IDAHO TRANSPORTATION DEPARTMENT
BRIDGE DESIGN LRFD MANUAL

JULY 2009

GENERAL INFORMATION

The Idaho Transportation Department LRFD Bridge Design Manual is intended to supplement and provide interpretation of the AASHTO LRFD Bridge Design Specifications, As well as provide guidance for designers, checkers, and consultants.

The LRFD Bridge Design Manual provides guidelines and standard details that are workable, serviceable, and reasonably economical. In addition, these guidelines and details have been approved by FHWA for general use. Any departure or deviation from these guidelines and standard details in this manual will require prior approval, and may result in delay caused by obtaining approval.

It will continue to be the responsibility of the designers to exercise their best engineering judgment when designing and ensure the guidelines and details provided in the LRFD Bridge Design Manual are applied properly.

All new bridges that carry traffic on Interstate Highways and Ramps or US and State Highways shall be designed using the AASHTO LRFD Specifications and the guidelines in this manual unless otherwise approved by the Bridge Engineer.

All rehabilitation, widening and modification to existing structures may use the AASHTO Standard Specification of Highway Bridges-17th Edition or the AASHTO LRFD specification if it is economically feasible.

Commentary

FHWA Policy Memorandum issued June 28, 2000, requires all new bridges on which States initiate preliminary engineering after October 1, 2007, shall be designed by the AASHTO LRFD Bridge Design Specification. All new culverts, retaining walls and other standard structures on which States initiate preliminary engineering after October 1, 2010, shall be designed by LRFD.

The term “preliminary engineering” shall be interpreted as the initiation of the studies or design activities related to identification of the type, size, and/or location of bridges. The term “initiate” means the date when Federal-Aid funds are obligated for preliminary engineering. In cases where Federal-Aid funds are not used in preliminary engineering, but are used in construction or other phases of the project, the term “initiate” means the date when the State obligates or expends their own funds for preliminary engineering.

Superstructure, substructure, and foundation bridge elements shall be designed by LRFD.

Shelved bridge projects designed and packaged for construction prior to October 1, 2007, are not subject to the LRFD Policy Memorandum, unless a redesign is required by the State after October 1, 2007.

The term “new Bridges” shall be interpreted to include both new and total replacement bridges.

The Policy applies to all States-initiated federal-Aid funded projects, not just those funded with Highway Bridge Program funds, including on system and off-system projects.

Revisions:

April 2008	Added Commentary.

CONSTRUCTION DRAWINGS

Submittal
Working drawings shall be submitted by the contractor in accordance with subsection 105.02 of the Standard Specifications for Highway Construction.

Checking
Checking of shop drawings, falsework drawings, and erection drawings is a normal duty of the Engineer of Record. The Bridge Section shall check drawings for designs completed by the Bridge Section staff. For consultant designs, the consultant should do the checking as provided for in the Consultant Services Agreement. Checking must be thorough, accurate, and complete.

If information is incomplete, the engineer shall request that the additional data be submitted in writing and shall not approve the shop drawings until the information has been received and reviewed. No verbal approval shall be given to a contractor, supplier, or fabricator. All approvals, verbal and written, shall be given to the Resident Engineer.

Approval of materials is generally the responsibility of the Materials Engineer. The Bridge Section will provide advice/recommendations on materials when requested.

Construction drawings will be checked promptly upon receipt of complete information and at a speed consistent with thoroughness and accuracy.

Archives
22”x34” 3 mil mylar of the approved shop drawings for all elements that become a permanent part of the structure shall be submitted by the Contractor and shall be retained with the contract plans for the structure.

Revisions:
April 2008 Added information on Submittal & Archives.

CONSULTANT PROJECT REVIEW PROCEDURE

Consultants are used on state bridge projects when the workload exceeds the capacity of the Bridge Section or when special expertise is required. Counties and cities also frequently use Consultants to design bridges for LPA projects with Federal Aid funds under LHTAC project management.

A Bridge Section Engineer will be assigned when a new bridge design project is provided to the consultant. The reviewing engineer should have experience in the design of structures similar to the one to be reviewed. The Review Engineer will then become the contact for technical questions raised by the consultant throughout the design phase. This communication allows early identification of critical design areas and reduces the chances of major revisions.

