for bridges with a deck area between 225 to 450 square meter.
- A minimum of four (4) per bridge for bridges with a deck area between 450 to 900 sq. m with one additional core for each additional 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:

- Obvious crumbling
- Stratification or delamination zones
- Soundness of aggregate
- 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 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 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 which 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 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.

<table>
<thead>
<tr>
<th>Measured % Unsound Area</th>
<th>Estimating Factor</th>
<th>Project % Plan Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>0-3</td>
<td>30</td>
</tr>
<tr>
<td>15</td>
<td>2.33</td>
<td>35</td>
</tr>
<tr>
<td>20</td>
<td>2</td>
<td>40</td>
</tr>
<tr>
<td>25</td>
<td>1.9</td>
<td>47.5</td>
</tr>
<tr>
<td>30</td>
<td>1.87</td>
<td>55</td>
</tr>
<tr>
<td>35</td>
<td>1.79</td>
<td>62.5</td>
</tr>
<tr>
<td>40</td>
<td>1.69</td>
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<td>45</td>
<td>1.56</td>
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<tr>
<td>70</td>
<td>1.21</td>
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<td>87.5</td>
</tr>
<tr>
<td>90</td>
<td>1.00</td>
<td>90</td>
</tr>
</tbody>
</table>

The plan quantity must 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 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 + 3/4". 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. A separate hand chipping bid item is not required if the method of removal is mechanical scarification.

Accurate records of actual quantities shall be maintained for each bridge.

It is recommended Districts review any criteria for selecting rehabilitation and replacement projects with the Offices of Highway Management and Structural Engineering to help assure state wide 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 must be clearly understood from a design standpoint.

413 REFERENCES

1. FHWA-RD-78-133, "Extending the Service Life of Existing Bridges by Increasing their Load Carrying Capacity," 1978


9. Park, Sung H., "Bridge Rehabilitation and Replacement (Bridge Repair Practice)," S.H.Park, P.O. Box 7474, Trenton, N.J., 08628-0474, 1984
SECTION 500   TEMPORARY STRUCTURES

501   PRELIMINARY DESIGN

During the preliminary design stage, the grade and the alignment of the temporary structure must be established and shown on the Site Plan. The roadway width, hydraulic design, clearance requirements, and all other design parameters must also be determined 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. A preliminary design submittal is required and the submission shall be made concurrently with the permanent structure preliminary design submittal.

Provisions for pedestrian traffic on the temporary structure shall be considered. If warranted, sidewalks on temporary structures generally should be a minimum of 1200 mm wide. Roadway width for two way traffic on the temporary bridge is normally not less than 7000 mm face to face of guardrail or toe to toe of parapet.

501.1 HYDRAULICS

The temporary bridge shall be designed to accommodate the 5 year storm. Upstream flooding and stream velocities produced by the temporary structure need to be considered when determining the structure waterway opening. Scour depths for the 5 year storm shall be calculated and must be accounted for in the design of the temporary bridge.

The designer shall show the water surface elevation and velocity of the 5 year storm on the temporary structure plans. The temporary bridge superstructure shall clear the 5 year storm.

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 5 year storm.

502   DETAIL DESIGN

The temporary structure detail plans shall be complete and independent of the permanent structure plans. The temporary structure detail plans shall include a general elevation plan, general notes, a table of estimated quantities, a reinforcing steel bar list and all necessary detail plan sheets. Provide a detailed list of the estimated quantities with individual pay items for the temporary structure. Final bid item shall be temporary structure Lump Sum.
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, MS18 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 150 mm, with a 1.4 MPa effective ice strength. The force shall be assumed to act at the 5 year highwater level.

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.

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 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. Guardrail, posts and anchorages shall be designed to resist loads specified in AASHTO. Post spacing shall not exceed 1905 mm center to center. In lieu of guardrail, precast concrete barriers (Standard Drawing PCB-91M) may be used provided they are anchored properly to the structure.

G. Field welded connections shall require nondestructive testing as per 513/863. Bolted connections are preferred and generally are more economical.

H. Designs which minimize debris accumulation should be considered.

I. Shop drawings are not required. Adequate plan details need to be provided.

503 GENERAL NOTES

The designer should provide plan note(s) with the Temporary Structure plans similar to the following:
The Contractor may substitute used or alternate members for the members shown on the Temporary Structure Plans, provided that the members structural adequacy is equal to or greater than the original member. Waterway opening size and required clearances are to be maintained. The Contractor shall submit to the Department calculations by a registered professional engineer showing the substituted member qualifies for the substitution. Only new bolts shall be used.

Structural steel need not be painted.

For each individual project the general notes and the pay item descriptions shall be developed. The following instructions are provided to assist in developing the necessary general notes.

When 513/863 Structural Steel is specified in the plans, only the following CMS descriptions shall apply:

- **Straightening**: 513.06/863.12
- **Holes for High-Strength and Bearing Bolts**: 513.14/863.20
- **High Strength Steel Bolts, Nuts and Washers**: 513.15/863.21
- **Welding**: 513.17/863.23
- **Nondestructive Testing**: 513.21/863.27
- **Shipping, Storage and Erection**: 513.22/863.28

When 511 Class "C" is specified in the plans CMS 511.15 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.

**MAINTENANCE**: The Contractor shall maintain all portions of the temporary structure in good condition with regard to strength, safety and ridability. This shall be included as incidental to 614 Maintenance of Traffic. The waterway opening shown on the plans shall be maintained at all times. If debris accumulates within the waterway opening or on any part of the structure the debris shall be promptly removed by the Contractor. Removal of debris will be compensated for as per CMS 109.04.

**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, the Contractor shall be responsible for immediately taking the actions necessary to protect traffic, repairing and reopening the temporary structure. When a Contractor closes a temporary structure he shall immediately notify the Engineer and the appropriate law enforcement agency. Water elevations exceeding the 5 year highwater elevation or an excessive accumulation of debris within the waterway opening shall
be sufficient reasons to close the temporary structure. The 5 year highwater elevation shall be painted with fluorescent paint on the temporary structure, at a visible location. This shall be included as incidental to 614 Maintenance of Traffic, as per plan.
SECTION 600  TYPICAL GENERAL NOTES

601  DESIGN REFERENCES

601.1  GENERAL

This section contains various typical general notes. Specific notes must be written, or these notes may need to be revised, to conform to the actual conditions which exist on each individual project.

601.2  STANDARD DRAWINGS AND SUPPLEMENTAL SPECIFICATIONS

All Standard Drawings and Supplemental Specifications which apply should be listed, giving date of approval or latest revision date, if revised.

The Designer shall also insure the listed Standard Drawings and Supplemental Specifications are transferred to the project plans title sheet and match the information on the title sheet.

[1]  REFERENCE shall be made to Standard Drawing(s):

___________Dated (revised) _______

___________Dated (revised) _______

___________Dated (revised) _______

and to Supplemental Specification(s):

___________Dated __________

___________Dated __________

___________Dated __________
601.3 DESIGN SPECIFICATIONS

A note similar to the following should be provided on the plans for all bridges if this note is a correct statement:


* Designer should fill-in current edition and latest interims.

602 DESIGN DATA

The following pertinent design information shall be included with the Structure General Notes for all bridge plans:

602.1 DESIGN LOADING

For bridges designed for highway loads, the design loading shall be:

[3] DESIGN LOADING: MS18, Case______(I or II) and the Alternate Military Loading.  

Future Wearing Surface (FWS) of ____* kPa.  

The statement "Case ___(I or II)" applies only to steel bridges.  

* Designer to fill in load.

For bikeway/pedestrian bridges that will not accommodate vehicular traffic the design loading shall be:

[4] 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:
DESIGN LOADING: * kN/m\(^2\) 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

For Load Factor design

DESIGN DATA:

Concrete * - compressive strength 31.0 MPa (superstructure)
Concrete * - compressive strength 27.5 MPa (substructure)

* Class S or High Performance Concrete HPC SS 844 for superstructure
  Class C or High Performance Concrete HPC SS 844 for substructure

Reinforcing steel - ASTM A615M, A616M or A617M
Grade 420 minimum yield strength 420 MPa.
Spiral reinforcement may be plain bars, ASTM A82M or A615M.

(If spiral reinforcing bars are not used, the portion of the note beginning with "spiral" should be omitted)

** Structural Steel
  ASTM A588M or A572M - yield strength 350 MPa
  A36M - yield strength 250 MPa

** If more than one grade of steel is selected description shall clearly indicate where the different grades are used in the structure.

For Service Load Design

DESIGN DATA:

Concrete * - unit stress 10.3 MPa (superstructure)
Concrete * - unit stress 9.2 MPa (substructure)

* Class S or High Performance Concrete HPC SS 844 for superstructure
  Class C or High Performance Concrete HPC SS 844 for substructure
Reinforcing Steel - ASTM A615M, A616M or A617M.  
Grade 420 - unit stress 160 MPa.  
Spiral reinforcement may be plain bars, ASTM A82M or A615M  

(If spiral reinforcing bars are not used, the portion of the note beginning with "spiral" should be omitted)  

** Structural Steel  
ASTM A588M or A572M - unit stress 186.2 MPa  
A36M - unit stress 138 MPa  

** If more than one grade of steel is selected description shall clearly indicate where the different grades are used in the structure.  

PRESTRESSED CONCRETE MEMBERS  

[8] DESIGN DATA:  

Concrete ___*__ - compressive strength 31.0 MPa  
* Class S or High Performance Concrete HPC SS 844 for superstructure  

Reinforcing steel - ASTM A615M, A616M or A617M  
Grade 420, minimum yield strength 420 MPa.  
Mild reinforcing steel for the concrete prestressed beams Grade 420, minimum yield strength 420 MPa  

Concrete for prestressed beams  
** Compressive Strength - 38 MPa  
Unit stress - 15.2 MPa compression  
3.1 MPa tension  

Prestressing strand ASTM A416M  

\[ f' = 1860 \text{ MPa} \]  

Initial stress = 0.75 \( f' \) (Low relaxation strands)  

** Value should be revised if a design strength different than 38 MPa is used.
602.3 FOR RAILWAY PROJECTS

For structures carrying railroad traffic, the following notes should be provided on the project plans:


CONSTRUCTION AND MATERIAL SPECIFICATIONS: State of Ohio, Department of Transportation, dated January 1, XXXX. *

* Designer should fill-in current edition and latest interims.

[10] DESIGN DATA:

Design Loading - Cooper E-80 with diesel impact

Concrete * - unit stress 10.3 MPa (superstructure)

Concrete * - unit stress 9.2 MPa (substructure)

* Class S or High Performance Concrete HPC SS 844 for superstructure
  
  Class C or High Performance Concrete HPC SS 844 for substructure

** Structural steel - ASTM A36M-unit stress 138 Mpa.
  ASTM A588M or A572M - unit stress 186.2 MPa

Reinforcing steel - ASTM A615M-unit stress 167 Mpa

** If more than one grade of steel is selected description shall clearly indicate where the different grades are used in the structure.
602.4 DECK PROTECTION METHOD

A list of the deck protection method(s) that have been specified in the plans, one or more of the following notes, shall be added to the Design Data section of the Structure General Notes:

[11] DECK PROTECTION METHOD:

- Epoxy coated reinforcing steel
- 65 mm concrete cover
- Superplasticized dense, Micro-silica, Epoxy, or Latex modified concrete overlay *
- Waterproofing and asphalt concrete overlay
- Steel drip strip
- Other (Specify)

(Select the appropriate method(s) from the list.)

* Only applicable for existing decks

602.5 MONOLITHIC WEARING SURFACE

The following note should be furnished for concrete bridge decks.

[12] MONOLITHIC WEARING SURFACE is assumed, for design purposes, to be 25 mm thick.

602.6 SEALING OF CONCRETE SURFACES - NOTE RETIRED

NOTE [13] RETIRED - NO LONGER REQUIRED

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. The notes should be modified as required to fit the conditions. For example, in the first note the word "dismantled" should be used if the steel is to be re-erected, otherwise the word "removed" is preferable.
SECTION 600 TYPICAL GENERAL NOTES - METRIC SEPTEMBER 1998

[14] REMOVAL OF EXISTING STRUCTURE: When no longer needed to maintain traffic the existing structure shall be removed upon receiving permission from the Engineer. (Structural steel shall be carefully dismantled and stored along the right-of-way for disposal by the State’s forces).

(The parenthetical sentence in the above note should be used if it is the desire of the owner to salvage any portion of the bridge. The degree of care to be exercised in the removal should be described 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, that should be stated, along with any other restrictions or allowances). If this option is used, the pay item shall include as per plan.

The following note should be used when removal of structure to 300 mm below ground line as specified in CMS 202 will not fill the specific requirements of the project.

[15] REMOVAL OF EXISTING STRUCTURE: When no longer needed to maintain traffic the existing structure shall be removed. Abutments shall be removed to Elev.____. Piers shall be removed to Elev.____.

[16] PORTIONS OF STRUCTURE REMOVED, AS PER PLAN shall include the elements indicated in the plans and general notes and 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. All work shall be done 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 41 kilogram class. Pneumatic hammers shall not be placed in direct contact with reinforcing steel that is to be retained in the rebuilt structure.
PROTECTION OF TRAFFIC: Prior to demolition of any portions of the existing superstructure, the contractor shall submit his plans for the protection of traffic (vehicular, pedestrian, boat, etc.) adjacent to and/or under the structure to the Director for approval. These plans shall include provisions for any devices and structures that may be necessary to ensure such protection. Temporary vertical clearances specified on the plans or in the proposal shall be maintained at all times except as otherwise approved by the Director.

PAYMENT: This work will be paid for at the contract lump sum price bid, which price and payment shall be full compensation for all labor, equipment, materials and incidentals necessary to complete the work in conformance with these requirements, with pertinent provisions of 202, and to the satisfaction of the Engineer.

603.1 CONCRETE DECK REMOVAL PROJECTS The following removal note should be used for concrete deck removal projects, where the existing steel superstructure is to remain. Delete the portions in the note that are not appropriate for the specific project.

PORTIONS OF STRUCTURE REMOVED, AS PER PLAN

DESCRIPTION: This work shall consist of the removal of concrete decks including sidewalks, parapets, railings, deck joints and other appurtenances from steel supporting systems (beams, girders, cross frames, etc.). Care shall be taken during deck removals to protect portions of such systems that are to be salvaged and incorporated into the proposed structure. In this respect, the use of explosives, headache balls and/or hoe ram type of equipment is prohibited.

PROTECTION OF TRAFFIC: Prior to demolition of any portions of the existing superstructure, the contractor shall submit plans for the protection of traffic (vehicular, pedestrian, boat, etc.) adjacent to and/or under the structure to the Director for approval. These plans shall include provisions for any devices and structures that may be necessary to ensure such protection. Temporary vertical clearances specified on the plans or in the proposal shall be maintained at all times except as otherwise approved by the Director.

PROTECTION OF STEEL SUPPORT SYSTEMS: Before deck slab cutting is permitted, the outline of primary steel members in contact with the bottom of the deck shall be drawn on the surface of deck. Small diameter pilot holes shall be drilled 50 mm outside these lines to confirm the location of flange edges. Deck cuts over or within 50 mm of flange edges shall not extend lower than the bottom layer of deck slab reinforcing steel. Cuts made outside 50 mm of flange edges may extend the full depth of the deck. During cutting of the deck slab, care shall be taken not to damage steel members that are to be incorporated into the proposed structure.
REMOVAL METHODS: Concrete may be removed by cutting and by means of hand operated pneumatic hammers employing pointed or blunted chisel type tools. For removals above steel members, a hammer heavier than 16 kilogram but not to exceed 41 kilogram may be used at the approval of the Engineer, to ensure adequate depth control and to prevent nicking or gouging the primary steel members.

DECK REMOVALS: Due to the possible presence of welded attachments to existing structural steel (finishing machine, scupper and form supports, etc.), care shall be taken during deck removal to avoid damaging stringers which are to remain. Stringers damaged by the Contractor’s removal operations shall, at no cost to the project, be replaced or repaired. Proposed repairs, developed by a registered professional engineer, shall be submitted in writing for review and approval by the Director.

EXTRANEOUS MEMBERS: Existing extraneous members (i.e., finishing machine and form supports, etc., and the support for scuppers and bulb angles which are to be removed) attached by welded connections to portions of the top flanges designated "tension" shall be removed and the flange surfaces ground smooth. Grinding shall be carefully done and parallel to the flanges.

LOADING LIMITATIONS: No part of the structure shall be subjected to unit stresses that exceed 136.5% of allowable unit stresses as defined in the AASHTO Standard Specifications for Highway Bridges due either to demolition, erection or construction methods, or to the use or movement of demolition or erection equipment on or across the structure. Structural analysis computations, by a registered professional engineer, showing the allowable stresses and the maximum stresses produced by the Contractor’s methods or equipment shall be submitted to the Director for review and approval at least two weeks prior to the start of the work.

PAYMENT: This work will be paid for at the contract lump sum price bid, which price and payment shall be full compensation for all labor, equipment, materials and incidentals necessary to complete the work in conformance with these requirements, with pertinent provisions of 202, and to the satisfaction of the Engineer.
603.2 For modifications to or extensions of existing concrete substructure members where appearance is a concern the following notes should be provided.

[19] CUT LINE CONSTRUCTION JOINT PREPARATION: Saw cut boundaries of proposed concrete removals 25 mm deep. Remove concrete to a rough surface. Where practicable, the existing reinforcing steel where required in the plans shall be left in place. Install dowel bars if specified. Prior to concrete placement abrasively clean joint surface and exposed reinforcement to remove loose and disintegrated concrete and loose rust. The joint surface and exposed reinforcement shall be thoroughly cleaned of all dirt, dust, or other foreign material by the use of water, air under pressure, or other methods that produce satisfactory results. Concrete bonding surfaces shall be wet without free water as concrete is placed.

[20] SUBSTRUCTURE CONCRETE REMOVAL shall be 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 16 kilograms for removal within 450 mm of portions to be preserved. Outside the 450 mm limit, a hammer heavier than 16 kilograms, but not to exceed 41 kilograms, may be used upon the approval of the Engineer. Pneumatic hammers shall not be placed in direct contact with reinforcing steel that is to be retained in the rebuilt structure.

604 TEMPORARY STRUCTURE CONSTRUCTION

The applicable portions of the following temporary structure note should be included on the plans if the bridge roadway width is other than 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 _______ meters. The existing structure may be moved and used for the temporary structure without strengthening.

605 EMBANKMENT CONSTRUCTION

For all abutments, appropriate embankment construction notes should be provided in the Structure General Notes.

The piers that are adjacent to or set in the embankment area and that would be subject to lateral forces during embankment construction should be specifically identified by number in the notes, otherwise reference the to piers should be omitted.
605.1 ABUTMENTS ON PILES IN NEW EMBANKMENTS For abutments on piles placed in new embankments (modify for spread footings):

[22] PILE DRIVING CONSTRAINTS: Prior to driving piles, the spill through slopes and the bridge approach embankment behind the abutments shall be constructed up to the level of the subgrade elevation for a minimum distance of * behind each abutment. The excavation for the abutment footings and the installation of the abutment piles shall not begin until after the above required embankment has been constructed.

605.2 ABUTMENTS AND PIERS ON PILES IN NEW EMBANKMENTS For abutments on piles and capped pile piers placed in new embankments (modify for spread footings):

[23] PILE DRIVING CONSTRAINTS: Prior to driving piles, the spill through slopes and the bridge approach embankment behind the abutments shall be constructed up to the level of the subgrade elevation for a minimum distance of * behind each abutment. The excavation for the abutment footings and the installation of the abutment and pier piles, for pier(s) *, shall not begin until after the above required embankment has been constructed.

* Generally 60 meters. May be defined by station to station dimensions.

** Identify specific piers.

605.3 WALL TYPE ABUTMENTS IN NEW EMBANKMENTS For wall type abutments on piles with new embankment:

[24] PILE DRIVING CONSTRAINTS Prior to driving piles at the abutments, the bridge approach embankment behind the abutments shall be constructed 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 75 meters behind the abutments. The installation of the abutment piles shall not begin until after the above required embankment has been constructed. After the footing and the breastwall have been constructed, the embankment immediately behind the abutments shall be constructed 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.
605.4 WALL TYPE ABUTMENTS - NO NEW EMBANKMENT For wall type abutments on spread footings with no new embankment:

[25] CONSTRUCTION CONSTRAINTS All embankment material for filling the void created by excavating for the abutment footings shall be 203 granular embankment material. After the footing and the breastwall have been constructed, the void behind each abutment shall be filled 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.

605.5 EMBANKMENT CONSTRUCTION NOTE In an attempt to reduce settlements of the roadway approaches, use 150 mm lifts for embankment construction. Include a plan note similar to the following sample notes with 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: This note should not be incorporated into a set of structure plans except on approval of the foundation engineer of the Office of Structural Engineering.

To define the limits of measured pay quantities for bridges with wall-type abutments; excavation, backfill, and embankment diagrams (or a composite diagram where suitable), usually schematic abutment cross-sections, should be provided showing the boundaries between structure and roadway excavation, and between structure backfill and roadway embankment.

[26] ITEM 203 EMBANKMENT, AS PER PLAN: All fill material for the construction of the approach embankment placed between stations **** to **** and for filling the excavation void created by removal of the existing abutments, shall be placed in 150 mm lifts and compacted in accordance with 304.04.

** The approximate limits should be 30 meters behind each abutment

or

[27] ITEM 203 EMBANKMENT, AS PER PLAN: All fill material for construction of the approach embankment and for filling the excavation void created by removal of the existing FORWARD/REAR abutment, shall be placed in 150 mm lifts and compacted in accordance with 304.04.

The following note [28] may be necessary in conjunction with the notes in [25], [26] or [27], when there is a pay item for Item 503.
*** Use of excavation 503 items

When an excavation includes 10 $m^3$ or more of rock and/or shale, the quantity of rock excavation shall be separately itemized under

503 $m^3$ Rock and/or Shale Excavation

When the rock or shale excavation is under 10 $m^3$ the rock and shale shall not be separately itemized but included in either

503 $m^3$ Unclassified excavation including rock

503 $m^3$ Unclassified excavation including shale

When excavation includes no rock or shale the quantities should be separately itemized under

503 $m^3$ Unclassified excavation

In computing the quantity of 503 excavation, the designer should confirm that all removals under items 201, 202 or 203 have been excluded, as per CMS 503.01.

ITEM 503, UNCLASSIFIED EXCAVATION  ****, AS PER PLAN:

Unclassified excavation shall be in accordance with 503 except that the backfill material behind the abutments shall be 203 material placed in 150 mm lifts and compacted in accordance with 304.04.

606 FOUNDATIONS

606.1 PILES DRIVEN TO BEDROCK

The following note generally will apply where steel-H piles are to be driven to bedrock:

PILES TO BEDROCK Piles shall be driven to refusal on bedrock. Refusal shall be considered as obtained by penetrating soft bedrock for several millimeters with a minimum resistance of 20 blows per 25 mm or refusal shall be considered as obtained after the pile has contacted hard bedrock and the pile has then received at least 20 blows.

The Ultimate Bearing Value is ____ kN per pile for the ____ abutment piles. The Ultimate Bearing Value is ____ kN per pile for the ____ pier piles.
Abutment piles:
   _____ piles _____ meters long, estimated length
   _____ piles of order length _____ meters long
   _____ piles of order length _____ meters long
   _____ splices

Pier piles:
   _____ piles _____ meters long, estimated length
   _____ piles of order length _____ meters long
   _____ piles of order length _____ meters long
   _____ splices

# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 250 x 62, 300 mm Diameter), the estimated length, the order lengths and any splices required. The Ultimate Bearing Value is two(2) times the design load, based on actual dead and liveloads required for the pile. Ultimate Bearing Value is not the maximum capacity for the size of pile selected.

The estimated length shall include the required design length for ultimate bearing plus 1.5 meters for cut off purposes. (For piles to bedrock total order length per pile would be bottom of footing elevation minus bedrock elevation + 1.5 meters.) Standard length without splice is 18 m. Number of splices shall either be the actual number of splices required if the individual pile order length is greater than 18 meters or 50% of the number of required piles if no splices are theoretically required

606.2 PILES NOT DRIVEN TO BEDROCK

The following note, modified to fit the conditions, will apply in all cases except where the piles are to be driven to bedrock. The actual calculated Ultimate bearing load shall be given below:

[30] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is _____ kN per pile for the _____ abutment piles. The Ultimate Bearing Value is _____ kN per pile for the _____ pier piles.
Abutment piles:

- ____ piles ____ meters long, estimated length
- ____ piles of order length ____ meters long
- ____ piles of order length ____ meters long
- ____ splices

Pier piles:

- ____ piles ____ meters long, estimated length
- ____ piles of order length ____ meters long
- ____ piles of order length ____ meters long
- ____ splices

# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 250 x 62, 300 mm Diameter), the estimated length, the order lengths and any splices required. The Ultimate Bearing Value is two(2) times the design load, based on actual dead and liveloads required for the pile. Ultimate Bearing Value is not the maximum capacity for the size of pile selected.

The estimated length shall be based on the estimated driven length plus 1.5 meters. Individual pile order lengths shall add up to the estimated length with no individual pile order length greater than 18 meters. Number of splices shall either be the actual number of splices required if the individual pile order length is greater than 18 meters or 50% of the number of required piles if no splices are theoretically required.

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 ____ kN per pile for the ____ abutment piles. The Ultimate Bearing Value is ____ kN per pile for the ____ pier piles. The pier piles include an additional ____ kN per pile of Ultimate Bearing Value due to the possibility of losing frictional resistance due to scour.

Abutment piles:

- ____ piles ____ meters long, estimated length
- ____ piles of order length ____ meters long
- ____ piles of order length ____ meters long
- ____ splices
Pier piles:

____ piles ____ meters long, estimated length
____ piles of order length ____ meters long
____ piles of order length ____ meters long
____ splices.

[#] Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 250 x 62, 300 mm Diameter), the estimated length, the order lengths and any splices required. The Ultimate Bearing Value is two(2) times the design load, based on actual dead and liveloads required for the pile. Ultimate Bearing Value is not the maximum capacity for the size of pile selected.

The estimated length shall be based on the estimated driven length plus 1.5 meters. Individual pile order lengths shall add up to the estimated length with no individual pile order length greater than 18 meters. Number of splices shall either be the actual number of splices required if the individual pile order length is greater than 18 meters or 50% of the number of required piles if no splices are theoretically required.

The following note, modified to fit the conditions, will apply where piles are driven through new embankment.

[30b] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is ____ kN per pile for the ____ abutment piles. The Ultimate Bearing Value is ____ kN per pile for the ____ pier piles. An additional ____ kN of Ultimate Bearing Value per abutment pile is due to the possibility of down drag forces induced by embankment settlement. The pile Ultimate Bearing Value for the pier piles is ____ kN per pile. The pier piles should be driven to an Ultimate Bearing Value of ____ kN per pile. The additional ____ kN per pile of Ultimate Bearing Value is due to the possibility of down drag forces induced by embankment settlement.

Abutment piles:

____ piles ____ meters long, estimated length
____ piles of order length ____ meters long
____ piles of order length ____ meters long
____ splices
Pier piles:
____ piles ____ meters long, estimated length
____ piles of order length ____ meters long
____ piles of order length____ meters long
____ splices

#  Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 250 x 62, 300 mm Diameter), the estimated length, the order lengths and any splices required. The Ultimate Bearing Value is two(2) times the design load, based on actual dead and liveloads required for the pile. Ultimate Bearing Value is not the maximum capacity for the size of pile selected.

The estimated length shall be based on the estimated driven length plus 1.5 meters. Individual pile order lengths shall add up to the estimated length with no individual pile order length greater than 18 meters. Number of splices shall either be the actual number of splices required if the individual pile order length is greater than 18 meters or 50% of the number of required piles if no splices are theoretically required.

606.3 STEEL PILE POINTS  The following note shall be used where steel points are required, see Section 204.2.1.

[31] ITEM 507, STEEL POINTS, AS PER PLAN: Steel pile points shall be used to protect the tips of the proposed steel "H" piling. The steel points shall be furnished by Associated Pile and Fitting Corporation, 262 Rutherford Blvd., Clifton, New Jersey 07014; International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, North Carolina 28015; Dougherty Foundation Products, Inc., P.O. Box 688, Franklin Lakes, New Jersey 07417; Versa Steel Inc., 3601 N.W. Yeon Ave., P.O. Box 10559, Portland, Oregon 97210; Piling Accessories, Inc., 3467 Gribble Road, Mathews, North Carolina 28105; 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 A27M 450/240 - Class 2 - Heat Treated or AASHTO M103M 450/240 - Heat Treated. A notarized copy of the mill test report shall be submitted to the Engineer.
606.4 The following note shall be used where a minimum pile wall thickness is required, see Section 204.2.3. 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 250 (250 MPa) and the monotube piles are Grade 350 (350 MPa).

The 1997 CMS specifications establish minimum pile wall thickness by formula. Use of this note should only be with approval of the foundation engineer of the Office of Structural Engineering.

[32] ITEM 507 _______ PILES, AS PER PLAN

PILE WALL THICKNESS: The responsibility of choosing and providing 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 _____ mm. If a pile wall thickness greater than _____ mm is necessary to resist the pile installation driving stress, the Contractor shall make this determination and shall furnish a pile with an acceptable wall thickness. If monotube piles are used, the minimum wall thickness shall be _____ mm.

606.5 The following note shall be used where a minimum pile hammer size is required, see Section 204.2.4. This note is not required when piles are driven to refusal on bedrock and the estimated pile length is less than 15 meters.

The 1997 CMS specifications for piling establish required hammer rating based on Ultimate design load. Use of this note should only be with approval of the foundation engineer of the Office of Structural Engineering.

[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 _____ 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.
606.6 The following note shall be used where capped pile piers and steel "H" piles are to be used and the standard drawing does not already specify pile encasement.

[34] ITEM SPECIAL - PILE ENCASEMENT

All piles for the capped pile piers shall be encased in Class S or Class C concrete (499.03) and shall be in accordance with 511, except as modified herein. The required slump is 150 mm, plus or minus 12 mm. The maximum water to cement ratio shall be 0.50. If concrete is placed under water, the requirements of adding 10 percent more cement to the concrete shall be waived. The concrete shall be placed within a form that consists of polyethylene pipe (707.33), or PVC pipe (707.42). The encasement shall extend from 1 meter below the finished ground surface up to the concrete pier cap and shall be positioned so that at least 75 mm of concrete cover is provided around the exterior of the pile.

The length of pile encasement shall be measured in meters along the length of the pile. This item includes all work and materials necessary to furnish the required encasement. Payment will be made at the contract unit price per meter of pile encasement approved in place.

In lieu of encasing the pile in concrete, at the option of the Contractor, the pile may be galvanized as per 711.02. The galvanizing shall be continuous from a minimum of 1 meter below the finish ground surface up to the concrete pier cap. The galvanized coating thickness shall be a minimum of 100 µm (0.1 mm). All gouges, scrapes, scratches or other surface imperfections caused by the handling or the driving of the pile shall be repaired to the satisfaction of the Engineer. Payment for the galvanizing will be made at the contract unit price for Item Special, Pile Encasement. Payment will only be made for the galvanized length of pile as required by the plan and/or approved by the Engineer. All galvanizing provided beyond the project requirements is at the Contractor's expense.

606.7 FOUNDATION BEARING PRESSURE

The following note, with the blanks filled in as appropriate for each individual project, should be provided if there are abutments or piers which are supported by spread footings. The maximum bearing pressure to be shown is the actual calculated maximum pressure under the footing.

[35] FOUNDATION BEARING PRESSURE: _____ footings, as designed, produce a maximum bearing pressure of _____ MPa. The allowable bearing pressure is _____ MPa.
606.8 FOOTINGS

The following note should be provided if the footing excavation is mainly bedrock and the footings are to be at an elevation no higher than plan elevation:

[36] FOOTINGS shall be placed in bedrock at the elevation shown.

The following note should be provided, where footings are to be founded in bedrock at an elevation no higher than plan elevation.

[37] FOOTINGS shall extend a minimum of 75 mm* into bedrock or to the elevation shown, whichever is lower.

The following note should be provided, 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 75 mm* into bedrock. If necessary, the footings should be lowered. However, if the low point of the surface of the bedrock occurs 0.6 meters or more above plan elevation, the footings may be raised, upon approval by the Director, but to an elevation not higher than ____. Stepping of individual footings will not be permitted unless shown on the plans.

* (shall be greater than 75 mm if required by design considerations.)

The elevations, dimension and field adjustments in the preceding "FOOTINGS" notes should be chosen to satisfy the requirements of minimum cover, clearance, embedment and type of bedrock, without adversely affecting the strength or stability of the substructure unit.

606.9 DRILLED SHAFTS

The following drilled shaft notes should be added when applicable and revised for the specific project. Notes should be adjusted for number of different drilled shaft designs on the project.
[38A] DRILLED SHAFTS

The design load to be supported by each drilled shaft is **KN** at the abutments and **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 **MPa** assumed to act along the bottom **Mm** of the bedrock socket for the abutments and **Mm** of the bedrock sock for the piers. The allowable end bearing pressure is **MPa**.

* Loads and dimensions should be completed 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 may be required to coordinate the bridge work with the maintenance of traffic plans. The designer is responsible for assuring that notes in bridge plans match clearance requirements in project plans or refer to the maintenance of traffic plans.

608 RAILROAD GRADE SEPARATION PROJECTS

608.1 CONSTRUCTION CLEARANCE

The actual dimensions used in the text of this note should be obtained from the "Agreement" (a legal document signed by the Director and Railroad). See section 607.

[39] CONSTRUCTION CLEARANCE OF ____ meter horizontally from the center of tracks and ____ meters vertically from a point level with the top of the higher rail, and 1.8 meters from the center of tracks, shall be maintained at all times.

608.2 RAILROAD AERIAL LINES

[40] RAILROAD AERIAL LINES will be relocated by the Railroad. The Contractor shall use all precautions necessary to see that the lines are not disturbed during the construction stage and shall cooperate with the Railroad in the relocation of these lines. The cost of the relocation shall be included in the railroad force account work.
609 UTILITY LINES

The following note should be used only if existing utilities are involved.

[41] UTILITY LINES All expense involved in relocation (installing) the affected utility lines shall be borne by the Utility(ies). 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 TO BE ALTERED, WIDENED OR REPAIRED

The following note should be provided 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 513.02/863.07. (Delete the reference to 513.02 or 863.07 if structural steel is not involved.)

Contract bid prices shall be based upon a recognition of the uncertainties described above and upon a prebid examination of the existing structure by the Contractor. However, all project work shall be based upon actual details and dimensions which have been verified by the Contractor in the field.

The following note should be placed 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:
ITEM 509 REINFORCING STEEL, REPLACEMENT OF EXISTING REINFORCING STEEL, AS PER PLAN:

Any existing reinforcing bars which are to be incorporated into the new work and are made unusable by concrete removal operations shall be replaced with new steel at the Contractor’s cost.

Any existing reinforcing bars deemed by the Engineer to be unusable because of corrosion shall be replaced with new steel.

The number of kilograms of reinforcing steel paid for at contract prices shall be the actual kilograms of reinforcing steel specified by the Engineer as unusable due to corrosion and shall include placement, doweling, bending, supporting, tie wires and tying of that specified reinforcing steel.

* The designer shall include a bid item as defined above with a specific weight of reinforcing steel.

On bridge rehabilitation and widening projects where incidental 513, 863 or 516 steel items (i.e., cross frames, bearing plates, etc.) must be fabricated to dimensions obtained in the field during the contract, delays may occur when shop drawings and pay weights are approved in a manner which is satisfactory and desirable for new bridge projects. To avoid these delays, the following plan note should be provided. Before using this option, approval by the Office of Structural Engineering is required. All steel members to be included in this pay item must be defined. Insert the appropriate item description in the plan note title as indicated.

Note [44] was developed for use with 513 bid items. Structural steel since Jan 1998 should normally be bid using SS 863. Note [44a] is intended for use with SS 863. Whether bid 513 or 863, the recommended bid item quantity for rehabilitation work is in Kilograms rather than Lump Sum.
ITEM 513 STRUCTURAL STEEL FOR REHABILITATION, AS PER PLAN, (STRUCTURAL STEEL, REPLACEMENT OF DETERIORATED END CROSS FRAMES, AS PER PLAN) or (OTHER DESCRIPTION). Steel members to be fabricated under this item will not require shop drawings prior to fabrication. The Contractor shall make necessary measurements and prepare sketches, drawings, tables, etc. The Engineer shall have authority and responsibility for ensuring that the fabricated steel is acceptable. Technical assistance will be provided on request by the Office of Structural Engineering. Mill test reports and shipping documents shall be submitted to the Engineer for review and approval prior to incorporating steel items into the work, as required by 501.07. After fabrication, the Contractor shall submit shop drawings to the Engineer for review and approval to ensure that the drawings depict the steel as actually incorporated into the work. The Engineer will then send one approved set to the Office of Structural Engineering for information. Pay weights shall be computed in compliance with 513 of the Construction and Material Specifications and submitted to the Engineer for his review and approval. The fabricator shall furnish a 35 millimeter microfilm copy of each shop drawing, which shall be mounted on an aperture card as specified in 501.05.

Steel members included in this item include _____, _____, and _____.

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 fabricated steel supplied under this bid item is acceptable. The requirements for submittal of shop drawings to the Office of Structural Engineering is waived. The Contractor shall supply the Engineer with shop drawings stamped by a Professional Engineer and dated, as per 863.08, prior to any incorporation of fabricated steel at the project. The Engineer shall assure the submitted drawings match the fabricated steel delivered before the steel is incorporated into the work. If the Engineer is satisfied the Contractor shall supply a copy set, stamped and dated as per 863.08, to the Office of Structural Engineering for record purposes. SS 863’s required test data submittal to the Office of Structural Engineering is waived, but the Contractor’s written acceptance of the material test reports shall be furnished both the Engineer and the Office of Structural Engineering prior to installation of any steel.

At or before the pre-fabrication meeting the Engineer may choose to request assistance from the Office of Structural Engineering in whatever capacity is required.

Steel members included in this item include _____, _____, and _____.

6-24
610.2 When the following note is used a separate plan note and pay quantities for jacking or temporary support of the superstructure is required. This note should be revised 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 as required by System______, 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 16° C, lubricating sliding surfaces, and reassembly of the bearings. The Contractor shall assure all bearings are shimmed adequately and that no beams and/or bearing devices are “floating”. At the option of the Contractor and at no additional cost to the State, new bearings of the same type as the existing may be installed in place of refurbishing the bearings. All work shall be to the satisfaction of the Engineer. Payment for all the above described labor and materials will be made at the contract price bid for Item 516 - Refurbish Bearing Devices, As Per Plan.

610.3 The following note should be used and modified as necessary where jacking and/or temporary support of the existing superstructure is required.

[46] ITEM 516, JACKING AND TEMPORARY SUPPORT OF SUPERSTRUCTURE, AS PER PLAN
This item shall consist of furnishing all necessary labor, materials, and equipment to raise or re-position any existing structures to the dimensions and requirements defined in the project plans.

The Contractor shall be responsible for the design, installation and operation of an adequate jacking system, including any temporary or permanent supports necessary to perform the work described in the project plans. Three (3) sets of jacking plans, which include the information described in this note, shall be submitted to the Director for approval at least thirty (30) days before actual work is to begin. The plans shall be prepared and stamped by a registered professional engineer.

Jacking submittals shall include at least the following:

1. The signature and number, or professional seal, of the registered professional engineer who prepared the submittal.

2. Calculations and analyses of the structure to determine and define the actual loading applied at the Contractor’s selected jacking points.
3. A drawing showing the physical and dimensional position of the jacks with respect to the structure including clearances and center of lift.

4. A schematic layout of jacks, check valves, pumps with 3 way retractor valve, pressure gages, flow control valves, etc. in accordance with manufacturer's recommendations. All jacks for each abutment or pier shall be connected together. All jacks at each abutment or pier shall be the same size.

5. Analysis and calculations of the stresses induced or created in the structure and any temporary or permanent supports. Design calculations for any temporary or permanent supports.

6. Physical dimensions, materials, and fabrication details of any temporary or permanent supports. Horizontal and vertical movement restraint shall be provided.

7. A step by step procedure detailing all steps in the jacking operation.

8. Method of attachment to structural members. Welding to tension areas will not be permitted.

The entire system including jacks shall have 20% more capacity than required based on calculated loads.

For lifts greater than 25 mm, jacks shall have locking nuts to positively lock and support the structure during the lift.

Jacks shall have a swivel load cap, a domed piston head or some other device to protect against the effects of side load on the jack.

Jacks alone shall not be used to support loads except during the actual jacking operation. Temporary supports, blocking or other methods approved by the Director shall be used.

Single acting rams with no over-travel protection system shall not be used.

Spare equipment shall be available on site for the required structure raising to proceed in the event of breakdown. A list of spare equipment shall be provided to the Engineer.

At a minimum, a jacking operation shall lift all beams at any one abutment or pier simultaneously. The only exception is the situation where the work involves replacing or rehabilitating individual bearings; no permanent shimming is required and the height of the lift shall not exceed 6 mm.

Maximum differential jacking height between any adjacent abutments or piers shall be 25 mm or less.
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, the jacking operation shall immediately cease and approved supports shall be installed. The Contractor shall then analyze the damage and submit a method of correction to the Engineer for approval. Any beams that separate from the deck shall be epoxy injected for the distance of the separation in accordance with the proposal note "Concrete Repair by Epoxy Injection". Cost of this epoxy injection or other required repairs shall be borne by the Contractor.

The Contractor shall demonstrate to the Engineer that the bridge bearings are fully seated at all contact areas. If full seating is not attained, suitable means of repair, subject to the Engineer’s approval, will be required at the Contractor’s expense.

The jacking operation shall be directed by a Professional Engineer employed by the Contractor. Failure to have a Professional Engineer present shall be cause for ceasing jacking operations.

Payment shall be made at the lump sum price bid for Item 516, Jacking and Temporary Support of Superstructure, As Per Plan. This shall include all necessary tools, labor, equipment and materials necessary to complete this item of work.

610.4 When redecking a continuous beam bridge containing top flange fillet-welded cover plates and/or field butt-welded beams, the following note should be provided to facilitate the Engineer's inspection of the welded connections.

[47] INSPECTION OF EXISTING STRUCTURAL STEEL: The Engineer shall 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. The deck slab haunch forms immediately adjacent to such welds shall not be erected until after the Engineer has completed this inspection. This inspection shall not take place until after the top flanges are cleaned as specified in 511.08, but it shall be done before the deck slab reinforcement is installed. The cost associated with this inspection shall be included with Item 511, Superstructure concrete for payment. Any cracks found should be reported to the Office of Structural Engineering, 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.

610.5 The following note should be used where the existing parapet is to be refaced, the note will need to be modified for each specific project.

[48] ITEM 517 - RAILING FACED AS PER PLAN
Description: This item of work shall consist of facing curb style parapets, using cast in place concrete, to obtain the deflector shape as shown in the plans.
Removal: The contractor shall carefully remove the existing aluminum railing, posts, curb plates, existing concrete curb and bulb angle gutter. All loose or unsound concrete shall be removed. Also to be removed shall be any sound concrete necessary to obtain a minimum 100 mm thickness of new concrete.

NOTE TO DESIGNER: The list of items in the above removal portion of this note shall be modified as necessary to fit the actual conditions of your particular project.

Dowel Holes and Reinforcing Steel: Dowel holes shall be drilled where shown in the plans. Reinforcing steel shall be installed using epoxy grout per 510 and (CMS 705.20). All existing reinforcing steel bars in the area of the dowel hole shall be located with the aid of a reinforcing steel bar locator (pachometer) prior to drilling the holes. If an existing bar is encountered at the same location as a proposed dowel hole, the dowel hole shall be moved to either side of the existing bar. All reinforcing steel, dowel holes and grouting shall be included with item 517 for payment.

Surface Preparation: The parapet surface next to the refacing shall be thoroughly cleaned by abrasive blasting followed by an air blast. Use of hand tools may be necessary to remove scale from any exposed reinforcing steel. The surface shall be made free from spalls, latence, and all traces of foreign material. Detergent cleaning shall precede blast cleaning as necessary to ensure removal of contaminants that are detrimental to achieving an adequate bond.

Materials:
- Reinforcing steel - 709.00, grade 420
- Concrete: (511 Class S, Class C or SS 844 HPC Mix __ ) designer specified

Shrinkage Crack Control Joints: As soon as a concrete saw can be operated without damaging the freshly placed concrete, 32 mm deep control joints shall be sawed into the perimeter of the concrete parapet. The saw cut shall be made in the complete perimeter of the parapet, starting and ending at the elevation of the concrete deck. The control joint saw cuts shall be placed in the new concrete at the same location as the existing deflection joints and shall be made at right angle to the deck by sawing to match alignment of existing deflection joints. The use of an edge guide, fence or jig is required to ensure that the cut 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 6 mm. The perimeter of the deflection control joint shall be sealed with a caulking material conforming to Federal Specification, TT-S-00227E to a minimum depth of 25 mm.
Method of Measurement: The quantity shall be the actual length of railing faced as measured from end of wingwall to end of wingwall. This item shall include the furnishing of all labor, equipment and materials necessary to complete this work. All costs of removal, dowel holes, reinforcing steel, concrete and shrinkage control joints complete and in place shall be included in the unit price bid for:

<table>
<thead>
<tr>
<th>ITEM</th>
<th>UNIT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>517</td>
<td>Meters</td>
<td>RAILING FACED AS PER PLAN</td>
</tr>
</tbody>
</table>

**NOTE TO DESIGNER:** Reinforcing steel shall be 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** The below note should only be used where Dowel holes are to be drilled and there is no new concrete item 511 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.

**611.2** The following note shall be included in the General Notes of the roadway plans for ITEM 611 Reinforced Concrete Approach Slab (T=___mm), As Per Plan. This note is only required when the abutment is a integral or semi-integral design.

**[50] Note retired - see appendix**
611.3 A plan note is required for integral and semi-integral abutments to include for payment a neoprene sheet, with Item 511. The neoprene sheet is required for waterproofing of the backside of the joint between the integral backwall and the bridge seat.

[51] ITEM 511 CLASS C CONCRETE, ABUTMENT, AS PER PLAN: Install a 900 mm wide strip, 2.5 mm thick, general purpose, heavy duty neoprene sheet with nylon fabric reinforcement at locations shown in the plans. Secure the 1 meter wide neoprene sheeting to the concrete with 32 x 3 mm (length x shank diameter) galvanized button head spikes through a 25 mm outside diameter, 3 mm galvanized washer. Maximum fastener spacing is 225 mm. Other similar galvanized devices which will not damage either the neoprene or the concrete may be used 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 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 150 mm (+/-) from the vertical edge of the neoprene strip nearest to the centerline of roadway. For vertical joints, install 2 additional fasteners at 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 should completely overlap the horizontal strips. Laps in the length of the horizontal strips due to material manufacturing shall be at least 300 mm in length, if not vulcanized or adhesive bonded, or 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 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, mm</td>
<td>D 751</td>
<td>2.5 +/- .25</td>
</tr>
<tr>
<td>Breaking strength, grab WXF, N, minimum</td>
<td>D 751</td>
<td>3130 x 3130</td>
</tr>
<tr>
<td>Adhesive 25 mm strip, 50 mm minimum, N minimum</td>
<td>D 751</td>
<td>27</td>
</tr>
<tr>
<td>Burst strength(mullen) MPa, minimum</td>
<td>D 751</td>
<td>9.65</td>
</tr>
<tr>
<td>Heat aging 70 hours T 100 °C, 180 bend without Cracking</td>
<td>D 2136</td>
<td>No Cracking of Coating</td>
</tr>
</tbody>
</table>

6-30
Low temperature brittleness 1 hour at -40 °C, bend around 6 mm mandrel

D 2136  No Cracking of Coating

In lieu of the neoprene sheeting the Contractor may choose to supply Type 3 Membrane, 711.29.

Payment for labor, materials and installation of these items shall be included in Item 511 Class C Concrete, Abutment, As Per Plan.

611.4 The following notes shall be included in all bridge plans where a pipe drainage system behind the abutment backwall is being installed

1997 CMS 518.04 specifies if project plans require drainage pipe the pile shall be 707.33. Only requirements for designer under 1997 specifications is to assure size, location, length and perforated or non-perforated is specified on project plan abutment sheets.

[52] Note retired - see appendix

[53] Note retired - see appendix

611.5 The following note should be included in the Structural General Notes when concrete parapets are used and standard drawing BR-1M is not being referenced.

[54] CONCRETE PARAPETS: As soon as a concrete saw can be operated without damaging the freshly placed concrete, 32 mm deep control joints shall be sawed into the perimeter of the concrete parapet. The saw cut shall be made in the complete circumference of the parapet, starting and ending at the elevation of the concrete deck. The sawcuts shall be placed at a minimum of 1800 mm and a maximum of 3050 mm centers. The use of an edge guide, fence, or jig is required to insure 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 6 mm. The perimeter of the deflection control joint shall be sealed a caulking material conforming to Federal Specification, TT-S-00227E to a minimum depth of 25 mm. The bottom 13 mm of the inside and outside face should be left unsealed to allow water to escape.
611.6 The following note shall be added to insure proper seating of prestressed concrete box beams for skewed bridges.

[55] BEARING PAD SHIMS: 3 mm thick preformed bearing pad shims, plan area ___ mm by ___ mm (the same size of the elastomeric bearing pad should be used) shall be placed under the elastomeric bearing pads where required for proper bearing. The amount supplied is sufficient for 2 shims per beam. Payment will be made at the contract bid price for Item 516 - 3 mm Preformed Bearing Pads. Any unused shims shall become the property of the State.

611.7 This note should be used with any concrete patching bid items such as 519 or other concrete patching bid items which refer to the cleaning requirements as specified in CMS 519.04.

[55a] All surfaces to be patched and the exposed reinforcing steel within shall be thoroughly cleaned by abrasive blasting prior to the cleaning specified by 519.04. Cleaning shall precede application of the patching material or erection of the forms by not more than 24 hours.
SECTION 700  TYPICAL DETAIL NOTES

701  SUBSTRUCTURE DETAILS

701.1 STEEL SHEET PILING

The following note should be placed on the substructure or retaining wall sheet on which are shown the details of steel sheet piling that are to be left in place.

[56] STEEL SHEET PILING left in place shall have a minimum section modulus of________ mm$^3$ per meter of wall.

701.2 POROUS BACKFILL

The following porous backfill note should be provided on the appropriate detail sheets.

[57] POROUS BACKFILL WITH FILTER FABRIC, 600 mm thick shall extend up to the plane of the subgrade, to 300 mm 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, 600 mm thick shall extend up to the plane of the subgrade, to 300 mm below the embankment surface, and laterally to the ends of the wingwalls. 0.06 kg/m$^3$ of bagged No. 3 aggregate shall be placed at each weephole. Bagged aggregate is included with porous backfill for payment.
701.3 BRIDGE SEAT REINFORCING

For structures which contain bearing anchors, one of the two following notes should be placed 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, the first note should be provided. Where drilling of anchors into the bridge seat is required, the second note should be provided. (Formed holes are not practical for prestressed concrete box beam bridges.)

[59] BRIDGE SEAT REINFORCING: Reinforcing steel in the vicinity of the bridge seat shall be accurately placed to avoid interference with the drilling of bearing anchor holes or the pre-setting of bearing anchors.

[60] BRIDGE SEAT REINFORCING: Reinforcing steel in the vicinity of the bridge seat shall be accurately placed 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, the following note with the necessary modifications shall be placed on the appropriate substructure detail sheet.

[61] BRIDGE SEAT ELEVATIONS have been adjusted upward _____ mm at abutments and _____ 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, an optional backwall construction joint should be shown at the level of the approach slab seat and the following note provided either on the appropriate abutment detail sheet or in the General Notes.

[62] BACKWALL CONCRETE: In addition to CMS 511.08, backwall concrete above the optional construction joint at the approach slab seat shall not be placed 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, the seating of beams on bearings shall be carefully observed to assure that positive bearing is maintained. Proper elevation of the support/armor shall be achieved by adjusting the connection angles and bolts between beam and expansion joint.