The Review Engineer will not generally perform an exhaustive check on the design. All details, plans, and related work will be reviewed to ensure conformance with the criteria that follows.

The consultant shall apply his own seal and signature to the plans, and thereby assumes full responsibility for their correctness and general conformance with good engineering practice.

The following indicates the degree and type of checking to be performed by the Bridge Section:

Structure Concept Study Review

All concept studies will be thoroughly reviewed to ensure adequate evaluation of:

- Structure types that are compatible to the site conditions
- Preliminary cost estimates
- Advantages/disadvantages of each structure type
- Economy, feasibility, and constructibility
- Structure types recommended for additional study or final design

A review of the bridge layouts will be made to ensure that span lengths, clearances, and all site conditions are adequately addressed.

Final Situation and Layout

Plans should be thoroughly checked by using the checklist located in Chapter 17 of the Bridge Design Manual (*BDM)*. Conformance of grades, alignments, and other data between roadway and bridge plans should be checked.

The hydraulic and foundation criteria shown on the plans shall conform to the approved ITD-210 hydraulic report and approved Phase 4 foundation report.

Final Design Review

Plans should be reviewed for completeness, constructibility, compliance with current ITD standards, and good engineering practices.

A review of major structural elements should be performed. However, no stress analysis is generally required unless the detail appears questionable.

Pay items and Special Provisions should be reviewed for conformance with the Standard Specifications. Quantity calculations and rebar schedules are generally not checked in detail.

Design calculations shall be on 8.5"x11" paper with a proper heading and placed a binder with an index. The signature, date and Idaho seal of a registered engineer of the consulting firm shall be on a title sheet.

Final Plan Review

Plans will be checked for all changes required by the Final Design Review.

The signature, date and Idaho seal of a registered engineer of the consulting firm shall be on each drawing.

Any new or significantly modified Special Provisions shall be added to the X:drive SPB folder.

CONSULTANT SUBMITTAL PROCEDURE

Local Sponsor Projects
Sponsors shall make all submittals to the Local Highway Technical Assistance Council (LHTAC), and LHTAC will make distribution to the necessary ITD Sections for review. Two copies of the Drawings should be submitted in 11" x 17" format. Returned transmittals by the Bridge Section will be sent to the Roadway Design Area Engineer. This procedure is to be used on all projects to include Local Public Agencies (LPA) projects by consultants.

State Consultant Projects
Consultants shall make all submittals to the District Design Section, and the District will make distribution to the necessary ITD Sections for review. Two copies of the Drawings should be submitted 11" x 17" format. Returned transmittals by the Bridge Section will be sent to the Roadway Design Area Engineer.

Submittal Criteria
The following data is required for the various submittals:

Structure Concept Review
A submittal of data is to be made showing the concepts of the project in general. Drilling for the foundation investigation for multi-span structures should be delayed until the concept is approved. The data should include:

- Bridge layout showing plan and elevation views
- Bridge cross-section
- Roadway cross-section
- Stream cross-section,
- Vicinity map
- Preliminary profile grade
- Draft Phase 4 Foundation Report (if available)
- Draft ITD-210 hydraulic report
- Other data pertinent to type or location selection
- Type, Size & Location Report

Show as much of the above data as possible on the layout drawing.
Refer to Article 0.7 for TS&L Report criteria.
Consultants are encouraged to contact the Bridge Section during development of the structure concept.

Final Situation and Layout Review
The plans shall consist of the following:

- Situation and layout
- Foundation investigation sheet
- Sketches or views of unusual structural details

Refer to the Situation and Layout Checklist in Chapter 17 of the *BDM*.

The plans should also be accompanied by:

- Approved Phase 4 Foundation Report
- Approved ITD-210 Hydraulic Report
- Approved ITD-783 Design Standards forms
- District approved roadway profile and alignment data
- Topographic map with contours

The Bridge Section shall approve the Situation and Layout plans before proceeding with final design.

Intermediate Design Reviews
If needed, these reviews can be handled informally between the Local Public Agency/State Consultant and the Bridge Section.