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, the following note shall be provided:

[64] ABUTMENT CONCRETE above the bridge seat construction joint shall not be placed until the prestressed concrete box beams have been erected.
702 SUPERSTRUCTURE DETAILS

702.1 STEEL BEAM DEFLECTION AND CAMBER

For steel beam or built-up girder bridges a tabulation similar to that shown below shall be provided 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.

The deflection and camber data shall be shown as described in Section 302.4.1.8. The table is to include center of span, splice points and maximum 10 meter increments. An even closer spacing may be required due to unique geometry and a more precise definition of the cambered shape is required for proper fabrication.

<table>
<thead>
<tr>
<th>DEFLECTION AND CAMBER</th>
</tr>
</thead>
<tbody>
<tr>
<td>** SPAN NUMBER XX **</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Point</th>
<th>**</th>
<th>**</th>
<th>**</th>
<th>**</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>Deflection due to weight of steel</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># Deflection due to remaining dead load</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>
</tr>
<tr>
<td>* Adjustment required for horizontal curve</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Required shop camber</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Provide if applicable
** Define distance and position
# No separate wearing surface should be included that is not installed during the project

702.2 STEEL NOTCH TOUGHNESS REQUIREMENT (CHARPY V-NOTCH)

The following note shall be placed on a structural steel detail sheet for bridges having main load-carrying members which must meet minimum notch toughness requirements:

[65] CVN Where a shape or plate is designated (CVN) the material shall meet specified minimum notch toughness requirements as specified in 711.01.

702.3 HIGH STRENGTH BOLTS
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:

**HIGH STRENGTH BOLTS** shall be ___________ diameter A325M, galvanized, unless otherwise noted.

### 702.4 SCUPPERS

The following note shall be provided if reference is made to Standard Drawing SD-1-69 for scupper details:

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, the following note should be included with the pertinent bearing details:

**WELDING** shall be controlled so that the plate temperature at the elastomer bonded surface does not exceed 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 the following plan note shall be included.

**BEARING REPOSITIONING**: If the steel is erected at an ambient temperature higher than 27° C or lower than 4° C and the bearing shear deflection exceeds 1/6 of the bearing height at 15° C (+/-) 5° C, the beams or girders shall be raised to allow the bearings to return to their undeformed shape at 15° C (+/-) 5° C.

### 702.7 CONCRETE PLACEMENT SEQUENCE NOTES

Also see section 701.5 note.

#### 702.7.1 CONCRETE INTERMEDIATE DIAPHRAGM FOR PRESTRESSED CONCRETE
I-BEAMS

[71] INTERMEDIATE DIAPHRAGM concrete shall be _____ (Class "C" or "S" or HPC SS844). Deck slab concrete shall not be placed until at least 48 hours after placement of intermediate diaphragm concrete.

702.7.2 SEMI-INTEGRAL OR INTEGRAL ABUTMENT CONCRETE PLACEMENT FOR STEEL MEMBERS

The following notes need to be added depending on whether the project is phased construction, the type of superstructure member, and whether the project does or does not have a closure pour.

[71a] ABUTMENT DIAPHRAGM, STEEL SUPERSTRUCTURE Concrete encasing structural steel members supported in semi-integral and integral type abutments may be placed at least 48 hours before the actual deck concrete is placed.

[71b] ABUTMENT DIAPHRAGM, PRESTRESSED I-BEAM SUPERSTRUCTURE Concrete encasing prestressed I-beam structural members supported in semi-integral and integral type abutments may be placed at least 48 hours before the actual deck concrete is placed if the bridge’s skew is 10° or less. For bridges with skews greater than 10° the abutment diaphragm concrete should be placed either after the deck concrete has been placed or at the same time the deck concrete is being placed.

[71c] PHASED CONSTRUCTION ABUTMENT DIAPHRAGM CONCRETE, STEEL SUPERSTRUCTURE Concrete encasing the structural member sections supported in semi-integral and integral type abutments may be placed at least 48 hours before the actual deck concrete is placed if a closure pour section has been required and detailed in the abutment diaphragms. If the structure has no detailed closure pour section in the deck, the abutment diaphragm concrete shall be poured simultaneously with the deck pour to allow for expected deadload rotation at the abutments.

[71d] PHASED CONSTRUCTION ABUTMENT DIAPHRAGM CONCRETE,
PRESTRESSED I-BEAM SUPERSTRUCTURE  Concrete encasing the structural member sections supported in semi-integral and integral type abutments may be placed at least 48 hours before the actual deck concrete is placed if a closure pour section has been required and detailed in the abutment diaphragms and the skew of the bridge is 10° or less. If the structure has no detailed closure pour section in the deck or the bridge’s skew is greater than 10°, the abutment diaphragm concrete shall be poured either simultaneously with the deck pour or after the deck pour to allow for expected deadload rotation at the abutments.

702.8  CONCRETE DECK SLAB DEPTH

For a rolled steel beam bridge with a concrete deck the following note shall be provided on a superstructure detail sheet near the typical cross-section of the deck.

[72]  DECK SLAB DEPTH: The distance shown from top of deck slab to top of steel beam is the theoretical design dimension including the design haunch thickness of ____ mm. The quantity of deck concrete to be paid for shall be based on this dimension, minus the design haunch thickness, even though deviation from it may be necessary because the top flange of the beam may not have the exact camber or conformation required to place it parallel to the finished grade.

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.

702.9  CONCRETE DECK HAUNCH WIDTH

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.

7-7
A HAUNCH WIDTH of 225 mm shall be used for computing quantity of concrete. However, the haunch width may vary between 150 mm and 300 mm.

702.10 PRESTRESSED CONCRETE BRIDGES

For prestressed concrete I-beam bridges with concrete deck, the slab depth at beam bearings is computed as follows:

\[
\begin{align*}
A &= \text{Design slab thickness.} \\
B &= \text{Anticipated total camber in beam = established by design} \\
C &= \text{Adjustment for vertical curve. Positive for crest vertical curve.} \\
D &= \text{Design Haunch Depth} \\
E &= \text{Slab depth at beam bearings} = A + B + C + D \quad \text{(Sign of } C \text{ will change if sag vertical curve)}
\end{align*}
\]

The depth at midspan is computed as follows:

\[
F = \text{Anticipated slab depth at mid-span} = A + D
\]

The superstructure cross-section shall show the slab depth at beam bearings and the anticipated slab depth at mid-span, the latter as a nominal dimension with the following note:

This is the nominal dimension. The pay quantity of that portion of the deck concrete over the beams shall be based on the average of this dimension and the depth at beam bearings even though deviation from this average may occur because the top of the beam may not have the camber anticipated in the design; i.e., \*B\*. The camber of beams shall be measured in the field before the deck is placed. The actual depth at mid-span shall be the nominal dimension plus or minus the difference between actual and anticipated camber.

* In the above note actual values shall be substituted for B.
For prestressed concrete box beam bridges with an asphaltic concrete surface course or composite concrete decks (with asphalt references deleted) a note similar to the following shall be provided:

[76] Calculated camber at time of paving is ______mm. (A simplified method is two times initial (elastic) camber at transfer plus 12 mm.)

Calculated deflection due to weight of surface course and railing (deflector parapets, railings, sidewalks, etc.) is ______mm.

Camber of ______mm at center of spans is required for the vertical curve.

(1) Net final camber of beams equals required camber. No variation in thickness of 448 Intermediate course is required.

(2) ____mm extra camber is required. This shall be provided by thickening the 448 Intermediate course from a minimum of 32 mm (or other design nominal thickness) at ends of spans to _____mm at center of span.

(3) Net final camber of beams is _____mm. This is _____mm in excess of the amount required to place the top of the beam parallel to profile grade. This excess amount shall be compensated for by thickening the 448 Intermediate course from a minimum of 32 mm (or other design nominal thickness) at center of spans to _____mm at ends of spans.

The note shall be concluded with (1), (2), or (3) as appropriate. A diagram similar to Figure 701, Page 7-10, showing the required wearing surface thicknesses of 448 Intermediate and 448 surface courses of asphalt at the piers and/or abutments and center of span to meet the final required elevations shall be incorporated in the bridge plans.
ASPHALT THICKNESS DIAGRAM
(Crest Vertical Curve)

32 mm Uniform Thickness 448 Surface

32 mm Minimum Uniform Thickness 448 Intermediate

Variable Thickness 448 Intermediate as required to match final elevation (Dimensions shall be detailed)

Proposed Roadway Surface

Excessive Curve

64 mm

Box Beam Member

$\xi$ Span

ASPHALT THICKNESS DIAGRAM
(Straight Grade or Sag Vertical Curve)

32 mm Uniform Thickness 448 surface

32 mm Minimum Uniform Thickness 403

Variable Thickness 448 Intermediate as required to match final elevation (Dimensions shall be detailed)

Proposed Roadway Surface

64 mm

Box Beam Member

$\xi$ Span

Figure 701
7-10
702.12 ASPHALT CONCRETE SURFACE COURSE

The following note should be placed on the plans for prestressed concrete box beam bridges having an asphaltic concrete surface course. If the nominal thickness of 448 varies from the 32 mm shown, the note should be revised accordingly:

[77] ASPHALT CONCRETE SURFACE COURSE shall consist of a variable thickness of 448 Intermediate course and 32 mm thickness of 448 Surface course. The 448 intermediate course shall be placed in two operations. The first portion of the course shall be of 32 mm uniform thickness. The second portion of the course shall be feathered to place the surface parallel to and 32 mm below final pavement surface elevation.

702.13 PAINTING OF A588M STEEL

[78] Note retired - see appendix

The following note should be provided for bridge superstructures using unpainted A588M steel and having deck expansion joints. Integral or semi-integral bridges will not require painting of the beam ends.

[79] PARTIAL PAINTING OF A588M STEEL: A 3 meter length from the ends of beams (girders) adjacent to abutments (on both sides of intermediate expansion joints) and all cross frames and other A588M steel within these limits shall be painted. Paint shall be System IZEU. The prime coat shall be 708.17. The top coat color shall closely approach Federal Standard No. 595a - 20045 or 20059 (the color of weathering steel).

702.14 ERECTION BOLTS

Where erection bolts are specified for girder or beam bridges, and the differential dead load deflection between two adjacent members at any line drawn perpendicular to the centerline of the bridge is less than or equal to 13 mm the following note shall be provided if crossframes have not been referenced to GSD-1-96M:

[80] ERECTION BOLTS: The hole diameter in the cross frames and girder stiffeners shall be 4 mm larger than the diameter of the erection bolts. Erection bolts should be high strength bolts and shall remain in place. Two hardened washers shall be supplied with each high strength bolt and the bolts shall either be fully torqued or a lock washer used in addition to the two hardened washers. Bolts shall be furnished as part of Item 513/863.

In lieu of erection bolts and at the option of the Contractor, alternative means of temporary bracing may be used subject to the approval of the Director.
If the crossframes are not being referenced to Standard Drawing GSD-1-96M, and the differential deflection at any perpendicular line to the centerline of the bridge between two adjacent members is greater than 13 mm the following note shall be provided. (Note: if part of a structure has a differential deflection of greater than 13 mm and part of the structure does not, only the above ERECTION BOLT note shall be used)

**ERECTION BOLTS AND CROSS FRAME FIELD WELDING**: The hole diameter in girder stiffeners shall be 4 mm larger than the diameter of the erection bolts. The cross frame members shall have slotted holes 40 mm long, parallel to the cross frame member, and 2 mm wider than the erection bolts. Erection bolts should be high strength bolts and shall remain in place. Two hardened washers shall be supplied with each high strength bolt and the bolts shall either be fully torqued or a lock washer used in addition to the two hardened washers. Bolts shall be furnished as part of Item 513/863.

No welding of the cross frame members to the stiffeners shall be done until the concrete deck has been placed.

In lieu of erection bolts and at the option of the Contractor, alternative means of temporary bracing may be used subject to the approval of the Director.

**702.15 WELDED ATTACHMENTS**

The following note should be provided on plans for steel beam or girder bridges:

**WELDED ATTACHMENT** of supports for concrete deck finishing machine may be made to areas of the facia stringer flanges designated "Compression". Attachments shall not be made to areas designated "Tension". Fillet welds to compression flanges shall be not closer than 25 mm from edge of flange, be not more than 50 mm long, and be not smaller than the minimum size required by AWS D1.5

**702.16 SCREED ELEVATION**

In addition to the screed elevation table or diagram, a screed elevation note similar to the one below should be provided to describe the elevations that are given.

**SCREED ELEVATIONS** shown are for the deck slab surface prior to concrete placement. Allowance has been made for anticipated calculated dead load deflections.
702.17 STEEL DRIP STRIP

[84] Note retired - see appendix

[85] Note retired - see appendix

702.18 REINFORCING STEEL FOR REHABILITATION

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.

702.19 ELASTOMERIC BEARING MATERIAL REQUIREMENTS

All project plans which specify elastomeric bearings should contain the following plan note to supplement the CMS specifications, 711.23, which contain references to obsolete AASHTO (1997 Interim Specification) article numbers and acceptance criteria.

[87] ELASTOMERIC BEARINGS shall comply with item 516 and AASHTO Standard Specification for Highway Bridges, Section 18, Bearing Devices, Division II, Construction, Articles 18.4.5.1 and 18.5.6.2. Bearings shall be Grade 3, _____ durometer elastomer, and shall be subjected to the load testing requirements defined in Article 18.7.4.5 of the AASHTO document listed above. Bearings were designed under section 14.6. ___ of section 14, Bearings, Division I, Design. Testing shall be included in the unit price bid for the bearings, each.
SECTION 800 NOISE BARRIERS

801 INTRODUCTION

During Stage 2 Phase of a project a determination is made regarding the need to provide noise barrier walls. This information is contained in the Environmental Impact Document.

When noise barriers are necessary, the required noise barrier height, length and location(s) will be furnished to the Design Agency by the Office of Environmental Services.

The Office of Structural Engineering maintains a list of approved noise barrier suppliers which are in conformance with this Manual and the Department’s "Noise Barrier Design" plan insert sheets. This list is incorporated as part of the plan insert sheets.

Noise barrier panel material types currently approved include steel, wood, vinyl, fiberglass, concrete and brick or masonry.

Preliminary and detail design plan reviews will be administered during the appropriate review stage by the Department.

802 PRELIMINARY DESIGN

Preliminary design submittals shall include two (2) sets of the following:

- Design drawings showing plan and elevation views.
- Beginning and ending stations for each noise barrier section.
- Offsets from a centerline of roadway to centerline of noise barrier for beginning, ending and all intermediate stations.
- Elevations for the top and bottom of the noise barriers for each panel.
- Location of the finished grade line at each noise barrier section.
- The Design Agency’s foundation design
- A copy of the Office of Environmental Services letter specifying noise barrier wall locations.
- A copy of the Subsurface Investigation Report.
The Departmental or Local Political Authority’s selected noise barrier panel type(s) to be specified for this project (i.e.: Concrete, Wood, Fiberglas, Steel, Vinyl, Brick or Block Masonry).

Special project requirements, such as placing noise barriers on a bridge structure, avoiding interference with utilities, and specifying special wall colors, textures or finishes.

The District Production Administrator should be contacted for the approved noise barrier material types, suppliers, alternate bid requirements, and special features in accordance with the Department’s Noise Wall Policy 417-001(P). A copy of the letter from the District Production Administrator stipulating the information in this paragraph should be part of the preliminary design submittal.

Borings for noise barriers shall be considered as structure borings, except that the requirement of penetrating 10 meters into relatively stiff or dense material is waived.

Borings shall be 7.5 meters deep. If bedrock is encountered within the 7.5 meter depth the boring may be terminated after 1 meter of penetration into sound bedrock or 3 meters of penetration into soft bedrock.

802.1 NOISE BARRIER FOUNDATIONS

The Design Agency shall perform a subsurface investigation at all noise barrier sites. The subsurface work shall be in accordance with the Specifications for Subsurface Investigation, (September 1996), with the following revisions:

2.3 Planning

- Borings shall be located approximately on the centerline of the noise barrier at a frequency, approximately, of 45 to 60 meters. Generally a minimum of three borings should be obtained for each noise barrier wall section.

- Borings shall be 7.5 meters deep. If bedrock is encountered within the 7.5 meter depth the boring may be terminated after 1 meter of penetration into sound bedrock or 3 meters of penetration into soft bedrock.

4.0 Testing Program

- Consider obtaining a press sample and performing laboratory tests for strength parameters when cohesive soils are encountered.

5.0 Subsurface Investigation Reports

- Drawings shall be 559 x 864 mm in conformance with Section 100 of this Manual. Report format shall follow that specified in section 5.2 of the Department’s Specifications for Structure Subsurface Investigations, (September 1996).

803 DETAIL DESIGN

Detail design submittals shall include two (2) sets of the following:
All noise barrier project plan sheets

- All necessary dimensions, to adequately describe the required position of the noise barrier relative to the centerline of roadway,

- The length and height of barrier

- Generic post spacing dimensions

- Top of barrier sound line

- The finished ground line

- A noise barrier general summary containing all bid items and any required noise barrier material type alternate bids,

- Plan notes,

- Special plan details required to show features such as locations of utilities, special access for fire hydrants, termination at structures, (such as bridges, culverts, overhead sign supports) and details not covered by the Department’s "Noise Barrier Design" plan insert sheets,

- Subsurface investigation plan sheets

- Department plan insert sheets incorporating the foundation design depths for the project

- Revised Department’s "Noise Barrier Design" plan insert sheets, listing only approved suppliers for the material noise barrier types authorized for the project and any alternate bid noise barrier material types authorized for the project.

- Special considerations authorized for the project such as color, texture, pattern or brick style.

804  STRUCTURAL DESIGN CRITERIA

Designs should conform with the information provided in the ODOT "Noise Barrier Design" plan insert sheets, latest revision. When possible, noise walls should not be mounted on bridge structures. When a bridge mounted noise barrier design is to be used, the Department’s "Noise Barrier Design" plan insert sheets should be followed for design items such as: material selections, loadings and bridge mounted post details.


804.1 STRUCTURAL DESIGN

- Design of Posts

Steel and Reinforced Concrete - Load Factor Design
Timber and Glulam - Working Stress Design

- Design of Foundations
  Working Stress Design

- Design of Panels

- Manufacturer’s standards on materials used - Structurally designed to AASHTO or other nationally recognized standard subject to review and approval by the Office of Structural Engineering.

- Reinforcing Steel
  ASTM A615M, A616M or A617M
  Grade 420 - Fy = 420 MPa
  Reinforcing steel in noise barrier, if bridge mounted, shall be epoxy coated.

804.2 STRUCTURAL DESIGN DATA

Concrete - Class C
  f’c = 27.5 MPa
  unit stress = 9.1 MPa

Concrete Prestressed
  f’c = 34.5 MPa
  f’ci = 27.5 MPa
  unit stress 15.2 MPa compression
  unit stress 3.1 MPa tension

Concrete strength reduction factors
  Flexure = 0.90
  Shear = 0.85
  Axial compression = 0.70

Structural Steel
  ASTM A572M or A588M
  fy = 345 MPa

Prestressing Strands
  f’s = 1860 MPa
  Initial stress(stress relieved) = .70 f’s
  Initial stress (low lax) = .75 f’s

Timber
  fb = 10.3 MPa

Wind pressures

Roadway barriers = 1.2 kPa.
Bridge mounted barriers = 1.4 kPa

804.3 ACOUSTIC DESIGN REQUIREMENTS

Reflective noise barriers
  - Minimum TL(Transmission Loss) = 22 dBA

Absorptive noise barriers
  - Minimum TL(Transmission Loss) = 22 dBA
  - Minimum NRC (Noise Reduction Coefficient) = 0.70

Any barrier material submitted shall be acoustically tested at an independent laboratory capable of performing the following tests:

8-4
ASTM Standard Test Method for Sound Absorption and Sound Absorption Coefficients by Reverberation Room Method

ASTM E423 and E795 (Latest editions)

ASTM E90 and E413 (Latest editions)
SECTION 900    STRUCTURE RATING

901 INTRODUCTION

This section contains requirements and guidelines to be followed when rating the followings:

- New Structure
- Existing Structure
- Rehabilitated Structure

The bridge rating results and reports shall be submitted to the responsible ODOT District office.

AASHTO BARS-PC (BARS-PC) computer program shall be used to rate the bridges.

The BARS-PC program’s input and output are based on English units. The metric dimensions need to be converted in English units for input to the program.

BARS-PC program installation disks and User Manuals are available from the Office of Structural Engineering, Structural Rating Engineer, for use on ODOT projects. For system requirements and capabilities of the BARS-PC program, refer to Section 903 and the BARS-PC User Manuals.

902 REQUIREMENTS FOR BRIDGE RATING

902.1 GENERAL

For new, existing and rehabilitated structures, the designer will analyze and rate the entire super-structure including main spans and approach or ramp spans, if any, designed to carry vehicular traffic.

For new structures, the analysis shall be based on the design plans, assuring that all the changes in the final approved plans have been incorporated in the rating analysis and report.

The existing structures shall be analyzed based on the original design plans and actual field conditions.

Structures with major rehabilitation should be analyzed using the original design plans, actual field conditions, and rehabilitation plans, incorporating all the major changes during original construction and in the final approved plans, if any.

902.2 SPECIFIC REQUIREMENTS FOR RATING

All main structural members affected by live load shall be analyzed.

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, accordingly.

The members shall be analyzed as to the intended method of construction. Structural members constructed as non-composite should be analyzed as non-composite and so on.

All dead loads are to be calculated as per actual field conditions and shall be used in the
structural analysis. Future dead loads shall not be applied, unless directed otherwise in the scope of services.

AASHTO live load distribution factors shall be used for analysis.

All analysis shall be performed using Load Factor (Strength) Design Method. The load factors as defined in AASHTO shall be applied.

Working Stress Method can be used for those members that can not be analyzed by Load Factor (Strength) method in BARS-PC.

The analysis shall be performed for AASHTO MS18 (HS20-44) Loading, an Alternate Military Loading and the four Ohio Legal Loads (2S1, 3S1, 4S1, and 5C1) for both Inventory and Operating conditions according to Table 9-1.

The analysis for special or superload vehicle, if required in the scope, shall be performed for Operating condition only.

Refer to ODOT Procedure No.: 518-001 (P), “Rating of Bridges and Posting for Reduced Load Limits,” for rating procedures.

All vehicles used for analysis shall have transverse spacing between centerline of wheels, or wheel groups, of 1830 mm (6 ft.).

<table>
<thead>
<tr>
<th></th>
<th>Live Load Designation</th>
<th>Figure</th>
<th>Analysis*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MS18 (HS20-44) &amp; Alternate Military Loading</td>
<td>SEE AASHTO</td>
<td>I &amp; O</td>
</tr>
<tr>
<td>2</td>
<td>2S1</td>
<td>Fig. 901 Fig. 902</td>
<td>O</td>
</tr>
<tr>
<td>3</td>
<td>3S1</td>
<td>Fig. 901 Fig. 902</td>
<td>O</td>
</tr>
<tr>
<td>4</td>
<td>4S1</td>
<td>Fig. 901 Fig. 902</td>
<td>O</td>
</tr>
<tr>
<td>5</td>
<td>5C1</td>
<td>Fig. 901 Fig. 902</td>
<td>O</td>
</tr>
</tbody>
</table>

* I = Inventory Rating Analysis  
O = Operating Rating Analysis

902.3 REQUIREMENTS FOR BRIDGE RATING - FATIGUE ANALYSIS

If a fatigue analysis is required in the scope, the analysis shall be performed in accordance with AASHTO (see Section 905, References).

The fatigue analysis calculations and resulting ratings shall be presented in the final report.

902.4 ANALYSIS OF STRUCTURES WITH SIDEWALKS

Sidewalks shall have AASHTO live loads
applied, but reduced by 50% to reflect actual service conditions.

902.5 BRIDGE ANALYSIS FOR LEGAL AND SPECIAL LOADS - MULTILANE LOADING

For Operating rating analysis of floor beams, trusses, non-redundant girders or other non-redundant main structural members, position identical Legal Vehicles in one or more of the through traffic lanes on the bridge, spaced and shifted laterally on the deck so as to produce maximum stress in the member under consideration.

AASHTO reduction factors for multiple lane loadings shall be applied where appropriate. Traffic lanes to be used for rating purposes, shall be the actual marked travel lanes.

When a bridge is to be analyzed for a Special or Superload vehicle in a multilane loading condition, the lanes adjacent to the Special or Superload vehicle lane shall be loaded with MS 18 (HS20-44) Loading to simulate normal traffic on the structure, concurrent with the Special or Superload vehicle.

The Special or Superload vehicle shall laterally be placed in such a lane, concurrent with MS 18 (HS20-44) Loading, to produce maximum stresses in the member under consideration.

When a structure is required to be analyzed for Special or Superload vehicle, a second analysis shall be performed for a single lane loading of Special or Superload vehicle condition. The Special or Superload vehicle shall be placed laterally on the bridge to produce maximum stresses in the critical member under consideration.

902.6 BRIDGE ANALYSIS - MATERIAL STRESSES

Allowable stresses for 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 to conduct specific tests to determine the material strengths or the design plans omit this information.

The rater should review the original design plans as the first source of information for material strengths and stresses. If material strengths are not explicitly stated on the design plans, ODOT Construction and Material Specifications (CMS) applicable at the time of bridge construction shall be reviewed. This may require investigations into old ASTM or AASHTO Material Specifications active at the time of construction.

Table 9-4 provides general information about ODOT Allowable Stresses in bending and shear and material strengths based on the year of construction. These material properties are used as default values in the BARS-PC. Any material stresses and specifications specified on design plans supersede the values in Table 9-4.

The rater is cautioned to pay extra attention to design plans when determining material strengths for structures built during transition years of Table 9-4 (e.g., for SS years 1964-68, or 1988-93, etc.) as materials may have been substituted.

903 AASHTO BARS-PC PROGRAM - CAPABILITIES & REQUIREMENTS
BARS-PC is the PC version of AASHTO BARS (Bridge Analysis and Rating System) program that can analyze and rate bridges based on the current AASHTO Specifications.