Final Design Review
The submittal should include the following:

- Drawings in reproducible form
- Special Provisions
- Cost Estimate
- Quantity Calculations
- Construction Schedule
- Stamped Design Calculations
- Stamped Check Calculations
- Consultant QA/QC Check Lists

Plans, Specifications & Estimates Submittal
After the consultant has made the necessary corrections from the Final Design Review, the final drawings and the revised final design data shall be submitted. In addition, the bridge shall be load rated using Virtis in accordance with the latest version of the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*.

The final drawings shall include the following:

- 22”x34” 3 mil mylars stamped by the Engineer
- 11”x17” prints
- Electronic CADDS files in .dgn format

The 11”x17” prints should preferably be either photographically reduced or have an electronic stamp.

The initial Virtis load rating may be based upon the details shown in the contract plans for the PS&E submittal. The final load rating shall reflect the changes made during construction (fabrication change, field adjustment, contractor change, etc.). The electronic file shall be sent to ITD Bridge Inspection.

The Bridge Section will publish a PS&E letter of acceptance.

In the transmittal letter for Local Sponsor Projects, the Bridge Section will include an estimate of man-hours for checking shop plans and construction drawings. The District will arrange for a supplemental engineering agreement to cover this additional work.

Revisions:

April 2008	Added TS&L Report requirements in Structure Concept Review. Added requirement for Virtis load rating.
July 2009	Added Check Calculations & QA/QC check lists submittal at Final Design.

IN-HOUSE DESIGN & CHECKING PROCEDURES

The primary aim in both designing and checking is to produce a structure that will safely carry the anticipated loads.

The design team, consisting of the designers, checkers, and structural detailers, is responsible for developing a set of practical, clear, and concise design notes, plans, and specifications by the assigned due date with the allotted manpower.

The *BDM* provides standard details that are workable, serviceable, and reasonably economical. In addition, these details have been approved by FHWA for general use. Departure from these standards may result in delays caused by obtaining approval.

Design Calculations
The design calculations shall be prepared on 8½"x11" sheets. The cover sheet for the final design notes of record shall be stamped by the designer & checker, and shall be indexed with numbered pages. The design calculations of record and check calculations shall be microfilmed and returned to the designer/checker.
A sample form for stamping the design calculations is shown on page A0.1. An electronic copy is available on the X:drive in the Bridge Design Aids folder.

Designer
The designer's primary responsibilities include:

- Design concept and layout
- Structural design
- Preparing complete and legible calculations
- Producing a complete set of plans and specifications
- Resolving construction problems
- Load rating new/replacement bridges using VIRTIS in accordance with the latest version of the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*.

The designer should advise and get concurrence from the Group Leader whenever deviating from approved office standards and practices.

The initial Virtis load rating may be based upon the details shown in the contract plans for the PS&E submittal. The final load rating shall reflect the changes made during construction (fabrication change, field adjustment, contractor change, etc.).

The designer should inform the Group Leader of any areas of the design that should receive special attention during checking and review.

The designer's responsibilities also include the following project planning activities:

- Preparing a Design Time Estimate CPM Chart
- Identifying tasks and planning order of work
- Preparing design criteria for inclusion in the front of the design calculations
- Determining the number and titles of plan sheets
- Coordinating plan sheet detailing
- Coordinating computation of quantities
- Preparing the Cost Estimate, Construction Schedule CPM Chart, and Special Provisions
  Any new or significantly modified Special Provisions shall be added to the X:drive SPB folder.

Design Checker
The primary purpose of a design check is to insure that the designer has not, through an error in mathematics, misunderstanding of the specifications, or other cause, produced an unsafe design.

The design checker's primary responsibilities include:

- Verifying the design theory and correct interpretation of the design code
- Accuracy and completeness of the design calculations to confirm the structural adequacy of the components
- Independent check of major controlling geometry

The design calculations should not be checked until the Situation Layout check is completed and any differences are resolved with the designer. If revisions are necessary, the designer should revise the design and details before the design checker proceeds.

The design checker may perform an independent analysis by using a methodology different from the original design. The check notes shall be stamped and shall be returned to the designer who will coordinate changes.

For designs checked by an experienced engineer, the original calculation sheets may be initialed by the checker.

For special designs or those done by inexperienced designers, the Group Leader may require a more complete design check by the design checker.