The BARS-PC program, detailed User Manuals, and installation instructions are available from ODOT, the Office of Structural Engineering, Structural Rating Engineer, exclusively for use on ODOT projects. Limited technical support to install and execute the program is also available through that office.

903.1 TYPES OF MATERIALS & METHODS OF CONSTRUCTION

The type of materials, methods of construction and type of sections that can be handled by BARS-PC are given in Table 9-2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Method of Construction</th>
<th>Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Composite</td>
<td>variety of sections</td>
</tr>
<tr>
<td></td>
<td>Non-composite</td>
<td></td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Composite</td>
<td>Rectangle</td>
</tr>
<tr>
<td></td>
<td>Non-composite</td>
<td>T-section</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Composite</td>
<td>Box beam</td>
</tr>
<tr>
<td></td>
<td>Non-composite</td>
<td>I-section</td>
</tr>
<tr>
<td>Timber</td>
<td>Non-composite</td>
<td>Rectangle</td>
</tr>
</tbody>
</table>

903.2 TYPES OF STRUCTURES

AASHTO BARS-PC can analyze and rate the following type of bridge members (Table 9-3):

<table>
<thead>
<tr>
<th>Member Type</th>
<th>Method*</th>
<th>Type of Analysis</th>
<th>Spans</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truss - Simple or Cantilever</td>
<td>WS</td>
<td>Axial</td>
<td>1 - 5</td>
</tr>
<tr>
<td>Transverse Floor Beam Simple Span</td>
<td>WS or LF</td>
<td>Bending</td>
<td>Single</td>
</tr>
<tr>
<td>Transverse Floor Beam cantilever on ends</td>
<td>WS</td>
<td>Bending</td>
<td>Single main Span</td>
</tr>
<tr>
<td>Stringer, Beam or Girder</td>
<td>WS or LF</td>
<td>Bending &amp; Shear</td>
<td>1 - 18</td>
</tr>
<tr>
<td>Reinforced Concrete Frame</td>
<td>WS or LF</td>
<td>Bending</td>
<td>1 - 6</td>
</tr>
</tbody>
</table>

* WS: Working Stress  
  LF: Load Factor

903.3 BARS-PC SYSTEM REQUIREMENTS

The following are computer equipment requirements to run BARS-PC:
Hardware Requirements

Minimum configuration

- 386 - 12 MHz with math co-processor
- 6 MB RAM
- 60MB disk drive (uncompressed)
- 3.5" high density diskette drive
- EGA display adapter
- mouse

Optimum configuration

- Pentium or compatible CPU
- 32 MB RAM
- 150 MB free space on hard drive (uncompressed)
- 3.5" disk drive
- VGA or SVGA display adapter
- mouse

Software Requirements (any of the following)

- Windows 3.11 (MS-DOS 5.0 or higher)
- Windows 95
- Windows 98
- Windows NT 3.51 and higher

Three (3) copies of the written Bridge Rating Report shall be submitted to the responsible ODOT District office.

The responsible District Office will review, approve and send a copy of the final Bridge Rating Report to the Office of Structural Engineering, attention: Structure Rating Engineer.

The report must list final Inventory and Operating Ratings of each critical bridge member, overall ratings of each structure unit (mainline, ramps, etc.), and the final ratings of the entire structure. The ratings of each member and the structure shall be presented for each live load vehicle according to Table 9-1.

The Inventory Ratings shall be expressed in terms of the AASHTO-HS loading (English Units), rounded off to nearest single decimal point, not to be listed more than HS20.0, (HS18.5, HS10.6, etc.).

The Operating Ratings for MS18 (HS20-44) Loading shall be expressed in terms of HS (English Units) loading, rounded off to nearest single decimal point, not to be listed more than HS25.0 (HS20.8, HS 23.2, etc.).

The Operating Ratings for Ohio Legal Loads (2S1, 3S1, 4S1, and 5C1) shall be expressed as a percent of the Ohio Legal Load, rounded off to the nearest 5 percent, not to be listed more than 150% (85%, 140%, etc.).

The original design method and loading, used for the design of the bridge and/or specific members (if different for different members or units) shall be explicitly stated in the report.

904 FINAL REPORT SUBMITTAL

904.1 GENERAL
The assumptions made to model a structural member or unit for computer analysis shall be clearly stated in the report.

For existing bridges, the report should include in the narrative, how the material properties and geometry was determined and any specific details about the current conditions. All hand calculations should be part of the report.

904.2 COMPUTER INPUT AND OUTPUT FILES

The hard copies of computer input and output files of bridges analyzed using a computer software shall be included with the report.

If the structure is rated using AASHTO BARS-PC program, the electronic copy of the bridge rating input file on PC-compatible computer disks shall accompany the report.

The District office will send the final copy of the electronic data input file to the Structure Rating Engineer in the Office of Structural Engineering, for transfer to the Bridge Master File.

The Bridge Master File is maintained on the ODOT Central Office Mainframe computers and contains the input data files of all the bridges rated on BARS program. This Master File is used to quickly access the input data files for the future Special and Superload analyses.

If the structure is rated using a computer software other than BARS-PC, consult with the Office of Structural Engineering, Structural Rating Engineer, for specific instructions on submittal of electronic copies of data input file.

905 REFERENCES

APPENDIX  
MISCELLANEOUS BRIDGE INFORMATION

APP-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

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.

Materials. The steel shall conform to ASTM A527M, Grade D, Yield Stress = 280MPa (40,000 psi), Tensile Stress = 386MPa (55,000 psi), or an approved equivalent. The carbon content shall not exceed 0.3%.

Fabrication. The steel sheets shall be fabricated into corrugated plates with a minimum width of 225mm (9 inches) and of a length as indicated on the plans. The corrugations shall be not less than 75mm (3 inches) deep and spaced not more than 225m (9 inches) 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>4.554mm (7 gauge)</td>
<td>0.217</td>
</tr>
<tr>
<td>5.314mm (5 gauge)</td>
<td>0.251</td>
</tr>
<tr>
<td>6.373mm (3 gauge)</td>
<td>0.281</td>
</tr>
<tr>
<td>7.9375mm (5/16&quot;)</td>
<td>0.350</td>
</tr>
<tr>
<td>9.525mm (3/8)</td>
<td>0.403</td>
</tr>
</tbody>
</table>

The plates may be fabricated from two or more sheets buff-spliced end to end, with 300mm (12") 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.

APPENDIX -1-1
Holes shall be provided in every corrugation valley on both sides of the stringer for attaching the plates to the stringer by a 19.05mm (3/4") bolt and approved clip at 225mm (9") centers on alternate sides of the stringer. Holes shall be provided along both 225mm edges of the plate at the midpoint between stringers for attaching the plates together with a 19.05mm (3/4") bolt. Holes shall be provided in each end of the plate at the center for attaching a L50.8mm x 6.35mm (L2" x 2" x 1/4") with a 19.08mm (3/4") bolt to retain a 88.9mm (3-1/2") x 4.554mm (7 gauge) end dam.

**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.

**Method of Measurement.** The quality of flooring paid for shall be the actual square meter (feet) of flooring completed in place and accepted, including all necessary labor, equipment, hardware and incidentals required to complete the item.

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 Meter (Square Feet)</td>
<td>Corrugated Steel Bridge Flooring</td>
</tr>
</tbody>
</table>
APPENDIX -METRIC SEPTEMBER 1998

APP-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 511, CLASS S CONCRETE, USING SHRINKAGE COMPENSATING CEMENT

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 Item 511 shall apply except as modified herein.

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.

Proportions
The maximum water/cement ratio given for Class S concrete in 499.03 shall be revised from 0.44 to 0.50. The Engineer shall make appropriate adjustments in the aggregate batch weights to maintain the yield in accordance with 499.03.

499.031 Proportioning Options, shall not apply to this item.

Slump
Slump at the time and point of concrete placement shall be 75 TO 150mm (3 to 6 inches), except for concrete used to slipform bridge medians and parapets.

Entrained Air
Concrete shall contain 6 plus or minus 2 percent of entrained air at the time and place of concrete placement.
Mixing Concrete
The last sentence of the third paragraph of 499.06. 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.

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 or 4.6 cubic meters (6 cubic yards), whichever is the smaller, 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.

Placing Concrete
Maximum ambient temperature at the time of placement of concrete shall be 27C (80F).

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.

At the Contractor’s option an evaporation retardant and finishing aid may be used after finishing and prior to the texturing operation. Any product used for such purpose shall be specifically marketed for such use (plain water is not acceptable). The product may also be sprayed over textured areas. The evaporation retardant and finishing aid shall be applied as per the manufacturer’s recommendations. The wet burlap cure shall follow this operation as closely as possible.

Decks and slabs shall be given a fog spray of water when the rate of evaporation exceeds 1 kg/sq.m/hour (0.2 lb./sq. ft./hour)(see attached chart). The Contractor shall determine and document the atmospheric conditions, subject to verification by the Engineer. Fogging shall continue until the wet burlap is placed. Fog misting is to keep the environment surrounding the concrete humid to prevent excessive evaporation from the surface of unhardened concrete. Fog misting shall not be used to apply water to the surface of the concrete to facilitate lubrication for finishing purposes. Fogging equipment shall have water pressure systems rated at 17MPa (2400 p.s.i.) or greater and discharge approximately 7 to 12 liters/min. (2 to 3 gallons per minute). Wide angle and sharp angle nozzles shall be used for low wind and windy conditions, respectively.

Silpforming 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 shrinkage compensating concrete as specified in the slipforming proposal note.
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 10 micrometer (4-mil) 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.

Surface Finish
All exposed surfaces of the parapet and vertical faces of deck edges shall have a rubbed finish in accordance with 511.15. Defects shall be corrected prior to rubbing with a Type K cement mortar of the same proportions as the concrete.

Pre-Pour Testing
The Laboratory shall be notified seven (7) days prior to the pour and, if directed by the Laboratory, the supplier shall batch a minimum of 2.3 cubic meters (3 cubic yards) of concrete using Type K shrinkage-compensating cement meeting specification requirements, to be used for pre-pour laboratory testing. The concrete shall be delivered to the job site or batched at the concrete plant to simulate job conditions, as directed by the Laboratory. Upon completion of the laboratory testing, the concrete shall be wasted and disposed of by the supplier off the right-of-way unless otherwise approved by the Engineer. The cost for complying with this requirement shall be in accordance with "Basis of Payment".

Method of Measurement
The quantity shall be measured as per 511 and shall include all labor, materials, equipment and incidentals necessary to complete this item of work

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 Description</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>511</td>
<td>Cubic Meter (Cubic Yd)</td>
<td>Class S Concrete, Superstructure, Using Shrinkage Compensating Cement</td>
</tr>
<tr>
<td>511</td>
<td>Lump</td>
<td>Class S Concrete, Using Shrinkage Compensating Cement, for Pre-Placement Testing</td>
</tr>
</tbody>
</table>
APPENDIX -METRIC SEPTEMBER 1998

APP-3 ARES RETAINING WALLS BY TENSAR The following un-numbered note should be part of any project allowing the use of the Ares wall system. The designer should revise this note to meet project conditions and forward the revised note for inclusion into a project as Special Provisions.

1.1 Description

This work shall consist of the design and construction of Tensar Ares precast panel retaining walls in accordance with this specification. See Section 7.0 for Tensar Ares Retaining Wall design requirements. The design submittal shall include detailed design calculations and a set of drawings describing the proposed retaining wall. The contractor submittal will include a site plan, an elevation drawing of the full length of the retaining wall and representative cross-sections at each design change. Drainage details are to be included as well as detailed construction specifications.

2.0 Materials

The Contractor shall make arrangements to acquire the Tensar Ares Retaining Wall System, which shall include the reinforcing geogrids, precast facing panels, joint materials and all necessary incidentals from Tensar Earth Technologies, Inc., 5775-B Glenridge Drive, Suite 450, Atlanta, GA 30328, Phone 404-250-1290.

The Contractor shall provide certification of the above Ares wall components as per CMS Item 101.061. Certification shall also be provided for the concrete, reinforcing steel, and geogrid reinforcement which are components of the Ares Retaining wall. (Refer to section of the proposal concerning steel produced in the United States.)

2.1 Reinforced Concrete Face Panels

Material used in the concrete face panels shall conform to the requirements of CMS Item 499.02 except as stated in this document. Cement shall be Types I, II, or III and shall conform to the requirements of AASHTO M 85-96. Concrete shall have a compressive strength at 28 days in accordance with Section 2.1.7 Compressive Strength. Concrete shall contain 6 +/- 2 percent air. Air entraining admixture shall conform to AASHTO M 154-89. Retarding agents, accelerating agents or any additive containing chlorides shall not be used without the approval of Tensar Earth Technologies Engineer.

Reinforcing steel and lifting devices shall be set in place to the dimensions and tolerances shown on the plans prior to casting.
2.1.1 Testing and Inspection

Acceptability of the concrete for the precast panels will be determined on the basis of compression tests, certifications, and visual inspection. The concrete strength requirements for the precast panels shall be considered attained regardless of curing age when compression test results indicate strengths conform to the 28 day strength specifications.

The Supplier shall furnish facilities and perform all necessary sampling and testing in an expeditious and satisfactory manner. Panels utilizing Type I or II cement shall be considered acceptable for placement in the wall when the 7-day initial strength tests exceed 85 percent of the 28-day requirements. Panels utilizing Type III cement shall be considered acceptable for placement in the wall prior to 28 days only when the compressive strength test results indicate that the strength exceeds the 28 day specification.

2.1.2 Casting

The panels shall be cast on a flat area, with the front face down. The geogrid retention devices, reinforcement, inserts, lifting devices shall be set in place to the dimensions and tolerance shown on the drawings, prior to casting. The concrete in each unit shall be placed without interruption and shall be consolidated by the use of an approved vibrator, supplemented by such hand-tamping as may be necessary to force the concrete into the corners of the forms and prevent the formation of stone pockets, air bubbles or cleavage planes. Clear form oil from the same manufacturer shall be used throughout the casting operation as approved by the Tensar Earth Technologies Engineer.

Special care shall be given to the geogrid retention devices. The geogrid retention devices shall be positioned within 25 mm (1 inch) of their specified locations immediately after placement and consolidation of the concrete using the locator tabs on the side rails. The geogrid retention devices shall be rapped with a rubber mallet or other approved hand compaction method to ensure consolidation in and around the sleeve. No concrete or other debris shall be on the exposed portion of the geogrid retention devices in the finished panels.

2.1.3 Curing

The curing method will be dependent on the precaster’s capabilities and shall be per the instructions of Tensar Earth Technologies. The reinforced concrete panels shall be cured for a sufficient length of time so that the concrete will develop the specified compressive strength. Any production lot which does not conform to the strength requirements of Section 2.1.7-Compressive Strength, shall be rejected.
2.1.4 **Removal of Forms**

The forms, including the geogrid retention devices, shall remain in place until they can be removed without damage to the panel.

2.1.5 **Concrete Finish and Tolerances**

The front face of the reinforced concrete panels shall have an architectural surface finish treatment as shown on the plans. The rear face of the reinforced concrete panels shall have an unformed surface finish and shall be rough screeded to eliminate open pockets of aggregate and surface distortions in excess of 6 mm (1/4 inch). All panels, barriers and coping shall be sealed with an epoxy sealer in conformance with Item Special “Sealing of Concrete Surfaces, Epoxy”, unless special sealers have been called for on the plans.

2.1.6 **Tolerances**

All panels shall be manufactured within the following tolerances:

- **Panel Dimensions**
  
  Vertical position of the slot - within 25 mm (1 inch). All other dimensions - within 5 mm (0.2 inch).

- **Panel Squareness**
  
  Squareness, as determined by the difference between the two diagonals, shall not exceed 13 mm (½ inch).

- **Panel Surface Finish**
  
  Surface defects on smooth formed surfaces measured across a length of 1500 mm (5 feet) shall not exceed 3 mm (0.12 inch). Surface defects on textured finished surfaces measured across a length of 1500 mm (5 feet) shall not exceed 8 mm (0.312 inch).

- **Slot Opening**
  
  The opening in the slot shall be 3 mm (0.12 inch), and shall be tested for compliance on each panel cast using a feeler gauge supplied to the precaster by Tensar. A deviation which prevent insertion of the geogrid or which permits the feeler gauge to be pulled through the slot shall be cause for rejection of the panel.
2.1.7 **Compressive Strength**

Acceptance of the concrete panels with respect to compressive strength will be determined on the basis of production lots. A production lot is defined as a group of panels that will be represented by a single compressive strength sample and will consist of either 40 panels or a single day’s production, whichever is less.

During the production of the concrete panels, the manufacturer will randomly sample the concrete in accordance with AASHTO T 141-93. A single compressive strength sample, consisting of a minimum of four test cylinders, will be randomly selected for every production lot.

Cylinders for compressive strength tests shall be 150 mm (6 inch) by 300 mm (12 inch) specimens prepared in accordance with AASHTO T 23-93. For every compressive strength sample, a minimum of two cylinders will be cured in the same manner as the panels and tested at approximately 7 days. The average compressive strength of these cylinders, when tested in accordance with AASHTO T 22-92 will provide a test result which will determine the initial strength of the concrete. In addition, two cylinders shall be cured in accordance with AASHTO T 23-93 and tested at 28 days. The average compressive strength of these two cylinders, when tested in accordance with AASHTO T 22-92 will provide a compressive strength test result which will determine the compressive strength of the production lot.

If the initial strength test results indicate a compressive strength in excess of 31.0 MPa (5000 psi), then these test results will be utilized as the compressive strength test result for that production lot and the requirements for testing at 28 days will be waived for that particular production lot.

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 31.0 MPa (5000 psi). If the compressive strength test result is less than 31.0 MPa (5000 psi), the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

a. Ninety percent of the compressive strength test results for the overall production shall exceed 31.0 MPa (5000 psi).

b. The average of any six consecutive compressive strength test results shall exceed 31.0 MPa (5000 psi).

c. No individual compressive strength test result shall fall below 27.5 MPa (4000 psi).

In the event that a production lot fails to meet the specified compressive strength requirements, the production lot shall be rejected. Such rejection shall prevail unless the Manufacturer, at his own expense, obtains and submits evidence acceptable to the Engineer that the strength and quality of the production lot meet the specified requirements.
concrete placed within the panels of the production lot is acceptable. If such evidence consists of tests made on cores taken from the panels within the production lot, the cores shall be obtained and tested in accordance with the specifications of AASHTO T 24-93.

2.1.8 Rejection

Panels shall be subject to rejection because of failure to meet any of the requirements specified above. In addition, any of the following defects shall be sufficient cause for rejection:

a. Defects that indicate imperfect molding.

b. Defects indicating honeycombed or open texture concrete.

c. Defects in the physical characteristics of the concrete, such as broken or chipped concrete.

d. Stained form face, due to excess form oil or other contaminants.

e. Signs of aggregate segregation

f. Broken or cracked corners.

g. Slots plugged and unusable or those which failed the width test.

h. Lifting inserts not useable.

i. Exposed reinforcing steel.

j. Cracks at retention devices.

k. Insufficient concrete compressive strength.

l. Panel thickness in excess of 5 mm (3/16 inch) from that shown on the plans.

The Engineer will decide if an attempt may be made to repair a defective panel. The contractor or supplier shall make the repairs at his own expense. If the repairs are made to the Engineer's satisfaction, the panel will be acceptable.

2.1.9 Marking

The date of manufacture, the production lot number and the piece-mark, shall be clearly scribed on the back surface of each panel.
2.1.10  **Handling, Storage and Shipping**

All panels shall be handled, stored, and shipped in such a manner as to avoid chipping, cracking, fracturing and excessive bending stresses. Panels shall be supported on firm blocking while in storage.

2.1.11  **Reinforcing Steel**

The reinforcing steel shall be Grade 420 (Grade 60) epoxy coated and shall satisfy all applicable requirements of CMS Item 509, Reinforcing Steel.

2.2  **Reinforcing Geogrids**

The geogrids shall consist of Tensar structural geogrids formed by uniaxially drawing continuous sheets of select high density polyethylene material and shall have aperture geometry and rib and junction cross-sections sufficient to permit significant mechanical interlock with the material being reinforced and with the connection device embedded in the panels. The geogrid shall have high flexural rigidity, high tensile modulus in relation to the material being reinforced and shall also have high continuity of tensile strength through all ribs and junctions of the grid structure to develop the full long term design strength of the geogrid at the panel connection. The geogrids shall have high resistance to deformation under sustained long term design load while in service and shall also be resistant to ultraviolet degradation, to damage under normal construction practices and to all forms of biological and/or chemical degradation normally encountered in the material being reinforced.

The contractor shall check the geogrid upon delivery to ensure that the proper material has been received, and is free from defects that may impair its strength and durability. The geogrids shall be stored in conditions above -20 degrees F (-29 degrees C) and not greater than 140 degrees F (60 degrees C). The contractor shall prevent excessive mud, wet concrete, epoxy, and like materials from coming into contact with and affixing to the geogrid material. Geogrid rolls may be laid flat or stood on end for storage.

2.3  **Joint Materials**

2.3.1  **Bearing Pads**

Elastomeric or EPDM bearing pads shall be the type and grade recommended by Tensar Earth Technologies.
2.3.2 Joint Cover

As required on the plans, cover for the horizontal and vertical joints between panels, and at other specified locations, shall be polypropylene fabric such as TG650 or an Engineer approved equal. The adhesive used to attach the fabric shall be Pliobond 5001 or equal as approved by the Tensar Earth Technologies Engineer.

2.4 Select Granular Embankment Material

The select granular embankment material used in the reinforced structure volume shall be Item 304 Aggregate Base or Item 703.11 Granular Material Type 2; deviations from these are as follows:

1) No slag shall be allowed.

2) The total passing the 0.75 mm (number 200) sieve shall be 0 to 7 percent prior to the placement operation. The acceptance samples shall be taken from the stockpile. The Contractor shall transport and handle the material to minimize the segregation of the material prior to the placement. No slag shall be allowed.

The select granular embankment materials shall also conform to the requirements listed below:

1) AASHTO T 289-91 - The pH range of the material shall be between 4.5 and 9.0.

The Contractor or supplier, as his agent, shall furnish the Engineer a Certificate of Compliance from an independent Testing Laboratory, certifying that the above materials comply with the applicable contract specifications. A copy of all test results performed by the contractor or his supplier, necessary to assure contract compliance shall be furnished to the Engineer.

Acceptance will be based on a letter of certification that material to be utilized is within the parameters of the applicable contract notes and specifications. This letter shall be submitted along with the accompanying test reports to the Project Engineer. The Engineer shall provide written acceptance of material upon review of the accompanying test reports, and visual inspection of the material by the Engineer.

2.5 Leveling Pad Concrete

The leveling pad shall be constructed of 150 mm (6 inch) thick concrete having a compressive strength not less than 17.2 MPa (2500 psi) and shall have sufficient strength to adequately support the panels at the bottom of the wall in a level position during installation. The leveling pad may be constructed of precast concrete. Precast concrete leveling pads are advantageous to use when many short segment leveling pad steps are required.
2.6 Filter Fabric and Porous Backfill

As per Item 518, a 150 mm (6 inch) perforated plastic pipe shall be placed below the bottom row of geogrid reinforcement. One drain pipe shall be located behind the leveling pad and the other shall be at opposite end of the geogrid reinforcement. The pipe shall be continuous and be placed in the bottom portion of reinforced fill. Positive gravity flow of the drainpipe shall be provided. Perforations in the pipe shall be smaller than the surrounding porous backfill. Filter fabric shall be installed at the required plan locations and shall satisfy the requirements of CMS Item 712.09, Type B.

3.0 Construction Requirements

3.1 Wall Excavation

Unclassified excavation shall be in accordance with ODOT Item 503 except that the limits of excavation shall be shown in the plans.

3.2 Foundation Preparation

The foundation for support of the reinforced select granular embankment volume shall be graded level for a width equal to or exceeding the length of reinforcing geogrid. Prior to wall construction, the foundation soil shall be level and rolled with a smooth wheel vibratory roller and the exposed soil compacted to at least 95% Standard Proctor density. Any foundation soils found to be unsuitable shall be removed and replaced as directed by the Engineer.

The unreinforced concrete leveling pad shall be provided as shown on the plans. The leveling pad shall be cured a minimum of 12 hours before placement of wall facing panels.

3.3 Wall Erection

Precast concrete panels shall be placed as shown on the drawings. For erection, panels shall be handled by means of a lifting device set into the upper edge of the panels. Panels shall be set in successive horizontal lifts in the sequence shown on the plans, as backfill placement proceeds. As backfill material is placed behind the panels, the panels shall be maintained in vertical position by means of temporary wooden wedges placed in the panel joints on the external side of the wall. The wooden wedges shall be removed as soon as the panel above the wedged panel is completely erected and backfilled. External bracing is required for the initial lift. For vertical walls, vertical tolerances (plumbness) and horizontal alignment tolerances shall not exceed 20 mm (3/4 inch) when measured along 3000 mm (10 feet) straight edge. The maximum allowable offset in any panel joint shall be 20 mm (3/4 inch). For vertical walls, the overall vertical tolerance of the wall (plumbness from top to bottom) shall not exceed 13 mm (½ inch) per 3000 mm (10 feet) of wall height.
Reinforcing geogrids shall be placed normal to the wall face, unless otherwise shown on the plans or as directed by the Engineer. The geogrid shall be installed in the slot in strict accordance with the manufacturer’s construction guidelines. The reinforcing grid shall be continuous, from wall panel to end of reinforcing zone. No splicing of geogrid will be allowed. Prior to placement of the reinforcing geogrid, backfill shall be compacted in accordance with Section 3.4. Construction equipment shall not operate directly on the geogrid. Before the placement of any backfill over the geogrid, the reinforcing geogrids shall be pulled taut perpendicular to the orientation of the geogrid with enough force to eliminate wrinkles or folds in the geogrid.

3.4 Select Granular Embankment Material Placement

Select granular embankment material placement shall closely follow the erection of each lift of panels. At each geogrid level, the select granular embankment material should be roughly leveled before extending the geogrid over the fill. Reinforcing geogrids shall be installed normal to the wall unless the plans show otherwise to avoid an obstruction. The geogrids shall be pulled taut, by a method approved by the wall supplier, prior to placing the select granular embankment material. The maximum select granular embankment lift thickness shall not exceed 200 mm (8 inch) loose and shall closely follow panel erection. The Contractor shall decrease the select granular embankment lift thickness, if necessary, to obtain the specified density.

At the end of each day's operations, the Contractor shall shape the last level of embankment to rapidly direct rain water runoff away from the wall face. The Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Compaction of the select granular embankment material shall be in accordance with all pertinent requirements of Item 203.12. The moisture content of the select granular embankment material prior to and during compaction shall be uniformly distributed throughout each layer and shall conform to requirements of Item 203.11.

Embankment compaction of the reinforced zone of fill shall be accomplished without disturbance or distortion of reinforcing geogrids and panels. Compaction within 1000 mm (3 feet) of the backside of the wall panels shall be achieved by at least 3 passes of a light mechanical tamper. The embankment placed within 1000 mm (3 feet) of the backside of the panels does not have to satisfy density test requirements, but must be placed in 150 mm (6 inch) to 200 mm (8 inch) thick lifts which are compacted per above.

4.0 Inspection

Tensar Earth Technologies shall provide a company representative who shall monitor the precast operation to ensure that the operation complies with all the above requirements and the representative shall submit documentation to this effect to the Engineer.
Documentation shall be furnished by the company representative for any panel which he deems acceptable but which is subject to rejection under 2.1.8.

Tensar Earth Technologies shall provide sufficient on-site technical assistance by a company representative to ensure that the Contractor and the Engineer fully understand the proper construction procedures and operations of the Ares Retaining Wall System.

The Contractor shall provide a soils consultant who shall, through the Engineer, be responsible for inspection and testing to confirm that the select granular embankment material, placement and compaction are in compliance with this specification.

The soils consultant shall provide the Engineer with two copies of all inspection reports which contain the testing results, all pertinent measurements and the soil consultant’s conclusions.

The soil consultant’s field representative shall be an Ohio Registered Professional Engineer or work under the direction of an Ohio Registered Professional Engineer. The final inspection report shall be signed by an Ohio Registered Professional Engineer.

The Engineer will monitor the erection of the structure and placement of the select granular embankment material with the technical assistance from the soils consultant and Tensar Earth Technologies. Any work not satisfying the requirements of this special provision is subject to rejection by the Engineer.

5.0 Coping and Traffic Barrier

A cast-in-place coping and traffic barrier shall be provided on the top of the wall as shown in the plans. Concrete shall be Class C and shall conform to CMS Item 511. Reinforcing steel shall be epoxy coated and shall conform to CMS Item 509. For the payment for the concrete and reinforcing steel, see 8.1 “Basis of Payment”.
6.0 Pile Sleeves

When piles are required, sleeves shall be used to form a void in the select granular embankment material so that the piles can be installed after the wall construction has been completed. The sleeve material shall be satisfactory to the Tensar Earth Technologies. Consider using segments of plastic pipe strong enough to maintain the required void. A bentonite slurry shall be placed in the void located between the pile and the sleeve. The slurry shall consist of the following materials with the volume ratios of one part cement, one part bentonite, and ten parts water. The cost of the above described work shall be considered incidental to Item Special “Ares Retaining Walls.”

7.0 Design Requirements of MSE Panel Walls

The design of the Tensar Ares Retaining Wall shall be in strict conformance with the 1996 edition of 'AASHTO Standard Specifications for Highways and Bridges', including the 1997 and 1998 Interims, except as modified in this document; the latest edition of the Ohio Bridge Design Manual, except as modified in this document; and the design requirements listed below:

1. The design shall meet all of the plan requirements. The recommendations of the wall system suppliers shall not override the minimum performance requirements shown herein. Other systems offered by the approved supplier shall not be submitted in lieu of the system which is called for in the plans.

2. Where walls or wall sections intersect with an included angle of 130 degrees or less, a vertical corner element, separate from the standard panel face, shall abut and interact with the opposing standard panels. The corner element shall have geogrid reinforcement connected specifically to that panel and shall be designed to preclude lateral spread of the intersecting panels.

3. One hundred percent of the geogrid reinforcement designed and placed in the reinforced earth volume shall extend to and be connected to the facing element through the use of sleeve or another acceptable method. Field cutting of the geogrid reinforcement where it interferes with an obstacle is permitted, provided that Tensar and the Engineer approve of the cutting and a detail for the partial removal of the geogrid has been designed and shown in the plans.

4. Under service loads the minimum factor of safety at the connection between the face panel and the geogrid reinforcement shall be 1.5 as defined in the 1997 and 1998 AASHTO Interims 5.8.7.2. The minimum factor of safety against reinforcement pullout shall be 1.5 at 13 mm (½ inch) deformation. The maximum allowable reinforcement tension shall not exceed the LTDS of the geogrid, using criteria set by the AASHTO Standard Specifications.
5. The coefficient of lateral earth pressure $K_a$ and the application of the lateral forces to the reinforced soil mass for external stability analysis shall be computed using the Coulomb method, but assuming no wall friction.

6. Soil parameters for use in design are as follows:

<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>Compacted Select Granular Embankment Material with less than 7% P200 Material</td>
<td>18.9 kN/m$^3$ (120 lbs/ft$^3$)</td>
<td>34°</td>
<td>0</td>
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<tr>
<td>Retained Soil</td>
<td>On-site soil varying from sandy lean clay to silty sand</td>
<td>18.9 kN/m$^3$ (120 lbs/ft$^3$)</td>
<td>30°</td>
<td>0</td>
</tr>
<tr>
<td>Foundation Soil</td>
<td>Variable - ranging from existing uncontrolled fill to natural silty sand</td>
<td>18.9 kN/m$^3$ (120 lbs/ft$^3$)</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

** Refer to ODOT Bridge Design Manual 303.4.1.1

7. All walls shall be designed with coping or traffic barrier as specified in the plans.

8. The allowable reinforcement tension for polymeric (extensible) reinforcement shall be based on AASHTO Section 5.8.7.2. The following reduction factors shall be applied to the Tensar HDPE geogrid soil reinforcements for determination of the allowable long term design strength, $T_{al}$, based on the type of backfill used in the reinforced zone.

<table>
<thead>
<tr>
<th>TENSAR STRUCTURAL GEOGRIDS FOR ARES WALLS, AASHTO STANDARD SPECIFICATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{all} = T_{ult} / RF_{cr} x RF_{id} x RF_{fd}$</td>
</tr>
<tr>
<td>Ultimate Strength (kN/m$^2$)</td>
</tr>
<tr>
<td>131.3</td>
</tr>
<tr>
<td>Calculated Creep Reduction Factor, $RF_{cr}$</td>
</tr>
<tr>
<td>Durability, $RF_{id}$</td>
</tr>
<tr>
<td>Installation Damage, $RF_{fd}$</td>
</tr>
</tbody>
</table>

9. The design life of the Tensar Ares Retaining Wall shall be 100 years.
10. The minimum depth of embedment, measured from the finished ground line to the top of the leveling pad, shall be at least 1065 mm (3.5 feet), as shown on plan.

11. The internal stability, including the definition of the failure plane and the lateral earth pressure coefficient, for Tensar Ares retaining walls with extensible reinforcement shall be computed according to AASHTO Section 5.8.4.1, including the 1998 Interims which permit the use of the Coherent Gravity Method for design.

12. The maximum thickness of the concrete leveling pad shall be 150 mm (6 inch).

13. The connections for the geogrid reinforcement to the panels shall be in two places for standard panels and the connections shall be no more than 760 mm (2.5 feet) apart vertically.

14. The wall height for design purposes shall be measured from the top of the leveling pad to the top of the coping. When the wall is retaining a sloping surcharge then the wall height shall be defined as the equivalent design height \( h \) as shown in AASHTO Figure 5.8.2B. The minimum geogrid reinforcement length shall be 70 percent of the wall height, as appropriately defined for either a level or sloping backfill, but no less than 2440 mm (8 feet).

15. The minimum thickness for the precast reinforced concrete panels shall be 140 mm (5 ½ inch).

16. The wall system, regardless of the size of the panels shall accommodate up to one percent differential settlement in the longitudinal direction.

17. The vertical stress at each reinforcement level shall be computed by considering local equilibrium of all the forces acting above the level under investigation. The vertical stress (bearing pressure) at each reinforcement level may be computed using the Meyerhoff method in the same manner that the bearing pressure is computed at the base.

18. Design shall include a dynamic horizontal thrust using an acceleration, \( A \), of 0.06g to simulate earthquake loading in Cincinnati, Ohio.

8.0 Method of Measurement

The Mechanically Stabilized Earth Wall quantity to be paid for shall be the actual number of square feet of facial area of approved Tensar Ares Retaining wall panels complete in place. The total quantity is the computed and accepted front face wall area within the limits of the beginning and ending wall stations and from the top of the wall leveling pad to the top of the coping or barrier.
Item 503 Unclassified Excavation is defined in section 3.1.

8.1 Basis of Payment

Item Special "Retaining Wall, Misc.: Ares Retaining Wall by Tensar" will be paid for at the contract unit price per square meter (foot). This work shall include the fabrication, furnishing, and erection of the Ares retaining walls, including the reinforced concrete face panels, reinforcing geogrids, joint material, concrete leveling pad, concrete coping, concrete barrier, reinforcing steel, filter fabric, and all incidentals, labor, and equipment necessary to complete this item.

Item 203 "Select Granular Embankment, as per plan" will be paid for at the contract unit price per cubic meter (yard). Payment for the soils consultant’s work shall be included with this item.

Item 203 “Embankment, as per plan” will be paid for at the contract unit price per cubic meter (yard).

Item 503 “Unclassified Excavation, as per plan” will be paid for at the contract unit price per cubic meter (yard).

Item 304 “Aggregate Base” will be paid for at the contract unit price per cubic meter (yard).

Item 518 "150 mm (6 inch) Perforated Corrugated Plastic Pipe" will be paid for at the contract unit price per linear meter (foot).

Item 518 "150 mm (6 inch) Non-perforated Corrugated Plastic Pipe, including specials" will be paid for at the contract unit price per linear meter (foot).

Item Special "Sealing of Concrete Surfaces, Epoxy-Urethane" will be paid for at the contract unit price per square meter (yard).
REINFORCED EARTH WALLS

1.0 Description

This work consists of the design and construction of Reinforced Earth Retaining Walls in accordance with this specification. See Section 7.0 for Reinforced Earth Retaining Wall Design requirements. The design submittal shall include detailed design calculations and a set of drawings describing the proposed retaining wall. The contractor submittal will include a site plan, an elevation drawing of the full length of the retaining wall and representative cross-sections at each design change. Drainage details are to be included as well as detailed construction specifications.

2.0 Materials

The Contractor shall make arrangements to acquire the reinforced concrete face panels, galvanized steel reinforcing strips, fasteners, joint materials and all necessary incidentals from the Reinforced Earth Company, 760 Pasquinelli Drive, Suite 344, Westmont, Illinois 60559 (Telephone: 630-655-0044) or at 8614 Westwood Center Drive, Suite 1100, Vienna, Virginia 22182 (703-821-1175).

The Contractor shall provide certification of the Reinforced Earth wall components as per CMS 101.061. Certification shall also be provided for the concrete, reinforcing steel and tie strips which are components of the Reinforced Earth wall. (Refer to the section of the proposal concerning steel produced in the United States.)

2.1 Reinforced Concrete Face Panels

Material used in concrete for face panels shall conform to the requirements of CMS Item 499.02 except as stated in this document. Cement shall be Types I, II, or III and shall conform to the requirements of AASHTO M 85-96. Concrete shall have a compressive strength at 28 days in accordance with Section 2.1.7 Compressive Strength. Concrete shall contain 6 ± 2 percent air. Air entraining admixture shall conform to AASHTO M 154-89. Retarding agents, accelerating agents or any additive containing chloride shall not be used without approval from the Engineer.

Reinforcing steel, tie strips, 19 mm (3/4 inch) dowels, PVC pipe, and lifting inserts shall be set in place to the dimensions and tolerances shown on the plans prior to casting.
2.1.1. Testing and Inspection

Acceptability of the concrete for the precast panels will be determined on the basis of compression tests, certifications and visual inspection. The concrete strength requirements for the precast panels shall be considered attained regardless of curing age when compression test results indicate that strengths conform to 28-day specifications.

The Supplier shall furnish facilities and perform all necessary sampling and testing in an expeditious and satisfactory manner. Panels utilizing Type I or II cement shall be considered acceptable for placement in the wall when 7-day initial strengths exceed 85 percent of 28-day requirements. Panels utilizing Type III cement shall be considered acceptable for placement in the wall prior to 28 days only when compressive strength test results indicate that the strength exceeds the 28 day specification.

2.1.2. Casting

The panels shall be cast on a flat area with the front face down. Tie strip guides shall be set on the backside. The concrete in each panel shall be placed without interruption and shall be consolidated by the use of a concrete vibrator, supplemented by such hand-tamping as may be necessary to force the concrete into the corners of the forms and prevent the formation of stone pockets, air bubbles or cleavage planes. Clear form oil of the same manufacture shall be used throughout the casting operation and shall be approved by the Reinforced Earth Engineer.

Special care shall be given to the tie strips. The tie strips shall be positioned to within 25 mm (1 inch) of their specified locations. No concrete or debris shall be permitted between the projecting tabs of the tie strips which could interfere with the installation and bolting of the reinforcing strips.

2.1.3. Curing

The curing method will be dependent on the concrete precaster’s capabilities and shall be as per instructions by the Reinforced Earth Company. The reinforced concrete panels shall be cured for a sufficient length of time so that the concrete will develop the specified compressive strength.

Any production lot which does not conform to the strength requirements of Section 2.1.7-Compressive Strength, shall be rejected.
2.1.4. **Removal of Forms**

The forms shall remain in place until they can be removed without damage to the panel.

2.1.5. **Concrete Finish and Tolerances**

The front face of the reinforced concrete panels shall have an architectural surface finish treatment as shown on the plans. The backside of the reinforced concrete panels shall have an unformed surface finish and shall be rough screeded to eliminate open pockets of aggregate and surface distortions in excess of 6 mm (1/4 inch). All panels, barriers and coping shall be sealed with an epoxy sealer in conformance with Item Special “Sealing of Concrete Surfaces, Epoxy”, unless special sealers have been called for on the plans.

2.1.6. **Tolerances**

All panels shall be manufactured within the following tolerances:

a. **Panel Dimensions**

   Lateral position of tie strips within 25 mm (1 inch). All other dimensions within 5 mm (0.2 inch).

b. **Panel Squareness**

   Squareness as determined by the difference between the two diagonals shall not exceed 13 mm (½ inch).

c. **Panel Surface Finish**

   Surface defects on smooth formed surfaces measured on a length of 1500 mm (5 feet) shall not exceed 3 mm (0.12 inch). Surface defects on textured finished surfaces measured on a length of 1500 mm (5 feet) shall not exceed 8 mm (0.312 inch).

2.1.7. **Compressive Strength**

Acceptance of the concrete panels with respect to compressive strength will be determined on the basis of production lots. A production lot is defined as a group of panels that will be represented by a single compressive strength sample and will consist of either 40 panels or a single day’s production, whichever is less.
During the production of the concrete panels, the manufacturer will randomly sample the concrete in accordance with AASHTO T 141-93. A single compressive strength sample, consisting of a minimum of four test cylinders, will be randomly selected for every production lot.

Cylinders for compressive strength tests shall be 150 mm (6 inch) by 300 mm (12 inch) specimens prepared in accordance with AASHTO T 23-93. For every compressive strength sample, a minimum of two cylinders will be cured in the same manner as the panels and tested at approximately 7 days. The average compressive strength of these cylinders, when tested in accordance with AASHTO T 22-92, will provide a test result which will determine the initial strength of the concrete. In addition, two cylinders shall be cured in accordance with AASHTO T 23-93 and tested at 28 days. The average compressive strength of these two cylinders, when tested in accordance with AASHTO T 22-92, will provide a compressive strength test result which will determine the compressive strength of the production lot.

If the initial strength test results indicate a compressive strength in excess of 27.5 MPa (4000 psi), then these test results will be utilized as the compressive strength test result for that production lot and the requirement for testing at 28 days will be waived for that particular production lot.

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 27.5 MPa (4000 psi). If the compressive strength test result is less than 27.5 MPa (4000 psi), the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

a. 90% of the compressive strength test results for the overall production shall exceed 27.5 MPa (4000 psi).

b. The average of any six consecutive compressive strength test results shall exceed 27.5 MPa (4000 psi).

c. No individual compressive strength test result shall fall below 24.8 MPa (3600 psi).

In the event that a production lot fails to meet the specified compressive strength requirements, the production lot shall be rejected. Such rejection shall prevail unless the Manufacturer, at his own expense, obtains and submits evidence acceptable to the Engineer that the strength and quality of the concrete placed within the panels of the production lot is acceptable. If such evidence consists of tests made on cores taken from the panels within the production lot, the cores shall be obtained and tested in accordance with the specifications of AASHTO T 24-93.
2.1.8. **Rejection**

Panels shall be subject to rejection because of failure to meet any of the requirements specified above. In addition, any or all of the following defects may be sufficient cause for rejection:

a. Defects that indicate imperfect molding.
b. Defects indicating honeycombed or open textured concrete.
c. Defects in the physical characteristics of the concrete, such as broken or chipped concrete.
d. Stained form face, due to excess form oil or other contaminations.
e. Signs of aggregate segregation.
f. Broken or cracked corners.
g. Tie strips bent or damaged.
h. Lifting inserts not usable.
i. Exposed reinforcing steel.
j. Cracks at the PVC pipe or pin.
k. Insufficient concrete compressive strength.
l. Panel thickness in excess of 5 mm (3/16 inch) plus or minus from that shown on the plans.

The Engineer will decide if an attempt may be made to repair a defective panel. The contractor or supplier shall make the repairs at his own expense. If the repairs are made to the Engineer's satisfaction, the panel will be acceptable.

2.1.9. **Marking**

The date of manufacture, the production lot number and the piece mark shall be clearly scribed on the back surface of each panel.
2.1.10. Handling, Storage and Shipping

All panels shall be handled, stored, and shipped in such a manner as to avoid chipping, cracking, fracturing, and excessive bending stresses. To avoid bending the tie strips the panels in storage shall be supported on firm blocking located immediately adjacent to the strips.

2.1.11. Reinforcing Steel

The reinforcing steel shall be Grade 420 (Grade 60) epoxy coated and shall satisfy all applicable requirements of CMS 509, reinforcing steel.

2.2 Reinforcing Strips and Tie Strips

Tie strips shall be shop-fabricated from hot rolled steel conforming to the requirements of ASTM A 570M, Grade 345 (Grade 50) or equivalent.

Reinforcing strips shall be hot rolled from bars to the required shape and dimensions. The physical and mechanical properties of the reinforcing strips shall conform to ASTM A 572M Grade 450 (Grade 65) or equivalent.

Reinforcing strips and tie strips shall be galvanized in conformance with ASTM A 123.

All reinforcing strips and tie strips shall be carefully inspected to ensure they are true to size and free from defects that may impair their strength and durability.

2.3 Fasteners

Fasteners shall be 13 mm (½ inch) diameter, hexagonal cap screw bolts and nuts, which are galvanized and conform to ASTM A325M.

2.4 Joint Materials

2.4.1. Bearing Pads

Rubber bearing pads shall be a type and grade recommended by the Reinforced Earth Company, such as "Bearing Blok" or equivalent.

2.4.2. Inclined, Vertical and Horizontal Joints

The material to be attached to the rear side of the facing panels covering the vertical and horizontal joints between facing panels, shall be a polypropylene filter fabric, such as Q-Trans 80, or equal
as approved by the Engineer. The adhesive used to attach the fabric material to the rear of the facing panel shall be Pliobond 5001, as manufactured by Goodyear Rubber Company, or equal as approved by the Reinforced Earth Engineer.

2.5 Dowels

The 19 mm (3/4 inch) dowel bars shall be made of a material that is specified by the Reinforced Earth Company.

2.6 Select Granular Embankment Material

The select granular embankment material used in the reinforced structure volume shall be Item 304 Aggregate Base or Item 703.11, Granular Material Type 2; deviations from these are as follows:

1) No slag shall be allowed.

2) The total percent passing the 0.075 mm (number 200) sieve shall be 0 to 7 percent prior to the placement operation. The acceptance samples shall be taken from the stockpile. The Contractor shall transport and handle the material to minimize the segregation of the material prior to the placement.

The select granular embankment material shall also conform to the requirements listed below:

1) AASHTO T 289-91 The pH range of the material shall be between 5.0 and 10.0.

2) AASHTO T 288-91 The resistivity of the material shall be greater than 3,000 ohm-cm. If the resistivity is found to be greater than 5,000 ohm-cm, then AASHTO Test T 290-95 and T 291-94 may be waived.

3) AASHTO T 291-94 The chloride levels of the material shall be less than 100 ppm.

4) AASHTO T 290-95 The sulfate levels of the material shall be less than 200 ppm.

The Contractor or supplier, as his agent, shall furnish the Engineer a Certificate of Compliance from an independent Testing Laboratory, certifying that the above materials comply with the applicable contract specifications. A copy of all test results performed by the contractor or his supplier necessary to assure contract compliance shall also be furnished to the Engineer.
Acceptance will be based on a letter of certification that the material to be utilized is within the parameters of the applicable contract notes and specifications. This letter shall be submitted along with the accompanying test reports to the Project Engineer. The Engineer shall provide written acceptance of material upon review of accompanying test reports, and visual inspection of the material by the Engineer.

2.7 Leveling Pad Concrete

The leveling pad concrete shall be a maximum 150 mm (6 inch) thick, having a compressive strength not less than 17.2 MPa (2500 psi) and be of sufficient strength to adequately support the panels at the bottom of the wall in a level position during installation. The leveling pad may be constructed of precast concrete. Precast concrete leveling pads are advantageous to use when many short segment leveling pad steps are required.

2.8 Filter Fabric and Porous Backfill

As per Item 518, a 150 mm (6 inch) perforated plastic pipe shall be placed below the bottom row of reinforcing strips. One drain pipe shall be located behind the leveling pad and the other shall be at opposite end of the reinforcing strips. The pipe shall be continuous and be placed in the bottom portion of the reinforced fill. Positive gravity flow of the drainpipe shall be provided. Perforations in the pipe shall be smaller than the surrounding porous backfill. Filter fabric shall be installed at the required plan locations and shall satisfy the requirements of CMS Item 712.09, Type B.

3.0 Construction Requirements

3.1 Wall Excavation

Unclassified excavation shall be in accordance with ODOT Item 503 except that the limits of excavation shall be as shown in the plans.

3.2 Foundation Preparation

The foundation for support of the reinforced select granular embankment volume shall be graded level for a width equal to or exceeding the length of reinforcing strips. Prior to wall construction, the foundation soil shall be leveled, rolled with a smooth wheel vibratory roller and the exposed soil compacted to at least 95% Standard Proctor density. Any foundation soils found to be unsuitable shall be removed and replaced, as directed by the Engineer.

The unreinforced concrete leveling pad shall be provided as shown on the plans. The leveling pad shall be cured a minimum of 12 hours before placement of the wall facing panels.
3.3 Wall Erection

Precast panels are handled by means of a lifting device connected to the lifting insert which is cast into the upper edge of the panels. Panels shall be installed in successive horizontal lifts in the sequence shown on the plans as embankment placement proceeds. As embankment material is placed behind panels, the panels shall be maintained in vertical position by means of temporary wooden wedges placed in the panel joints on the external side of the wall. External bracing may be required for the initial lift. Vertical tolerance (plumbness) and horizontal alignment tolerance shall not exceed 20 mm (3/4 inch) when measured along a 3000 mm (10 feet) straight edge. The maximum allowable offset in any panel joint shall be 20 mm (3/4 inch). The overall vertical tolerance of the wall (plumbness from top to bottom) shall not exceed 13 mm (½ inch) per 3000 mm (10 feet) of wall height.

Reinforcing strips shall be placed normal to the face of the wall, unless otherwise shown on the plans or directed by the Engineer. Prior to placement of the reinforcing strips, embankment shall be compacted in accordance with Section 3.4.

3.4 Select Granular Embankment Material Placement

Select granular embankment material placement shall closely follow the erection of each lift of panels. At each reinforcing strip level, the select granular embankment material should be roughly leveled before placing and bolting the reinforcing strips. Reinforcing strips shall be placed normal to the face of the wall unless the plans show that the reinforcing strips must avoid an obstruction. The maximum select granular embankment lift thickness shall not exceed 200 mm (8 inch) loose and shall closely follow panel erection. The Contractor shall decrease the select granular embankment lift thickness if necessary to obtain the specified density.

At the end of each day's operations, the Contractor shall shape the last level of embankment to rapidly direct rain water runoff away from the wall face. The contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Compaction of the select granular embankment material shall be in accordance with all pertinent requirements of Item 203.12. The moisture content of the select granular embankment material prior to and during compaction shall be uniformly distributed throughout each layer and shall conform to requirements of Item 203.11.

Embankment compaction shall be accomplished without disturbance or distortion of reinforcing strips and panels. Compaction within 1 000 mm (3 feet) of the backside of the wall panels shall be achieved by at least 3 passes of a light mechanical tamper. The select granular embankment placed within 1 000 mm (3 feet) of the backside of the wall panels does not have to satisfy density test
requirements, but must be placed in 150 mm (6 inch) to 200 mm (8 inch) thick lifts which are uniformly compacted per above.

4.0 **Inspection**

The Reinforced Earth Company shall provide a company representative who shall monitor the precast operation sufficiently to ensure that the operation complies with all the above requirements and the representative shall submit documentation to this effect to the Engineer.

Documentation shall be furnished by the company representative for any panel which he deems acceptable but which is subject to rejection under 2.1.8.

The Reinforced Earth Company shall provide sufficient on-site technical assistance by a company representative to ensure that the Contractor and the Engineer fully understand the proper construction procedures and operations of the Reinforced Earth Wall System.