Plan Checker
The primary purpose of a plan check is to insure plans are constructible, consistent, clear, and complete. The checker's responsibilities should include, but not be limited to, the following items:

Situation and Layout
- Make a complete check of the geometric layout
- Check the Typical Section for conformance to the roadway width and bridge railing curb-curb requirements
- Check the girder spacing and type, and slab thickness for conformance to the Typical Section and office standards

Major Component Details
- Verify that the details are in agreement with the approved design calculations

Structural Detailer
The structural detailer's primary responsibilities include:
- Preparing neat, correct, and easy to follow plan sheets conforming to current detailing standards
- Drawing details to scale
- Determining dimensions and elevations as required by the designer/checker
- Calculating quantities as directed

Group Leader
The Group Leader should work closely with the designer, design checker, and structural detailer during the design and plan preparation phases to help avoid major changes late in the design process.

The Group Leader's primary responsibilities include:
- Compatibility of design and details within the project
- Determining the level of checking required by considering the complexity of the structure and the skill of the designer
- Approving the design criteria prepared by the designer before start of design
- Monitoring the design and detailing process and providing guidance and assistance as required
- Reviewing the design calculations for completeness and for agreement with office criteria and practices
- Reviewing the plans for completeness, constructibility, and agreement with office criteria and practices
- Reviewing the PS&E data for completeness and for agreement with office criteria and practices

Bridge Engineer
The Bridge Engineer provides leadership and support to assure bridge design quality for structural designs.

The Bridge Engineer's primary responsibilities include:
- Reviewing and approving the Situation Layout to assure that the most cost-effective and appropriate structure type is selected for a particular bridge site.
- Facilitating resolution of major project design issues.

- Performing a structural/constructability review of the plans.
- Reviewing the project special provisions and Supplemental Specifications

Revisions:

June 2006	Added paragraph for Design Calculations. Added preparation of design criteria in Designer's Responsibilities. Added paragraph for duties of Bridge Engineer.
April 2008	Added load rating responsibility for Designer
July 2009	Added requirement for microfilming check calculations.

MAJOR OR UNUSUAL BRIDGES

Many major or unusual structures are designed to utilize features that are at the edge of today's state-of-the-art or unique fracture critical details that often require special consideration during construction, inspection, and maintenance.

The following FHWA policy is to be followed in the preparation of design documents for major or unusual bridges to help avoid construction problems and provide guidelines for on-going maintenance:

- FHWA Memorandum November 13, 1998
  Project Oversight Unusual Bridges and Structures

Unusual bridges are those that have any of the following:

- Difficult or unique foundation problems
- New or complex designs with unique operational or design features
- Bridges with exceptionally long spans
- Bridges being designed with procedures that depart from currently recognized acceptable practices

Examples of unusual bridges include:

- Cable-stayed
- Suspension
- Arch
- Segmental Concrete
- Movable
- Truss

Examples of bridge types that deviate from AASHTO bridge design standards or AASHTO guide specifications for highway bridges include:

- Major bridges using LRFD design specifications
- Bridges requiring abnormal dynamic analysis for seismic design
- Bridges using a three-dimensional computer analysis
- Bridges with spans exceeding 500 feet
- Bridges with major supporting elements of ultra high strength concrete or steel

A manual for the inspection and maintenance of the structure shall be prepared during the design process that includes explicit guidelines and instructions. The lead designer, whether the ITD Bridge Section or a Consultant does the design, will be responsible for the preparation of the inspection and maintenance manual. The manual will identify critical areas to be inspected and recommend maintenance procedures. The following areas will be addressed as well as any other areas requiring special attention:

Deck
Girders
Expansion Joints
Bearing Units
Piers or Columns
Abutments
Connections
Fracture Critical Members (to be marked on the drawings)
Member cracking or deformation

Copies of the manual shall be sent to the District, Bridge Maintenance and Bridge Section.

Commentary

The FHWA November 13, 1998 Memorandum is available at http://www.fhwa.dot.gov/bridge/unusual.htm

The FHWA Implementing Guidance memorandum is available at http://www.fhwa.dot.gov/bridge/sec1305.htm

PUBLIC CONVENIENCE

The Bridge Section is committed to public safety and the enhancement of public convenience in all our operations. The Bridge Section shall strive to comply with Board Policy B-01-01, Public Convenience.