The Contractor shall provide a soils consultant who shall, through the Engineer, be responsible for inspection and field testing to confirm that the select granular embankment material requirements, placement and compaction requirements are in compliance with these special provisions.

The soils consultant shall provide the Engineer with two copies of all inspection reports which contain the testing results, all pertinent measurements and the soils consultant’s conclusions.

The soils consultant’s field representative shall be a Registered Professional Engineer or work under the direction of an Ohio Registered Professional Engineer. The final inspection reports shall be signed by an Ohio Registered Professional Engineer.

The Engineer will monitor erection of the structure and placement of select granular embankment material with technical assistance from the soils consultant and The Reinforced Earth Company. Any work not satisfying the requirements of these special provisions are subject to rejection by the Engineer.

5.0 **Coping and Traffic Barrier**

A cast-in-place coping and traffic barrier shall be provided on top of the wall as shown in the plans. Concrete shall be Class C and shall conform to CMS Item 511. Reinforcing steel shall be epoxy coated and shall conform to CMS Item 509.
6.0 Pile Sleeves

When piles are required, sleeves shall be used to form a void in the select granular embankment so that the proposed piles can be installed after the wall construction is completed. The sleeves shall be made of a material that does not promote corrosion within the select granular embankment and the sleeve material shall be satisfactory to the Reinforced Earth Company. Consider using segments of plastic pipe strong enough to maintain the required void. A bentonite slurry shall be placed in the void located between the pile and the sleeve. The slurry shall consist of the following materials with volume ratios of: one part cement, one part bentonite, and ten parts water. The cost of the above described work shall be considered incidental to Item Special "Reinforced Earth Walls."

7.0 Design Requirements for MSE Panel Walls

The design of the Reinforced Earth Wall shall be in strict conformance with the 1996 edition of ‘AASHTO Standard Specifications for Highway Bridges’, including the 1997 and 1998 Interims, except as modified in this document, the latest edition of the Ohio Bridge Design Manual, except as modified in this document, and the design requirements listed below:

1. The design shall meet all plan requirements. The recommendations of the wall system suppliers shall not override the minimum performance requirements shown herein. Other systems offered by the approved supplier shall not be submitted in lieu of the system which is called for in the plans.

2. Where walls or wall sections intersect with an included angle of 130 degrees or less, a vertical corner element separate from the standard panel face shall abut and interact with the opposing standard panels. The corner element shall have steel reinforcing strips connected specifically to that panel and shall be designed to preclude lateral spread of the intersecting panels.

3. One hundred percent of the steel reinforcing strips which are designed and placed in the reinforced earth volume shall extend to and be connected to the facing element through the use of tie strips or another acceptable method. No field cutting of steel reinforcing strips to avoid obstacles, such as abutment piles, shall be allowed. Also, steel reinforcing strips shall not be bent around such obstacles.

4. Under service loads, the minimum factor of safety at the connection between the face panel and the steel reinforcing strips shall be 1.5. The minimum factor of safety against reinforcement pullout shall be 1.5 at 13 mm (½ inch) deformation. The maximum allowable reinforcement tension shall not exceed two-thirds of the connection strength determined at 13 mm (½ inch) deformation.
5. The coefficient of lateral earth pressure $k_a$ and the application of the lateral forces to the reinforced soil mass for external stability analysis shall be computed using the Coulomb method, but assuming no wall friction.

6. All walls shall be designed with a coping or traffic barrier as specified in the plans.

7. Soil parameters for use in design are as follows:

<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>Compacted Select Granular Embankment Material with less than 7% P200 Material</td>
<td>18.9 kN/m³ (120 lbs/ft³)</td>
<td>34º</td>
<td>0</td>
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<td>Retained Soil</td>
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<td>18.9 kN/m³ (120 lbs/ft³)</td>
<td>30º</td>
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<tr>
<td>Foundation Soil</td>
<td>Variable - ranging from existing uncontrolled fill to natural silty sand</td>
<td>18.9 kN/m³ (120 lbs/ft³)</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

Refer to ODOT Bridge Design Manual 303.4.1.1

8. The allowable reinforcement tension of steel (inextensible) reinforcement elements for structural design and connection (pullout) design shall be based on the thickness of the elements at the end of the structure’s design life. In essence, the minimum thickness of the reinforcement elements shall be that thickness which will provide for the structural requirement plus the sacrificed thickness at the end of the design life.

9. The design life of the reinforced earth retaining walls shall be 100 years.

10. The minimum depth of embedment, measured from the finished ground line to the top of leveling pad shall be at least 1065 mm (3.5 feet) as shown on plan.

11. The internal stability, including definition of failure plane and lateral earth pressure coefficient, for Reinforced Earth walls shall be computed according to AASHTO Section 5.8.4.1, including the 1998 interims which permit the use of the Coherent Gravity Method for design.
12. The maximum thickness of the concrete leveling pad shall be 150 mm (6 inch).

13. The connections of the reinforcing strips to the panels shall be in two elevations for standard panels and the connections shall be no more than 760 mm (2.5 feet) apart vertically.

14. The wall height for design purposes shall be measured from the top of the leveling pad to the top of the coping. When the wall is retaining a sloping surcharge then the wall height shall be defined as the equivalent design height \( h \) as shown in AASHTO Figure 5.8.2B. The minimum reinforcing strip length shall be 70 percent of the wall height, as appropriately defined for either a level or sloping backfill, but no less than 2440 mm (8 feet).

15. The minimum thickness of the precast reinforced concrete face panels shall be 140 mm (5 ½ inch).

16. The yield stress for the metallic soil reinforcement shall not exceed 450 MPa (65 ksi).

17. The wall system, regardless of the size of panels, shall accommodate up to one percent differential settlement in the longitudinal direction.

18. The vertical stress at each reinforcement level shall be computed by considering local equilibrium of all the forces acting above the level under investigation. The vertical stress (bearing pressure) at each reinforcement level may be computed using the Meyerhof method in the same manner as the bearing pressure computed for the base of the wall.

19. Design shall include a dynamic horizontal thrust using an acceleration, \( A \), of 0.06 to simulate earthquake loading in Cincinnati, Ohio.

8.0 Method of Measurement

The Mechanically Stabilized Earth Wall quantity to be paid for shall be the actual number of square meter (feet) of facial area of approved Reinforced Earth wall panels complete in place. The total quantity is the completed and accepted front face wall area within the limits of the beginning and ending wall stations and from the top of the wall leveling pad to the top of the coping or barrier.

Item 503 Unclassified Excavation is defined in section 3.1.
8.1 **Basis of Payment**

Item Special "Reinforced Earth Walls" will be paid for at the contract unit price per square meter (foot). This work shall include the fabrication, furnishing, and erection of the Reinforced Earth walls, including the reinforced concrete face panels, reinforcing strips, fasteners, joint material, concrete leveling pad, concrete coping, reinforcing steel, filter fabric, and all incidentals, labor, and equipment necessary to complete this item.

Item 203 "Select Granular Embankment, as per plan" will be paid for at the contract unit price per cubic meter (yard). Payment for the soils consultant’s work shall be included with this item.

Item 203 "Embankment, as per plan" will be paid for at the contract unit price per cubic meter (yard).

Item 304 "Aggregate Base" will be paid for at the contract unit price per cubic meter (yard).

Item 503 "Unclassified Excavation, as per plan" will be paid for at the contract unit price per cubic meter (yard).

Item 518 "150 mm (6 inch) Perforated Corrugated Plastic Pipe" will be paid for at the contract unit price per linear meter (foot).

Item 518 "150 mm (6 inch) Non-perforated Corrugated Plastic Pipe, including specials" will be paid for at the contract unit price per linear meter (foot).

Item Special "Sealing of Concrete Surfaces, Epoxy-Urethane" will be paid for at the contract unit price per square meter (yard).
APPENDIX -METRIC SEPTEMBER 1998

APP-5 VSL RETAINED EARTH WALLS The following un-numbered note should be part of any project allowing the use of the VSL retained earth wall system. The designer should revise this note to meet project conditions and forward the revised note for inclusion into a project as Special Provisions.

VSL RETAINED EARTH WALLS

1.0 Description

This work consists of the design and construction of VSL Retained Earth walls in accordance with this specification. See Section 7.0 for VSL Retained Earth Wall Design requirements. The design submittal shall include detailed design calculations and a set of drawings describing the proposed retaining wall. The contractor submittal will include a site plan, an elevation drawing of the full length of the retaining wall and representative cross-sections at each design change. Drainage details are to be included as well as detailed construction specifications.

2.0 Materials

The Contractor shall make arrangements to acquire the reinforced concrete face panels, galvanized reinforcing mesh, attachment devices, joint materials, alignment pins and all necessary incidentals from the VSL Corporation, P.O. Box 866, 8006 Haute Court, Springfield, Virginia 22150 (Telephone: 703-451-4300).

The Contractor shall provide certification of the above Retained Earth wall components as per CMS 101.061. Certification shall be provided for the concrete, reinforcing steel, coil embed, coil bolt, galvanized reinforcing mesh, attachment devices, joint materials and alignment pins which are components of the Retained Earth wall. (Refer to the section of the proposal concerning steel produced in the United States.)

2.1 Reinforced Concrete Face Panels

Material used in concrete for face panels shall conform to the requirements of CMS Item 499.02 except as stated in this document. Cement shall be Type I, II, or III and shall conform to the requirements of AASHTO M 85-96. Concrete shall have a compressive strength at 28 days in accordance with Section 2.1.7 Compressive Strength. Concrete shall contain 6 +2 percent air. Air entraining admixture shall conform to AASHTO M 154-89. Retarding agents, accelerating agents or any additive containing chloride shall not be used without approval from the VSL Engineer.

Reinforcing steel and lifting devices shall be set in place to the dimensions and tolerances shown on the plans prior to casting.
2.1.1. **Testing and Inspection**

Acceptability of the concrete for the precast panels will be determined on the basis of compression tests, certifications and visual inspection. The concrete strength requirements for the precast panels shall be considered attained regardless of curing age when compression test results indicate strengths conform to 28-day strength specifications.

The Supplier shall furnish facilities and perform all necessary sampling and testing in an expeditious and satisfactory manner. Panels utilizing Type I or II cement shall be considered acceptable for placement in the wall when 7-day initial strengths exceed 85 percent of 28-day requirements. Panels utilizing Type III cement shall be considered acceptable for placement in the wall prior to 28 days only when compressive strength test results indicate that the strength exceeds the 28 day specification.

2.1.2. **Casting**

The panels shall be cast on a flat area with the front face down. Coil loop inserts, reinforcing steel, PVC pipe, and lifting devices shall be set in place to the dimensions and tolerance shown on the drawings prior to casting. The PVC pipe shall be placed in such a manner as to ensure that it is straight; not bent or bowed. Coil loop inserts shall be set on the rear face. The concrete in each unit shall be placed without interruption and shall be consolidated by the use of a concrete vibrator, supplemented by such hand-tamping as may be necessary to force the concrete into the corners of the forms and prevent the formation of stone pockets, air bubbles or cleavage planes. Clear form oil of the same manufacturer shall be used throughout the casting operation as approved by the VSL Engineer.

Special care shall be given to the coil loop inserts: All coil loop inserts must be attached to the alignment templates using the bolts provided with the forms. The vertical and horizontal alignment of the coil loop inserts shall be within ± 3 mm (1/8 inch) of the plan dimension. The holes inside the coil loop inserts must be 60 mm (2 3/8 inch) deep in the finished panel and must be free of all concrete and debris, loose or otherwise. No concrete or other debris shall be on the interior surfaces of the coils of the coil loop inserts in the finished panels. Immediately after the alignment template is removed, duct tape or equal approved by the VSL Engineer shall be placed over the coil loop insert holes in order to prevent debris from entering the holes. This duct tape shall not be removed except by the crew that is assembling the wall. Care shall be taken to ensure that the duct tape is not removed during shipping.
2.1.3. Curing

The curing method will be dependent on the concrete precaster’s capabilities and shall be as per instructions by the VSL Corporation. The reinforced concrete panels shall be cured for a sufficient length of time so that the concrete will develop the specified compressive strength. Any production lot which does not conform to the strength requirements of Section 2.1.7-Compressive Strength, shall be rejected.

2.1.4. Removal of Forms

The forms shall remain in place until they can be removed without damage to the panel.

2.1.5. Concrete Finish and Tolerances

The front face of the reinforced concrete panels shall have an architectural surface finish treatment as shown on the plans. The backside of the reinforced concrete panels shall have an unformed surface finish and shall be rough screeded to eliminate open pockets of aggregate and surface distortions in excess of 6 mm (1/4 inch). All panels, barriers and coping shall be sealed with an epoxy sealer in conformance with Special “Sealing of Concrete Surfaces, Epoxy”, unless special sealers have been called for on the plans.

2.1.6. Tolerances

All panels shall be manufactured within the following tolerances:

a. Panel Dimensions

All dimensions within 5 mm (0.2 inch).

b. Panel Squareness

Squareness as determined by the difference between the two diagonals shall not exceed 13 mm (1/2 inch).

c. Panel Surface Finish

Surface defects on smooth formed surfaces measured on a length of 1500 mm (5 feet) shall not exceed 3 mm (0.12 inch). Surface defects on textured surfaces measured on a length of 1500 mm (5 feet) shall not exceed 8 mm (0.312 inch).

2.1.7. Compressive Strength
Acceptance of the concrete panels with respect to compressive strength will be determined on the basis of production lots. A production lot is defined as a group of panels that will be represented by a single compressive strength sample and will consist of either 40 panels or a single day’s production, whichever is less.

During the production of the concrete panels, the manufacturer will randomly sample the concrete in accordance with AASHTO T 141-93. A single compressive strength sample, consisting of a minimum of four cylinders, will be randomly selected for every production lot.

Cylinders for compressive strength tests shall be 150 mm (6 inch) by 300 mm (12 inch) specimens prepared in accordance with AASHTO T 23-93. For every compressive strength sample, a minimum of two cylinders will be cured in the same manner as the panels and tested at approximately 7 days. The average compressive strength of these cylinders, when tested in accordance with AASHTO T 22-92, will provide a test result which will determine the initial strength of the concrete. In addition, two cylinders shall be cured in accordance with AASHTO T 23-93 and tested at 28 days. The average compressive strength of these two cylinders, when tested in accordance with AASHTO T 22-92, will provide a compressive strength test result which will determine the compressive strength of the production lot.

If the initial strength test results indicate a compressive strength in excess of 27.5 MPa (4000 psi), then these test results will be utilized as the compressive strength test result for the production lot and the requirement for testing at 28 days will be waived for that particular production lot.

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 27.5 MPa (4000 psi). If the compressive strength test result is less than 27.5 MPa (4000 psi), the acceptance of the production lot will be based on its meeting the following acceptance criteria in its entirety:

a. Ninety percent of the compressive strength test results for the overall production shall exceed 27.5 MPa (4000 psi).

b. The average of any six consecutive compressive strength test results shall exceed 27.5 MPa (4000 psi).

c. No individual compressive strength test result shall fall below 24.8 MPa (3600 psi).

In the event that a production lot fails to meet the specified compressive strength requirements, the production lot shall be rejected. Such rejection shall prevail unless the Manufacturer, at his own expense, obtains and submits evidence acceptable to the Engineer that the strength and quality of the concrete placed within the panels of the production lot is acceptable. If such evidence consists of
tests made on cores taken from the panels within the production lot, the cores shall be obtained and tested in accordance with the specifications of AASHTO T 24-93.

2.1.8. Rejection

Panels shall be subject to rejection because of failure to meet any of the requirements specified above. In addition, any or all of the following defects may be sufficient cause for rejection:

a. Defects that indicate imperfect molding.
b. Defects indicating honeycombed or open textured concrete.
c. Defects in the physical characteristics of the concrete, such as broken or chipped concrete.
d. Stained form face, due to excess form oil or other contaminations.
e. Signs of aggregate segregation.
f. Broken or cracked corners.
g. Lifting inserts not usable.
h. Exposed reinforcing steel.
i. Cracks at the PVC pipe or pin.
j. Insufficient concrete compressive strength.
k. Panel thickness in excess of 5 mm (3/16 inch) plus or minus from that shown on the plans.

The Engineer will decide if an attempt may be made to repair a defective panel. The Contractor or supplier shall make the repairs at his own expense. If the repairs are made to the Engineer’s satisfaction, the panel will be acceptable.

2.1.9. Marking

The date of manufacture, the production lot number and the piece-mark shall be clearly scribed on the back surface of each panel.
2.1.10. Handling, Storage and Shipping

All panels shall be handled, stored, and shipped in such a manner as to avoid chipping, cracking, fracturing, and excessive bending stresses. Panels shall be supported on firm blocking while in storage.

2.1.11. Reinforcing Steel

All reinforcing steel shall be Grade 420 (Grade 60) epoxy coated and shall satisfy all applicable requirements of CMS 509, reinforcing steel.

2.2 Reinforcing Mesh

The reinforcing mesh shall be shop fabricated of cold drawn steel wire conforming to the requirements of ASTM A82 and shall be welded into the finished mesh fabric in accordance with ASTM A 185. The reinforcing mesh shall be galvanized in conformance with ASTM A123.

2.3 Attachment Devices

2.3.1. Loop Embed

Loop embeds shall be fabricated from cold drawn steel rod conforming to ASTM-510M or ASTM-82. Loop embeds shall be welded in accordance with ASTM A-185. Loop embeds shall be galvanized in accordance with ASTM A123.

2.3.2. Connector Bar

Connector Bars shall be fabricated of cold drawn steel wire conforming to the requirements of ASTM A82 and shall be galvanized in accordance with ASTM A123.

2.4 Joint Materials

2.4.1. Inclined, Vertical and Horizontal Joints

The material to be attached to the rear side of the facing panels covering the inclined and horizontal joints between facing panels shall be Chicopee Polypropylene Erosion Fabric as manufactured by Chicopee Mills or equal approved by the Engineer. Adhesive used to attach the fabric material to the facing panel shall be Pliobond 5001 as manufactured by the Goodyear Rubber Company or equal approved by the VSL Engineer.
2.4.2. **Bearing Pads**

The material to be used in the horizontal joints between facing panels shall be high density polyethylene panel pads, 20 mm (13/16 inch) x 70 mm (2 3/4 inch) x 290 mm (11 7/16 inch), 2 per panel or equivalent as approved by the Engineer.

2.5 **Alignment Pins**

The pins used to align the face panels during construction shall be round, smooth number 5 bars made of mild steel or smooth 19 mm (3/4 inch) diameter PVC rod.

2.6 **Select Granular Embankment Material**

The select granular embankment material used in the reinforced structure volume shall be Item 304 Aggregate Base or Item 703.11 Granular Material, Type 2; deviations from these are as follows:

1) No slag shall be allowed.

2) The total percent passing the 0.075 mm (number 200) sieve shall be 0 to 7 percent prior to the placement operation. The acceptance samples shall be taken from the stockpile. The Contractor shall transport and handle the material to minimize the segregation of the material prior to the placement.

The select granular embankment material shall also conform to the requirements listed below:

1) AASHTO T 289-91 The pH range of the material shall be between 5.0 and 10.0.

2) AASHTO T 288-91 The resistivity of the material shall be greater than 3,000 ohm-cm. If the resistivity is found to be greater than 5,000 ohm-cm, then AASHTO Test T 290-95 and T 291-94 may be waived.

3) AASHTO T 291-94 The chloride levels of the material shall be less than 100 ppm.

4) AASHTO T 290-95 The sulfate levels of the material shall be less than 200 ppm.

The Contractor or supplier, as his agent, shall furnish the Engineer a Certificate of Compliance from an independent Testing Laboratory, certifying that the above materials comply with the applicable contract specifications. A copy of all test results performed by the contractor or his supplier necessary to assure contract compliance shall also be furnished to the Engineer.
Acceptance will be based on a letter of certification that the material to be utilized is within the parameters of the applicable contract notes and specifications. This letter shall be submitted along with the accompanying test reports to the Project Engineer. The Engineer shall provide written acceptance of material upon review of accompanying test reports, and visual inspection of the material by the Engineer.

2.7 Leveling Pad Concrete

The leveling pad concrete shall be a maximum 150 mm (6 inch) thick, having a strength that is not less than 17.2 MPa (2500 psi) and shall have sufficient strength to adequately support the panels at the bottom of the wall in a level position during installation. The leveling pad may be constructed of precast concrete. Precast concrete leveling pads can be advantageous when many short segment leveling pad steps are required.

2.8 Filter Fabric and Porous Backfill

As per Item 518, a 150 mm (6 inch) perforated plastic pipe shall be placed below the bottom row of reinforcing mesh. One drain pipe shall be located behind the leveling pad and the other shall be at the opposite end of reinforcing mesh. The pipe shall be continuous and be placed in the bottom portion of the reinforced fill. Positive gravity flow of the drainpipe shall be provided. Perforations in the pipe shall be smaller than the surrounding porous backfill. Filter fabric shall be installed at the required plan locations and shall satisfy the requirements of CMS Item 712.09, Type B.

3.0 Construction Requirements

3.1 Wall Excavation

Unclassified excavation shall be in accordance with ODOT Item 503 except that the limits of excavation shall be as shown in the plans.

3.2 Foundation Preparation

The foundation for support of the reinforced select granular embankment volume shall be graded level for a width equal to or exceeding the length of reinforcing mesh. Prior to wall construction, the foundation shall be level and rolled with a smooth wheel vibratory roller and the exposed soil compacted to at least 95% Standard Proctor density. Any foundation soils found to be unsuitable shall be removed and replaced, as directed by the Engineer.

The unreinforced concrete leveling pad shall be provided as shown on the plans. The pad shall be cured a minimum of 12 hours before placement of wall facing panels.
3.3 Wall Erection

Precast panels are handled by means of a lifting device connected to the lifting insert which is cast into the upper edge of the panels. Panels shall be set in successive horizontal lifts in the sequence shown on the plans as embankment fill placement proceeds. As embankment material is placed behind panels, the panels shall be maintained in vertical position by means of temporary wooden wedges placed in the panel joints on the external side of the wall. External bracing is required for the initial lift for vertical walls, vertical tolerance (plumbness) and horizontal alignment tolerance shall not exceed 20 mm (3/4 inch) when measured along a 3000 mm (10 feet) straight edge. The maximum allowable offset in any panel joint shall be 20 mm (3/4 inch). The overall vertical tolerance of the wall (plumbness from top to bottom) shall not exceed 13 mm (½ inch) per 3000 mm (10 feet) of wall height.

Reinforcing mesh shall be placed normal to the face of the wall, unless otherwise shown on the plans or directed by the Engineer. Prior to placement of the reinforcing mesh, the embankment backfill shall be compacted in accordance with Section 3.4.

3.4 Select Granular Embankment Material Placement

Select granular embankment material placement shall closely follow the erection of each lift of panels. At each reinforcing mesh level, the select granular embankment material should be roughly leveled before placing and attaching the mesh. The reinforcing mesh shall be placed normal to the face of the wall unless the plans show that the reinforcing mesh must avoid an obstruction. The maximum select granular embankment lift thickness shall not exceed 200 mm (8 inch) loose and shall closely follow panel erection. The Contractor shall decrease the select granular embankment lift thickness if necessary to obtain the specified density.

At the end of each day's operations, the Contractor shall shape the last level of embankment to rapidly direct runoff of rainwater away from the wall face. The contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Compaction of the select granular embankment material shall be in accordance with all pertinent requirements of Item 203.12. The moisture content of the select granular embankment material prior to and during compaction shall be uniformly distributed throughout each layer and shall conform to requirements of Item 203.11.

Embarkment compaction shall be accomplished without disturbance or distortion of reinforcing mesh or face panels. Compaction of the reinforced volume of fill shall not be accomplished by any type of equipment employing a foot, which in the opinion of the VSL Engineer, could penetrate the embankment material and damage the reinforcing mesh. Compaction within 1000 mm (3 feet) of the backside of the wall panels shall be achieved by at least 3 passes of a light mechanical tamper.
The select granular embankment material placed within 1000 mm (3 feet) of the backside of the wall panels does not have to satisfy density test requirements, but must be placed in 150 mm (6 inch) to 200 mm (8 inch) thick lifts which are compacted per above.

4.0 Inspection

The VSL Corporation shall provide a company representative who shall monitor the precast operation sufficiently to ensure that the operation complies with all the above requirements and the representative shall submit documentation to this effect to the Engineer.

Documentation shall be furnished by the company representative for any panel which he deems acceptable but which is subject to rejection under 2.1.8.

The VSL Corporation shall provide sufficient on-site technical assistance by a company representative to ensure that the Contractor and the Engineer fully understand the proper construction procedures and operations of the VSL Retained Earth Wall System.

The Contractor shall provide a soils consultant who shall, through the Engineer, be responsible for inspection and field testing to confirm that the select granular embankment material requirements and the placement and compaction requirements are in compliance with these special provisions and specification.

The soils consultant shall provide the Engineer with two copies of all inspection reports which contain the testing results, all pertinent measurements and the soils consultant’s conclusions.

The soils consultant’s field representative shall be an Ohio Registered Professional Engineer or work under the direction of an Ohio Registered Professional Engineer. The final inspection report shall be signed by an Ohio Registered Professional Engineer.

The Engineer will monitor erection of the structure and placement of the select granular embankment material with technical assistance from the soils consultant and the VSL Corporation. Any work not satisfying the requirements of these special provisions is subject to rejection by the Engineer.

5.0 Coping and Traffic Barrier

A cast-in-place coping and traffic barrier shall be provided on top of the wall as shown in the plans. Concrete shall be Class C and shall conform to CMS Item 511. Reinforcing steel shall be epoxy coated and shall conform to CMS Item 509. For payment for the concrete and the reinforcing steel, see 8.1 “Basis of Payment”.
6.0 Pile Sleeves

When piles are required, sleeves shall be used to form a void in the select granular embankment so that the proposed piles can be installed after the wall construction is completed. The sleeves shall be made of a material that does not promote corrosion within the select granular embankment and the sleeve material shall be satisfactory to the wall company. Consider using segments of plastic pipe strong enough to maintain the required void. A bentonite slurry shall be placed in the void located between the pile and the sleeve. The slurry shall consist of the following materials with volume ratios of: one part cement, one part bentonite, and ten parts water. The cost of the above described work shall be considered incidental to Item Special "Retained Earth Walls."

7.0 Design Requirements for MSE Panel Walls

The design of the VSL Retained Earth Wall shall be in strict conformance with the 1996 edition of ‘AASHTO Standard Specifications for Highway Bridges’, including the 1997 and 1998 Interims, except as modified in this document, the latest edition of the Ohio Bridge Design Manual, except as modified in this document, and the design requirements listed below:

1. The design shall meet all plan requirements. The recommendations of the wall system suppliers shall not override the minimum performance requirements shown herein. Other systems offered by the approved supplier shall not be submitted in lieu of the system which is called for in the plans.

2. Where walls or wall sections intersect with an included angle of 130 degrees or less, a vertical corner element separate from the standard panel face shall abut and interact with the opposing standard panels. The corner element shall have steel reinforcing strips connected specifically to that panel and shall be designed to preclude lateral spread of the intersecting panels.

3. One hundred percent of the steel reinforcing strips which are designed and placed in the reinforced earth volume shall extend to and be connected to the facing element through the use of tie strips or another acceptable method. No field cutting of steel reinforcing strips to avoid obstacles, such as abutment piles, shall be allowed. Also, steel reinforcing strips shall not be bent around such obstacles.

4. Under service loads, the minimum factor of safety at the connection between the face panel and the steel reinforcing strips shall be 1.5. The minimum factor of safety against reinforcement pullout shall be 1.5 at 13 mm (½ inch) deformation. The maximum allowable reinforcement tension shall not exceed two-thirds of the connection strength determined at 13 mm (½ inch) deformation.
5. The coefficient of lateral earth pressure $k_a$ and the application of the lateral forces to the reinforced soil mass for external stability analysis shall be computed using the Coulomb method, but assuming no wall friction.

6. All walls shall be designed with a coping or traffic barrier as specified in the plans.

7. Soil parameters for use in design are as follows:

<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>Compacted Select Granular Embankment Material with less than 7% P200 Material</td>
<td>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>18.9 kN/m³</td>
<td>30°</td>
<td>0</td>
</tr>
<tr>
<td>Foundation Soil</td>
<td>Variable - ranging from existing uncontrolled fill to natural silty sand</td>
<td>18.9 kN/m³</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

** Refer to ODOT Bridge Design Manual 303.4.1.1

8. The allowable reinforcement tension of steel (inextensible) reinforcement elements for structural design and connection (pullout) design shall be based on the thickness of the elements at the end of the structure’s design life. In essence, the minimum thickness of the reinforcement elements shall be that thickness which will provide for the structural requirement plus the sacrificed thickness at the end of the design life.

9. The design life of the reinforced earth retaining walls shall be 100 years.

10. The minimum depth of embedment, measured from the finished ground line to the top of leveling pad shall be at least 1065 mm (3.5 feet) as shown on plan.

11. The internal stability, including definition of failure plane and lateral earth pressure coefficient, for Reinforced Earth walls shall be computed according to AASHTO Section 5.8.4.1, including the 1998 interims which permit the use of the Coherent Gravity Method for design.
12. The maximum thickness of the concrete leveling pad shall be 150 mm (6 inch).

13. The connections of the reinforcing strips to the panels shall be in two elevations for standard panels and the connections shall be no more than 760 mm (2.5 feet) apart vertically.

14. The wall height for design purposes shall be measured from the top of the leveling pad to the top of the coping. When the wall is retaining a sloping surcharge then the wall height shall be defined as the equivalent design height \( h \) as shown in AASHTO Figure 5.8.2B. The minimum reinforcing strip length shall be 70 percent of the wall height, as appropriately defined for either a level or sloping backfill, but no less than 2440 mm (8 feet).

15. The minimum thickness of the precast reinforced concrete face panels shall be 140 mm (5 ½ inch).

16. The yield stress for the metallic soil reinforcement shall not exceed 450 MPa (65 ksi).

17. The wall system, regardless of the size of panels, shall accommodate up to one percent differential settlement in the longitudinal direction.

18. The vertical stress at each reinforcement level shall be computed by considering local equilibrium of all the forces acting above the level under investigation. The vertical stress (bearing pressure) at each reinforcement level may be computed using the Meyerhof method in the same manner as the bearing pressure computed for the base of the wall.

19. Design shall include a dynamic horizontal thrust using an acceleration, \( A \), of 0.06 to simulate earthquake loading in Cincinnati, Ohio.

8.0 Method of Measurement

The Mechanically Stabilized Earth Wall quantity to be paid for shall be the actual number of square meter of facial area of approved Retained Earth wall panels complete in place. The total quantity is the completed and accepted front face wall area within the limits of the beginning and ending wall stations and from the top of the wall leveling pad to the top of the coping or barrier.

503 Unclassified Excavation is defined in Section 3.1.
8.1 Basis of Payment

Item Special "Retained Earth Walls" will be paid for at the contract unit price per square meter (foot). This work shall include the fabrication, furnishing, and erection of the Retained Earth walls, including the reinforced concrete face panels, reinforcing mesh, fasteners, joint material, concrete leveling pad, concrete coping, reinforcing steel, filter fabric, and all incidentals, labor, and equipment necessary to complete this item.

Item 203 "Select Granular Embankment, as per plan" will be paid for at the contract unit price per cubic meter (yard). Payment for the soils consultant’s work shall be included with this item.

Item 203 "Embankment, as per plan" will be paid for at the contract unit price per cubic meter (yard).

Item 304 "Aggregate Base" will be paid for at the contract unit price per cubic meter (yard).

Item 503 "Unclassified Excavation, as per plan" will be paid for at the contract unit price per cubic meter (yard).

Item 518 "150 mm (6 inch) Perforated Corrugated Plastic Pipe" will be paid for at the contract unit price per linear meter (foot).

Item 518 "150 mm (6 inch) Non-perforated Corrugated Plastic Pipe, including specials" will be paid for at the contract unit price per linear meter (foot).

Item Special "Sealing of Concrete Surfaces, Epoxy-Urethane" will be paid for at the contract unit price per square meter (yard).
APPENDIX - METRIC SEPTEMBER 1998

APP-6 3 COAT SHOP PAINT SYSTEM IZEU Un-numbered proposal note for having structural steel superstructure members painted in the shop with 3 coat IZEU. The 863 item shall be AS PER PLAN and have a note “Shop painting per CMS section 863.29 shall not be included with the price bid for structural steel but shall be included with Item Special: Shop painting and Field Touch-Up of Structural Steel, Lump Sum.

ITEM SPECIAL - SHOP PAINTING AND FIELD TOUCH UP OF STRUCTURAL STEEL

DESCRIPTION

June 27, 1997

This specification covers cleaning and application of a 3 coat shop applied paint system for Item 863, structural steel, and includes requirements for field cleaning and coating of surfaces only primed 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.

This specification shall also include galvanizing, 711.02, of all nuts, washers, bolts, anchor bolts, and any other structural steel items requiring galvanizing and part of item 863.

MATERIAL

A. A three coat paint system consisting of an:

   Inorganic Zinc Prime Coat meeting the requirements of C & MS 708.17

   Epoxy Intermediate Coat meeting the requirements of Supplemental Specification 910 entitled "OZEU Structural Steel Paint"

   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.
PRE-PAINT CONFERENCE

A pre-paint conference shall be held separately from the pre-construction meeting. Attendees to this meeting shall include the Contractor, field painting sub-contractor, structural steel erector, fabricator, quality control specialist, The Engineer, A representative from the Office of Structural Engineering, and any others required in the plan. The meeting shall take place before the steel is fabricated or painted.

QUALITY CONTROL SPECIALISTS

This person will not be a Foreman or member of the Contractor's or fabricator's production staff (i.e. he will not abrasive blast, paint, recover spent abrasives, etc.). He will not be involved in any other miscellaneous task (i.e. mixing paint, running errands, running or working on equipment, etc). No work shall begin before the Quality Control Specialist (QCS) qualifications have been validated by the Engineer. Documentation/verification shall be provided to the Engineer that the QCS has received formal training from one of the following: KTA Tator, S.G. Pinney, or Corrosion Control Consultants. The QCS shall be equipped with material safety data sheets, tools and equipment to provide quality control on all facets of the work and shall have a thorough understanding of the plans and specifications pertaining to this project. He shall be responsible for inspecting the equipment at the specified intervals, the abrasives, and the work, at all quality control points. He shall also be responsible for verifying that all work is done within the specified work limitation. He shall cooperate with the Inspector and compare and document quality control readings. He shall have the authority to stop work and the responsibility to inform the Contractor's or fabricator's foreman of nonconforming work.

QCSs will be required in the shop and in the field. Before fabrication the fabricator shall designate one individual for each shop as a Quality Control Specialist. At the preconstruction or pre-paint meeting, the Contractor shall also designate one individual on each project as a Quality Control Specialist (only one person per project will be necessary unless the Contractor is working at more than three (3) sites simultaneously). It will then be necessary to provide an additional Quality Control Specialist and a set of testing equipment as described in the equipment section for each additional three sites being painted simultaneously.

Quality Control Points (QCP)

QCPs are points in time when one phase of the work is complete and ready for inspection by the Contractor or fabricator and the ODOT shop fabrication inspector or the Engineer prior to continuing with the next operational step. At these points: The Contractor or 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 QCP is past or failure of the final product
before final acceptance, shall not in any way prevent rejection or obligate the State of Ohio to final acceptance.

<table>
<thead>
<tr>
<th>Quality Control Points (QCP)</th>
<th>(PURPOSE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.) Washing</td>
<td>Remove water soluble oil, grease, salt, dirt, etc.</td>
</tr>
<tr>
<td>2.) Solvent Cleaning</td>
<td>Remove asphaltic cement, oil, grease, salt, dirt, etc., not removed during washing</td>
</tr>
<tr>
<td>3.) Grinding Edges</td>
<td>Grind edges required.</td>
</tr>
<tr>
<td>4.) Abrasive Blasting</td>
<td>Blast surface to receive paint</td>
</tr>
<tr>
<td>5.) Prime Coat Application</td>
<td>Check surface cleanliness, apply prime coat, check coating thickness</td>
</tr>
<tr>
<td>6.) Intermediate Coat Application</td>
<td>Check surface cleanliness, apply intermediate coat, check coating thickness</td>
</tr>
<tr>
<td>7.) Finish Coat Application</td>
<td>Check surface cleanliness, apply finish coat, check coating thickness</td>
</tr>
<tr>
<td>8.) Visual Inspection</td>
<td>Visually inspect paint before shipment of steel and check of total system thickness.</td>
</tr>
<tr>
<td>9.) 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>10.) Final Review</td>
<td>Wash structure as per QCP#1. Visually inspect system for acceptance and check total system thickness.</td>
</tr>
</tbody>
</table>

**SURFACE PREPARATION**

This item shall consist of washing, solvent cleaning, and abrasive cleaning of structural steel members.

**Washing (QCP #1) Field only**

Prior to field abrasive blasting, all surfaces to be painted shall be washed with potable water having a nozzle pressure of at least 1,000 PSI and a delivery rate of not less than 4 gallons per minute.
(QCP #1) The contractor or fabricator, shall provide equipment specifications to verify the above. The equipment shall also be equipped with gauges to verify the pressure. The water shall contain tri-sodium phosphate detergent at a rate of 2 ounces (by weight) per gallon of technical grade, hydrated water (minimum) to remove water soluble oil, grease, salt and dirt. Before the surfaces dry, the bridge or structural steel member shall be rinsed to remove all remaining detergent. The nozzle shall be held at a maximum of twelve (12) inches from the surface being washed or rinsed. Surfaces shall not be considered as clean until clear rinse water runs off the structure. After the surface is rinsed and allowed to dry, it shall be checked for remaining visible dirt. Surfaces shall be rewashed and rinsed as necessary to remove all remaining dirt. The finish coat shall be applied within three (3) months of washing the structure or structural steel member.

Solvent Cleaning (QCP #2)

After washing, all traces of asphaltic cement, oil, grease, diesel fuel deposits, and other soluble contaminants which remain, shall be removed by solvent cleaning (QCP #2) (see SSPC-SP 1 Solvent Cleaning for recommended practices). Under no circumstances shall any abrasive blasting be done to areas with asphaltic cement, oil, grease, or diesel fuel deposits. All solvent cleaned areas shall be rewashed as previously noted.

Grinding Edges (QCP #3)

The edges of all steel shall be rounded in accordance with AWS D1.5 Section 3.2.9 before abrasive blasting.

Abrasive Blasting (QCP #4)

All steel to be painted shall be blast cleaned according to SSPC-SP10 (near-white blast) as shown in SSPC-Vis 1-89 (pictorial surface preparation standards for painting steel surfaces). Steel shall be maintained in a blast cleaned condition until it has received a prime coat of paint.

During Shop application and field touch up galvanized steel (including corrugated steel bridge flooring), adjacent concrete, existing painted surface and other surfaces not intended to be painted, shall be masked to prevent damage from abrasive blasting and painting operations.

The abrasive shall be a recyclable steel grit. After each use and prior to reuse, the steel grit shall be cleaned of paint chips, rust, mill scale and other foreign material by equipment specifically designed for such cleaning.

The surface profile shall be a minimum of 1 mil and a maximum of 3.5 mils. 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 degrees above the dew point shall be re-blasted when the
steel temperature is five (5) degrees above the dew point. Dew point is defined as the temperature at which moisture condenses on the steel surfaces.

All fins, tears, slivers, and burred or sharp edges present on any steel member after blasting shall be removed by grinding and then the area re-blasted.

All abrasives and residue shall be removed from surfaces to be painted with a vacuum system equipped with a brush-type cleaning tool. All steel blast cleaned shall be kept dust free and prime coated before the blasted surfaces have degraded from the prescribed standards, but in every case within 24 hours in the shop and 12 hrs in the field. Steel, not prime coated with in 24 hrs in the shop or 12 hrs in the field, will be re-blasted before prime coating. No dust or abrasives from adjacent work shall be left on the finish coat.

The QCS shall perform the following test (and the Inspector will verify) to ensure that the air is not contaminated:

blow air from the nozzle for thirty (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 verified by another test. This test shall be done at the start of each shift and at four (4) hour intervals.

Abrasive blasting and painting may take place simultaneously at any one location as long as abrasive blasting debris and/or dust created by the blowing operation does not come in contact with freshly painted surfaces.

The Contractor shall remove all blasting residues from the roadway, pedestrian walkways, gutters and other drainage facilities at the end of each day’s work. Care shall be taken to keep all blasting residues out of drains or catch basins. Nearby drains and catch basins shall be covered during blasting operations. Blasting residue shall not be permitted on surfaces which are being used by vehicles or pedestrians. The blasting residues shall be disposed of outside the highway right of way.

TESTING EQUIPMENT

Both the Contractor for the field application and the Fabricator for shop application, shall provide and assign to the Engineer the following testing equipment in good working order, for the duration of the project, one set of testing equipment for each quantity control specialist. These shall be separate sets from those Contractor or fabricator provide for Quality Control Specialist.

Each Quality Control Specialist shall have his own testing equipment. When no test equipment is available, no work shall be performed.
1. One (1) Spring micrometer and 1 roll of coarse and 3 (unless otherwise specified on plans) rolls of extra-coarse replica tape.

2. One (1) Positector 2000-6000, Quanix 2200, or Elcometer (A345FBI1) and the calibration plates as per the NBS calibration standards in accordance with ASTM D-1186.

3. One (1) Sling Psychrometer including Psychometric tables used to calculate relative humidity and dew point temperature.

4. Two (2) steel surface thermometers accurate within 2 degrees.

5. Flashlight 2-D cell

6. SSPC Visual Standard for Abrasive Blast Cleaned Steel  SSPC-Vis 1-89

7. One (1) Recorder Thermometer capable of recording the date, time, and temperature over a period of at least 12 hours.

HANDLING

All paint and thinner shall be delivered to the project site or fabricator’s shop in original, unopened containers with labels intact. Minor damage to containers is acceptable provided the container has not been punctured. Thinner containers shall be a maximum of five (5) gallons.

Paint shall be stored at the temperature recommended by the manufacturer to prevent paint deterioration.

Each container of paint and thinner shall 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.

All containers of paint and thinner shall remain unopened until required for use. The label information shall be legible and shall be 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 shall not be used. However, thixotropic materials which can be stirred to attain normal consistency may be used.

The oldest paint of each kind shall be used first. No paint shall be used which has surpassed its shelf life.
Paint may be considered as eligible for payment for material on hand as specified in 109.07. However, only paint which the Contractor can prove to the Engineer will be used during the construction season shall be eligible for payment. The Contractor shall provide the Engineer calculations indicating the total square feet of steel to be painted during the construction season. He shall also provide calculations showing the total number of gallons required. The Contractor shall be responsible to store the paint on the project in such manner to prevent theft and adverse temperatures. He shall provide thermometers capable of monitoring the maximum high and low temperatures within the storage facility. The Contractor is responsible for properly disposing of all unused paint and paint containers.

The Contractor shall furnish shipping invoices for all materials used on the project to the Engineer, prior to use.

**MIXING AND THINNING**

All ingredients in any container of paint shall be thoroughly mixed immediately before use and shall be agitated often enough during application to maintain a uniform composition; however, the primer shall be continuously mixed by an automated agitation system (Hand held mixers not allowed). Paint shall be carefully examined after mixing for uniformity and to verify that no unmixed pigment remains on the bottom of the container. The paint shall be mixed with a high shear mixer (such as a Jiffy Mixer). Paddle mixers or paint shakers are not allowed. Paint shall not be mixed or kept in suspension by means of an air stream bubbling under the paint surface.

All paint shall be strained after mixing. Strainers shall be of a type to remove only skins and undesirable matter, but not the pigment.

No thinner shall be added to the paint without the Engineer's or the shop fabrication Inspector's approval, and only if necessary for proper application as recommended by the manufacturer. When the use of thinner is permissible, thinner shall be added slowly to the paint during the mixing process. All thinning shall be done under supervision of the Engineer or the shop fabrication Inspector. In no case shall 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 shall be added to the paint.

Catalysts, curing agents, or hardeners which are in separate packages shall be added to the base paint only after the base paint has been thoroughly mixed. The proper volume of the catalyst shall then be slowly poured into the required volume of base with constant agitation. Liquid which has separated from the pigment shall not be poured off prior to the mixing. The mixture shall be used within the pot life specified by the manufacturer. Therefore only enough paint shall be catalyzed for prompt use. Most mixed, catalyzed paints cannot be stored, and unused portions of these shall be discarded at the end of each working day.
COATING APPLICATION

General

All new item 863 structural steel and other areas shown in the plans shall be painted unless otherwise noted in the plans.

Galvanized or metallized surfaces and surfaces in contact with expansion joint seals, shall be masked and receive no paint. Where metallized and painted surfaces are attached by welding the area shall be repaired with the paint system.

All areas where field welding are required shall be masked prior to shop coating and receive no paint.

The top of flanges shall receive the prime coat only of 0.5 to 1.5 mil thickness.

Areas to receive studs shall not be masked but paint shall be removed before studs are applied.

Coating of Bolted Faying Surfaces
Surfaces indicated below shall be treated according to Method A as described in this specification:
1. Faying surfaces of main beam or girder bolted field splices.
2. All internal contact surfaces of filler and splice plates.
3. Other surfaces indicated in the plans.

Bolted crossframes on straight beams or girders do not need to meet the requirements of Method A unless indicated otherwise in the plans.

Method A
The faying surfaces of bolted splices shall be coated with inorganic zinc primer in the shop. After erection is complete the final coatings of epoxy tie coat, epoxy intermediate coat and urethane protective coats shall be field applied as to overlap the shop coatings shown in figure one(1) with the field coats shown in figure two(2).
All bolted shop connections and bolted cross frames shall be removed and disassembled prior to the blasting and coating of the girders or beams. The parts shall 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 shall be shop applied.
All galvanized nuts, bolts, and washers shall be solvent cleaned after installation. The epoxy tiecoat, epoxy coat and the urethane protective coat shall then be applied.

Erection marks added by the fabricator to highlight or enhance the required steel stamped erection marks shall be made without damaging the paint system. Erection marks shall be applied only after the finish coat is cured and shall 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 shall be applied by spray.

The Contractor for field application and the Fabricator for shop application, shall supply the Engineer with the product data sheets before any coating is done. The product data sheets shall indicate the mixing and thinning directions, the recommended spray nozzles and pressures and the minimum drying time for shop applied coats.

These product data sheets shall be followed except when they conflict with these specifications, in which case the specifications shall govern.

If the surface is degraded or contaminated after surface preparation and before painting, the surface shall be restored before painting application. In order to prevent degradation or contamination of cleaned surface, the prime coat of paint shall 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 shall be scheduled so that dust or other contaminants do not fall on wet, newly painted surfaces. Surfaces not intended to be painted shall be suitably protected from the effects of cleaning and painting operations. Overspray shall be removed with a stiff bristle brush, wire screen, or a water wash with sufficient pressure to remove overspray without damaging the paint. The overspray must be removed before applying the next coat. All abrasives and residue shall be removed from painted surfaces before re-coating, with a vacuum system equipped with a brush type cleaning tool.

No visible abrasives from adjacent work shall be left on any coat. Abrasives shall be removed.

Spray application for the intermediate coat (epoxy) shall 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 overspray.

**Spray Application (General)**

All spray application of paint shall be in accordance with the following:
Primer ingredients shall be kept uniformly mixed in the spray equipment shall be kept clean so that dirt, dried paint and other foreign materials are not deposited in the paint film. Any solvent left in the equipment shall be completely removed before using.

Paint shall be applied in a uniform layer with overlapping at the edges of the spray pattern. The border of the spray pattern shall 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 shall 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. All bolts and rivet heads shall be sprayed from at least two (2) directions or brushed to insure coverage.

Each spray operator shall demonstrate to the Engineer his ability to apply the paint as specified. Any operator who does not demonstrate this ability shall not spray.

If mud cracking occurs, the affected area shall be cleaned to bare metal in accordance with surface preparation above and repainted.

All spray equipment used shall be suitable for use with the specified paint. Paint manufacturer’s equipment recommendations shall be consulted in the event of paint application problems.

If air spray is used, traps or separators shall 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 shall be done by the Contractor and verified by the Engineer to insure that the traps or separators are working properly. Blow air from the spray gun for thirty (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 shall be suspended until the problem is corrected and verified by another test. This test shall be done at the start of each shift and at four (4) hour intervals. This is not required for an airless sprayer.

**Application Approval**

The beginning of the application of each of the three different coats shall be subject to inspection and approval. The purpose of this inspection is to detect any defects which might result from the Contractor’s method of application. If any defects are discovered, the Contractor shall make all necessary adjustments to his method of application to eliminate these defects before proceeding with application.
Temperature

Paint shall not be applied when the temperature of the air, steel, or paint is below 50 degrees F. Paint shall not be applied when the steel surface temperature is expected to drop below 50 degrees F before the paint has cured for the minimum times specified below:

<table>
<thead>
<tr>
<th>Temperature</th>
<th>50 F</th>
<th>60 F</th>
<th>70 F</th>
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</thead>
<tbody>
<tr>
<td>Primer</td>
<td>4 hrs.</td>
<td>3 hrs.</td>
<td>2 hrs.</td>
</tr>
<tr>
<td>Intermediate</td>
<td>6 hrs.</td>
<td>5 hrs.</td>
<td>4 hrs.</td>
</tr>
<tr>
<td>Finish</td>
<td>8 hrs.</td>
<td>6 hrs.</td>
<td>4 hrs.</td>
</tr>
</tbody>
</table>

The above temperatures and times shall be monitored with the recording thermometer.

Moisture

Paint shall not be applied when the steel surface temperature is less than 5 degrees F above the dew point. Paint shall not be applied to wet or damp surfaces or on frosted or ice-coated surfaces. Paint shall not be applied when the relative humidity is greater than 85%. Paint shall not be applied during rain, fog or mist unless the above moisture criteria is met.

Continuity

Each coat of paint shall 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 shall be repainted and permitted to dry before the next coat of paint is applied.

Dry Film Thickness

Prime thickness, cumulative prime and intermediate thickness and cumulative prime, intermediate and finish thickness shall be determined by use of type 2 magnetic gage in accordance with the following:

Five (5) separate spot measurement spaced evenly over each 100 square feet of area to be measured. For field measurements these measurements shall be taken on flanges, webs, cross bracing, stiffeners, etc. Three (3) gage readings shall be made for each spot measurement of either the substrate or the paint. Move the probe a distance of one to three inches for each new gage reading. Discard any unusually high or low gage reading that cannot be repeated consistently. Take the average (mean) of the three gage readings as the spot measurement. The average of five
spot measurements for each such 100 square foot area shall not be less that the specified thickness. No single spot measurement in any 100 square foot area shall be less than 80% of the specified thickness. Any one of three readings which are average to produce each spot measurement, may under- run by a greater amount. The five spot measurements shall be made for each 100 square feet of area as follows:

1. For structures or batch of structural steel not exceeding 300 square feet in area, each 100 square foot area shall be measured.

2. For structures or batch of structural steel not exceeding 1,000 square feet in area, three 100 square foot areas shall be randomly selected and measured.

3. For structures or batch of structural steel exceeding 1,000 square feet in area, the first 1,000 square feet shall be measured as stated in section 2 and for each additional 1,000 square feet, or increment thereof, one 100 square foot area shall be randomly selected and measured.

4. If the dry film thickness for any 100 square foot area (sections 2 & 3 is not in compliance with the requirements of paragraph 1 of this section, then each 100 square foot area shall be measured.

5. Other size areas or number of spot measurements may be specified in the contract plans as appropriate for the size and shape of the structure to be measured.

Each coat of paint shall have the following mil thickness measured above the peaks:

<table>
<thead>
<tr>
<th></th>
<th>Min. Spec</th>
<th>Max. Spec</th>
<th>Min. Thickness Spot</th>
<th>Max Thickness Spot</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prime</td>
<td>3.0</td>
<td>5.0</td>
<td>2.4</td>
<td>7.5</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5.0</td>
<td>7.0</td>
<td>4.0</td>
<td>10.5</td>
</tr>
<tr>
<td>Sub Total</td>
<td>8.0</td>
<td>12.0</td>
<td>6.4</td>
<td>18.0</td>
</tr>
<tr>
<td>Finish</td>
<td>2.0</td>
<td>4.0</td>
<td>1.6</td>
<td>6.0</td>
</tr>
<tr>
<td>Total</td>
<td>10.0</td>
<td>16.0</td>
<td>8.0</td>
<td>24.0</td>
</tr>
</tbody>
</table>
Film thicknesses greater than the maximum specified thicknesses that do not exhibit defects (such as runs, sags, bubbles, mudcracking, etc.) and for which the Contractor has received a written statement from the coating manufacturer stating that this excessive thickness is not detrimental, may remain in place at the discretion of the Director.

For any spot or maximum average thickness over 24 mils, it will be necessary for the Contractor to prove to the Department the excess thickness will not be detrimental to the coating system. This shall be accomplished by providing the Director, 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 shall 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 Director a written statement from the paint manufacturer stating that this excessive thickness is not detrimental.

If the Director does not approve the excessive coating thicknesses or the Contractor elects not to provide the required written statement from the paint manufacturer and the certified test results when required, the Contractor, at his own expense, shall remove and replace the coating. The removal and replacement of the coating shall be done as specified in the section of this specification titled Repair Procedures.

Prime, Intermediate And Finish Coat Application (QCP #5, #6 & #7)

Each coat of paint shall be in a proper state of cure or dryness before the application of the succeeding coat. Paint shall be considered ready for recoating when an additional coat can be applied without the development of any detrimental film irregularities, such as lifting, wrinkling or loss of adhesion of the undercoat. The time interval between coating applications shall be in compliance with manufacture’s written instructions and no more than thirteen (13) days between the intermediate and finish coats. These maximum re-coat times include weather related days.

No additional time for weather delays will be allowed. Any coat which has cured more than the above allotted time without recoating shall be removed and the steel reblast to SSPC-SP10.

The completion date (month and year) of the finish coat and the letters IZEU/SHOP shall be stenciled on the steel in 4” letters with a black urethane paint. This date shall be applied at four locations near the end of each outside beam on the outside web visible from the road or as directed by the Engineer.
HANDLING AND SHIPPING

Extreme care shall be exercised in handling the steel in the shop, during shipping, during erection, and during subsequent construction of the bridge. Painted steel shall not be moved or handled until sufficient cure time has elapsed and approval has been obtained from the inspector. The steel shall be insulated from the binding chains by softeners approved by the Engineer. Hooks and slings used to hoist steel shall be padded. Diaphragms and similar pieces shall be spaced in such a way that no rubbing will occur during shipment that may damage the coatings. The steel shall be stored on pallets at the job site, or by other means approved by the Engineer, 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 shall 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.

REPAIR OF DAMAGED AREAS (QCP #9)

Damaged areas of paint and areas which do not comply with the requirements of this specification, shall have the paint removed and all defects corrected. The steel shall then be retextured to a near white condition to produce a profile of between 1 to 3 ½ mils. This profile shall be measured immediately prior to the application of the prime coat to insure that the profile is not destroyed during the feathering procedure.

The existing paint shall be feathered to expose a minimum of ½ inch of each coat.

During the reapplication of the paint, care shall be used to insure that each coat of paint is only applied within the following areas. The prime coat shall only be applied to the surface of the bare steel and the existing prime coat, which has been exposed by feathering. The prime coat shall not be applied to the adjacent intermediate coat. The intermediate coat shall 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 shall not extend beyond the areas which has been feathered or lightly sanded.

The first two coats shall be applied by brush. The finish coat shall be applied by either brush or spray.

It may be necessary to make several applications in order to achieve the proper thickness for each coat.

During the application of the prime coat, the paint should be continuously mixed.

All abrasive blasting and painting shall still be done in accordance with the specifications.
All repairs should be made in a manner to blend the patched area with the adjacent coating. The finished surface of the patched area shall have a smooth even profile with the adjacent surface.

The first repair area shall be used as a test section and no more repairs made until the methods are approved by the Engineer.

The Contractor or fabricator shall submit his method of correcting runs in writing to the Director for approval.

Damaged paint which will be inaccessible for coating after erection shall be repaired and re-coated prior to erection.

In order to minimize damage to the painted steel, concrete splatter and form leakage shall be washed from the surface of the steel shortly after the concrete is placed and before it is dry. If concrete dries it shall be removed and paint repaired.

Temporary attachments, supports for scaffolding and, finishing machine or forms shall not damage the coating system. (In particular, on the fascias where bracing is used, sufficient size support pads shall be 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 shall inspect the structure for damaged paint. (QCP #10). Damaged areas shall be repaired by repeating QCP #1 to #8. The contractor shall wash the structure as per QCP#1 after all work to the structure is completed.

SAFETY REQUIREMENTS AND PRECAUTIONS

The contractor shall 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) shall be provided at the preconstruction meeting for all paint, thinners and abrasives used on this project. No work shall start until the MSDS has been submitted.

The fabricator shall also provide MSDS for all abrasives to be used on this project to the shop inspector. No work shall start until MSDS have been submitted.
SCAFFOLDING

Rubber rollers, or other protective devices meeting the approval of the Engineer, shall be used on scaffold fastenings. Metal rollers or clamps and other types of fastenings which will mar or damage coated surfaces shall not be used.

INSPECTION ACCESS FOR FIELD TOUCH UP

In addition to the requirement of 105.11, the contractor shall furnish, erect, and move scaffolding and other appropriate equipment, to permit the Inspector the opportunity to inspect closely observe, all affected surfaces. This opportunity shall 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 shall 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 shall be complied with:

When scaffolding is suspended forty three inches or more below the surface to be painted, two rows of guardrail shall be placed on all sides of the scaffolding. One row of guardrail shall be placed at forty two inches above the scaffolding and the other row at twenty inches above the scaffolding.

When the scaffolding is suspended at least twenty one inches, but less than forty three inches below the surface to be painted, a row of guardrail shall be placed on all sides of the scaffolding at twenty inches above the scaffolding.

Two rows of guardrail shall be placed on all sides of scaffolding not previously mentioned. The rows of guardrail shall be placed at forty two and twenty inches above scaffolding, as previously mentioned.

All scaffolding must be at least twenty four inches wide when guardrail is used and twenty eight inches wide when the scaffolding is suspended less than twenty one inches 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 shall 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 shall be an opening in the guardrail, properly located, to allow the Inspector access onto the scaffolding.
The rails and uprights shall be either metal or wood. If pipe railing is used, the railing shall have a nominal diameter of no less than one and one half inches. If structural steel railing is use, the rails shall be 2 X 2 X 3/8 inch steel angles or other metal shapes of equal or greater strength. If wood railing is used, the railing shall be 2 X 4 inch (nominal) stock. All uprights shall be spaced at no more than 8 feet on center. If wood uprights are used, the uprights shall be 2 X 4 inches (nominal) stock.

When the surface to be inspected is more than fifteen feet above the ground or water, and the scaffolding is supported from the structure being painted, the Contractor shall provide the Inspector with a safety belt and lifeline. The lifeline shall not allow a fall greater than six feet. The Contractor shall 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 above the ground, the Contractor shall provide a ladder for access onto the scaffolding. The ladder and any equipment used to attach the ladder to the structure shall be capable of supporting 250 pounds with a safety factor of at least four (4). All rungs, steps, cleats, or treads shall have uniform spacing and shall not exceed twelve inches on center. At least one side rail shall extend at least thirty six inches above the landing near the top of the ladder.

An additional landing shall be required when the distance from the ladder to the point where the scaffolding may be accessed, exceeds twelve inches. The landing shall be a minimum of at least twenty four inches wide and twenty four inches long. It shall 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 twelve inches. The landing shall be rigid and firmly attached to the ladder; however, it shall not be supported by the ladder. The scaffolding shall be capable of supporting a minimum of one thousand pounds.

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 shall furnish all necessary traffic control to permit inspection during and after all phases of the project.

PROTECTION OF PERSONS AND PROPERTY

The Contractor shall collect, remove and dispose of all buckets, rags or other discarded materials and shall leave the job site in a clean condition.

The Contractor shall 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 shall 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 shall be suitably anchored and reinforced to prevent interfering with normal traffic operations in the open lanes. Payment for the shields shall be included as incidental to the applicable field coating operation. Work shall 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 shall restore, at his own expense, such property, to a condition similar or equal to that existing before such damage or injury was done.

**POLLUTION CONTROL**

The Contractor shall take all necessary precautions to comply with pollution control laws, rules or regulations of Federal, State or local agencies.

**WORK LIMITATIONS**

Abrasive blasting and painting done in the field shall 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.

**METHOD OF MEASUREMENT**

The method of measurement shall be according to the pertinent of 863, 514 and 516 of the current Construction and Materials Specification.

**METHOD OF PAYMENT**

The area of the top of the top flange which shall receive a mist coat of the prime coat only shall not be included in the measurement for the pay item.

The cost of the above including labor, material, equipment and incidentals shall be included in the unit price bid for each item as listed below.

Payment for this item shall include all labor, surface preparation, materials, inspection and equipment to shop apply a prime, intermediate and finish paint coat at the item 863 structural steel fabricator's shop facilities. Also included in this item is all required field site surface preparation, cleaning, painting to coat the surfaces prime coated in the fabrication shop. Repair of
damaged shop paint caused during shipping, erection or construction procedures is the responsibility of the Contractor and shall not be considered for payment.

The cost of the surface preparation and prime coat to the top of the top flange shall be included as an incidental for payment under Shop Painting and Field Touch-Up of Structural Steel.

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<thead>
<tr>
<th>ITEM</th>
<th>UNIT</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Lump Sum</td>
<td>Shop Painting and Field Touch-Up of Structural Steel</td>
</tr>
</tbody>
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ITEM 516 - STEEL POT BEARINGS

This un-numbered proposal note is to be supplied when a pot bearing is being specified. The note needs to be revised for the specific project and a copy furnished with the plans to be included in proposal note. As this is a very special 516 item, the proposal note 540 shall not be specified for this item.

ITEM 516 - STEEL POT BEARINGS

March 7, 1997

1. 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.

The proposal note "516, 517, 518 Fabricated Members shall not apply to this item. Shop drawings are required for steel pot bearings"

1.2 The pot bearing shall consist of the following parts:

1. 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. On guided bearings, sole plate shall have a guide bar. Sole plate rides on piston.

2. Circular piston - Top faced with PTFE for expansion bearings. On guided bearings, piston is to have recess for guide bar with vertical sides faced with PTFE. Piston sits in steel pot on lubricating and elastomeric discs.

3. Combined Rectangular Sole Plate - Circular piston (for fixed bearings) - Top side beveled to the slope of the girder and welded to the girder flange. Bottom side level and sits in steel pot on lubricating and elastomeric discs.

4. Elastomeric Disc - Confined within pot for the purpose of providing rotation and support for the piston. Lubricating discs are provided above and below the elastomeric disc. The disc is sealed with brass sealing rings.

5. Sealing Rings - Seal between pot and piston used to contain the elastomeric disc.

6. Guide Bar (for Guided Bearings) - Attached to or integral with sole plate for purpose of guiding expansion bearings and transmitting lateral loads to the pot.
7. Circular Pot - Containment for the elastomeric disc and transmission of vertical and lateral loads to masonry plates. Field welded to masonry plates.

8. 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.

1.3 Bearing Height

1. The total bearing height shown in the plans shall be met by increasing the sole plate thickness or pot base thickness or masonry plate thickness or adding a make-up plate between the pot and masonry plate or a combination thereof.

2. If a make-up plate is used it shall have a minimum thickness of 0.50 inch, an outside diameter of 1.25 inches greater than the pot and be shop welded to the pot by a 5/16 fillet weld.

2. 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, Division I, Section 19 and Division II, Section 18.3.

2.2 Sole Plate

1. ASTM A588 or A572 Grade 50 Steel with stainless steel sliding surface on under side. Top side beveled to the slope of the girder. Bottom side level.

2. Stainless steel sheet surface shall conform to ASTM A167 or A240 Type 304. The minimum thickness shall be 0.06 inch. Stainless steel in contact with PTFE shall have a 20 micro-inch RMS finish or better. The surface shall be mechanically polished. Material and finish shall be such that the requirements of 4.2 (2) are met.

3. Rectangular or square in plan.

4. Minimum plan dimensions shall be pot diameter plus design movement shown in the plans plus 2 inches.

5. Minimum thickness shall be 0.75 inches.

6. For guided expansion bearing guide bar in sole plate, see 2.7.
2.3 Piston

(1) ASTM A572 Grade 50 or A588 Steel.

(2) Diameter of piston shall be 0.03 inch less than the inside diameter of the pot.

(3) Piston thickness shall be sufficient to provide a clearance of \((0.02 \times \text{Pot O.D.} + 0.12)\) inches between the top of the pot wall and surface above (guide bar if guided or bottom of sole plate) the pot wall when the piston in an unrotated position.

On guided expansion bearings, the piston thickness shall be sufficient to transmit a lateral load equal to 10% of the vertical loads shown in the plans from the guide bar to the pot wall without deflection/distortion.

(4) Piston walls shall be tapered inward, toward the top, to prevent binding against the pot walls during rotation, and the bottom edge shall be rounded with a machined 0.125 inch radius.

(5) The piston shall be machined from a single piece of structural steel.

(6) On guided bearings, the bottom of the recess shall be a minimum of 0.375 inch clear to guide bar in sole plate.

(7) The thickness of the piston shall ensure that the bottom of the piston shall be entirely below the top of the pot up to 200 percent of maximum design rotation.

(8) The top of the piston and the sides of guide bar recess on guided bearings used for expansion bearings shall be faced with PTFE. The PTFE surface shall consist of finished unfilled PTFE sheet made from virgin PTFE resin or 100 percent PTFE fabric made from virgin PTFE multi-filament fiber. Material and finish shall be such that the requirements of 4.2 (2) are met.

(9) PTFE fabric fibers shall conform to the following:

   A. The resin from which the fibers are produced shall be 100 percent PTFE conforming to ASTM D1457.

   B. Tensile Strength - ASTM D2256 - 24,000 PSI (minimum).

   C. Elongation - ASTM D2256 - 75 percent (Minimum).
D. The TFE fabric shall have a minimum thickness of 0.0625 inch (compressed). Maximum thickness shall be 0.125 inch (compressed).

(10) Finished unfilled PTFE sheet shall be made from 100 percent virgin PTFE resin and shall conform to the following requirements:

A. Tensile strength D1457 2800 PSI (minimum).
B. Elongation ASTM D1457 - 200 percent (minimum).
E. Minimum thickness shall be 0.187 inch. Sheet shall be recessed one half its thickness into steel substrate.
F. PTFE sheet shall be commercially etched on its bonding side.

2.4 Combined Sole Plate and Piston

(1) All applicable provisions of 2.2 and 2.3 shall apply.

(2) The piston thickness shall be sufficient to transmit a load equal to 10% of the vertical load shown in the plans to the pot wall without deflection/distortion.

2.5 Elastomeric Disc

(1) The elastomeric disc shall meet the following average compressive stress requirements:

A. Maximum of 3,500 psi when the bearing vertical design capacity specified in the plan is applied to the area of the disc.

B. Minimum of 700 psi when the lessor of the dead load or 20% of the bearing vertical design capacity specified in the plan is applied to the area of the disc.

(2) Minimum disc thickness shall be 0.067 x disc diameter.
(3) 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.

(4) Hardness shall be 50 durometer +/- 10.

(5) The disc shall consist of one solid piece of elastomer.

(6) The elastomeric disc shall be lubricated by means of 0.015 inch thick unfilled PTFE disc the same inside diameter as the pot, one located above and one below the elastomeric disc or by a lubricant recommended by the manufacturer and approved by the director.

(7) Two flat brass sealing rings shall be used to seal the disc. The upper edge of the disc shall be recessed to receive the sealing rings so that they sit flush with the upper surface of the top lubricating disc.

2.6 Sealing Ring

(1) Rings shall be flat and made of brass conforming to the requirements of ASTM B36, half hard.

(2) Minimum width shall be 0.375 inch.

(3) Minimum thickness shall be 0.050 inch.

(4) The rings shall have a smooth finish of 64 micro-inch (RMS) or less.

(5) Two rings are required.

(6) The rings shall be split and snugly fit the recess in the elastomeric disc as well as the inside diameter of the pot. 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 when installed. The rings shall be arranged to have the splits staggered a minimum of 90 degrees relative to one another.

2.7 Guide Bars

(1) ASTM A36, A572 Grade 50 or A588 faced with stainless steel.
(2) 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 faces with stainless steel, see 2.2 (2). Welding of guide bars to the sole plate shall be performed prior to welding of stainless steel to the sole plate or guide bars.

(3) The total space (both sides) between the guide bars and guided members shall be 0.125 inch minus 0, plus 0.0625 inch.

(4) The guide bars shall be designed for no less than a lateral horizontal force equal to 10% of the vertical force.

(5) Guiding arrangements shall be designed so that the guided member is always within the guides at all points of translation and rotation of the bearing.

(6) See 2.3 (6).

2.8 Pot

(1) A572 Grade 50 or A588 steel.

(2) The Pot shall consist of a solid plate into which a circular recess has been machined.

(3) Depth of the circular recess shall be equal to or greater than \((0.02 + R) \times \frac{D}{2} + 0.10 + \text{thickness of the elastomeric and lubricating discs}\) where \(R = \text{Design Rotation; } D = \text{Elastomer Diameter.}\)

(4) The pot inside diameter shall be the same as the elastomeric disc, see 2.5.

(5) The outside of the pot shall be circular.

(6) The thickness of the pot wall shall be sufficient to transmit a lateral horizontal force equal to 10% of the vertical load shown on Sheet 25 of 28 to the pot base with the load applied at a point contact at two times the design rotation without causing deflection/distortion to the pot wall or base.

(7) The minimum thickness of the pot beneath the elastomer for a bearing directly on a masonry plate shall be the greater or more of the 0.045 x pot I.D. or 0.50 inch and meet the requirement of 2.8 (6).
2.9 Make-up Plate
   (1) ASTM A572 Grade 50 or A588 steel.

2.10 Masonry Plate
   (2) ASTM A572 Grade 50 or A588 steel.

2.11 Bearing pad sheet lead shall conform to ASTM B-29.

3. **FABRICATION**

3.1 Attachment of Sheet PTFE to substrate.
   (1) PTFE sheet shall be recessed into and bonded to a steel substrate.

   (2) PTFE shall be recessed for one half its thickness.

   (3) 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.

   (4) The adhesive material and the bonding procedures to be used shall be submitted to the director for approval prior to performance of the bonding operation. The bonding operation shall then be performed under controlled conditions and in accordance with these approved procedures.

   (5) After completion of the bonding operation, the PTFE surface shall be smooth and free from bubbles.

3.2 Attachment of PTFE Fabric to Substrate
   (1) PTFE Fabric shall be mechanically interlocked and bonded to the steel substrate.

   (2) 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.

   (3) The mechanical interlock and adhesive bonding material and procedures shall be submitted to the Director for approval prior to performance of the bonding operation.
The bonding operation shall then be performed under controlled conditions and in accordance with these approved procedures as approved by the Director.

(4) Migration of epoxy through the fabric will not be permitted.

(5) 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

(1) Stainless steel shall be attached to its steel substrate with an approved epoxy to ensure complete contact, and then seal welded. Seal welds shall be continuous for the entire periphery of the stainless overlay. The entire stainless steel surface shall conform to the requirements of Section 2.2 (2) after welding.

3.4 Corrosion Protection

(1) All steel surfaces (including A588 Steel) exposed to the atmosphere, except stainless steel surfaces, shall be shop prime coated in accordance with Item 514, System IZEU.

3.5 Welding

(1) Welding as a means of attachment shall be done in a controlled manner and shall conform to CMS Item 513. 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 degrees (F), as determined by temperature, indicating pencils, or other suitable means.

3.6 Tolerances

(1) 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, 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 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 over 30": -0", +1/4".
2. Plan dimensions under 30": -0", + 3/16"
3. Flatness of surface in contact with beam or girder - Class B.
5. Thickness: -1/32", +1/8"

D. Tolerances - Piston

1. Diameters greater than 20", +0.007"
2. Diameters less than 20": +0.005"
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

E. Tolerances - Sole Plate/Piston

1. All applicable provisions of C and D
F. Tolerances - Elastomeric Disc

1. Diameters greater than 20" : +3/32"
2. Diameters less than 20" : +1/16"
3. Thickness: -0", +1/8"

G. Tolerances - Guide Bar

1. Length (unless integral): +1/8"
2. Flatness of backing surface for stainless steel: Class A
3. Inside of bar to inside of bar: Nominal dimension + or 1/32"
4. Guide bars shall be not more than 1/32" out of parallel.
5. Cross sectional dimensions: +1/16"

H. Tolerances - Pot

1. The inside diameter shall be machined to a tolerance of + 0.005" up to 20" diameter and + 0.007" over 20" diameter/
2. Pot underside shall be machined parallel to the inside to a Class A tolerance.

I. Tolerances - PTFE Substrates

1. 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 that there are no sharp edges.

L. Tolerances - overall heights of bearing: -1/16", and 1/8"
M. Tolerance - Make-up Plate

1. Plan dimension: -0", +1/4"

2. Thickness: -1/32", +1/8"

3. Flatness: Class B top and bottom

N. Tolerances - Masonry Plate

1. Plan Dimensions: -0", +1/4"

2. Thickness: -1/32", +1/8"

3. Flatness - Class C for the underside, Class B for the upper side.

4. TESTING

4.1 General

(1) 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 final results are obtained from raw data.

(2) Sampling

A. One guided expansion bearing and one fixed bearing shall be chosen, selected at random, from each applicable lot of completed bearings.

1. One lot shall consist of no more than 25 bearings of one load category.

2. One load category shall consist of bearings having vertical load capacity within a range of no more than 200 kips.

4.2 Friction test shall be performed on expansion bearing samples chosen as described in Section 4.1 (2) above.
The test shall be conducted at the maximum working stress for the bearing with the load applied continuously for 12 hours prior to measuring the friction. Maximum working stress shall be determined by dividing the maximum vertical force (obtained from the plans) by the areas of PTFE used on top of the piston.

The static and dynamic coefficient of friction shall be determined. A sliding speed of less than one inch per minute shall be used. The coefficient of friction thus determined shall not exceed 0.04.

Proof load test shall be performed on bearing samples chosen as described in Section 4.1 (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 rated 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 limited rings, or cracked steel, 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.

Adhesion between the PTFE and substrate shall be tested on a test specimen in accordance with ASTM D429, Method B. The minimum peel strength shall be 25 lbs. per inch. This test is in addition to adhesion determined under 4.2 and 4.3 above.

Test results shall be presented in a report showing raw test data, reduced test data, sample calculations, and final results along with photographs and conclusions.

Certified test data for all stainless steel, A36, A572 or 588 Steel and PTFE shall be furnished to the Director showing compliance with the requirements of this specification.

The Director may require additional bearings to be tested even though required bearing tests have been acceptable. Such additional tests will be paid for under Item Special, Additional Bearing Tests, pot bearings.
5. **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. This requirement does not apply to the masonry plate or 1/8" sheet lead which shall be shipped for separate installation. 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. **INSTALLATION**

6.1 Field welding of bearing to masonry plate shall meet the requirement of 3.5 (1).

6.2 Bearing shall be evenly supported over their upper and lower surfaces under all erection and service conditions.

6.3 Align the centerlines of the bearing assembly with those of the substructure and superstructure or on guided bearings, align the bearings to allow for the designated expansion directing of the structure.

6.4 Erection bars shall be fastened to the beam flange to accurately position the girder onto the sole plate of the bearing. The guide bar on guided bearings must be in parallel with the girder. Tolerance in setting the guide bar shall be 1/32" in the length of the bar out of parallel.

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 Set offsets of upper and lower bearing parts to compensate for ambient temperature and as required by plans.

6.7 Concrete bearing seats shall be prepared at the correct elevation and shall be level within 1:200.
6.8 Field paint exposed steel in accordance with Item 514, System IZEU.

7. **METHOD OF MEASUREMENT**

7.1 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.

7.2 Additional bearing tests, if required by the Director, will be measured on an each basis for a successfully tested and accepted bearing. Tests resulting in a rejected bearing will not be measured and paid for.

8. **BASIS OF PAYMENT**

8.1 Payment for pot bearings will be made at the contract unit price per each listed under: Item 516 - 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.
APPENDIX - METRIC SEPTEMBER 1998

APP-8  RETIRED NOTE 50

[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.

Note retired as the standard now includes epoxy and the plastic film was eliminated.

APP-9  RETIRED NOTES 52 & 53

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

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.

Notes 52 and 53 were retired as CMS 1997 covers issues of pipe.
APPENDIX -METRIC SEPTEMBER 1998

APP-10 RETIRED NOTE 66

If A-588M unpainted steel is used the following note should be placed on the structural steel detail sheet:

[66] HIGH STRENGTH BOLTS shall be __________ diameter A325M unless otherwise noted.

Original note used to assure weathering steel bolts - now covered by CMS 711.09

APP-11 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.

Note retired because standard drawing SD-1-69 retired.

APP-12 RETIRED NOTE 78

The following note should be provided for bridge superstructures using A588M steel which is to be left unpainted:

[78] A588M STEEL is to be left unpainted. The outside surfaces and bottom surfaces of the bottom flanges of facia 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 include in item 513.

Note retired as the requirements were added to CMS 513.221 and SS 863.30

APPENDIX -1-83
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 300 mm long drip strip shall also be installed centered on each post.

The strips shall be fastened at 450 mm C/C maximum with 32 x 3 mm galvanized or stainless button head spikes (length x shank diameter) or 3 mm 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 200 mm wide x 0.8 mm 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, meters, Steel Drip Strip, which shall include all materials, labor, tools, and incidentals necessary to complete the item.

Note retired as standard drawing was developed to replace it.

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 300 mm 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 0.8 mm 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, meter, Steel Drip Strip, which shall include all materials, labor, tools, and incidentals necessary to complete the item.

Note retired due to replacement by standard drawing.