The following are goals to increase both safety and public convenience:

Inspection: Eliminate or minimize traffic interference during inspection operations

Permits: Review overweight permits in a timely manner.

Design: Minimize traffic interference during construction.
Access for inspection and maintenance shall be considered as part of the design, with emphasis on utilization and safety of personnel. All facilities shall be designed to prevent unauthorized access to the structure.
Bridges replacements should enhance commercial routes.

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TYPE, SIZE & LOCATION REPORT for CONSULTANT PROJECTS

GENERAL

The purpose of the TS&L Report is to provide enough background information so that reviewers can effectively evaluate the proposed final design and the concepts it is based on. The report should describe the project, the proposed structure, and give reasons why the bridge type, size, and location were selected. The report should concisely summarize the information in 2-3 pages. A separate TS&L Report shall be prepared for each structure except minor projects such as deck joint rehabilitations or rail retrofits.

The intent of the TS&L Report is the collection of pertinent data required for the design of the structure, in addition to documentation of design decisions. Pertinent data shall be extracted from other discipline reports and summarized in this Report for use by the structure designers.

Even though the Hydraulics Report or Foundation Report may not be available at the time the TS&L Report is prepared, always include comments about assumptions made in consultation with the Hydraulics or Foundation Engineer.

The TS&L Report will include the following sections, where applicable. Not all sections will apply to all structures.

REPORT OUTLINE

General Background

- Project description
- Structure location (may include roadway plan & profile and aerial photography)
- Right-of-way restrictions
- Permits and restrictions
- Utility conflicts or restrictions
- Railroad clearances or restrictions
- Design Specifications

Environmental

- Wetlands
- Historical Sites
- Contaminated areas
- Recreation areas
- Threatened Species areas
- Environmental commitments listed in the environmental docu,emtation

Design Concepts Rationale for:

- Building new bridge versus widening existing one
- Use of bridge versus culvert
- Foundation support assumptions
- Assumed pile or bearing capacity loads
- Assumed lateral soil pressure against abutment
- Seismic load assumptions

Geometry and Layout

- Roadway width
- Traffic volumes
- Profile grade
- Horizontal alignment
- Design exceptions
- Sidewalks
- Railing

Hydraulics

- Waterway opening
- High water elevation
- Clearance
- Bank protection
- Floodway information
- Deck drains

Foundations

- Piling or spread footings
- Fills, surcharges
- Settlement
- Lateral earth loads
- Seismic loads

Structure Features

- Span length and span arrangement
- Type of superstructure
    - Girder type and spacing
    - Fixity at abutment & piers
    - Location of expansion joints
    - Deck thickness and corrosion protection system
- Type of substructure and location
- Alternate structure types considered and estimated costs
- Stage construction and detour requirements
- Aesthetics
- Maintenance considerations
- Feasibility of construction

Recommendations

Many bridge replacement projects require a Biological Assessment. To aid in the process, try to address as many of the following subjects as practical.

- Project timing and chronology
- Alignment and span configurations of the new bridge in relation to the existing bridge
- Proposed treatment of the runoff and comparison of the number of drains on the existing bridge to the number of drains on the new bridge.
- Discuss the sizes, numbers, and removal methods of the existing bents, footings and piles within the Ordinary High Water Mark.
- Discuss the sizes, numbers, and construction methods of the new bents, footings and piles within the Ordinary High Water Mark.
- Method of dewatering
- Method of removal and disposal of existing bridge members with lead based paint
- Discuss the construction and removal methods for detour bridges, work bridges, or falsework within the Ordinary High Water Mark
- Extent and duration of heavy machinery work in the wetted channel
- Amount or extent of fill and/or riprap
- Possible staging area and access
- Amount and type of vegetation to be removed outside and within the Ordinary High Water Mark
- Amount of wetland impacted
- Any planned mitigation

Revisions:

April 2008	Added new article.

BRIDGE PLANS

Plan sheets for the following type of projects should be prepared as Bridge Plans using the bridge sheet title block shown on page B17.3.

- New and replacement structures
- Rehabilitation/repair or widening of existing structures
- Retaining walls attached to the structure

New/replacement structures and retaining walls will be assigned a new bridge drawing number by the Group leader.

The drawing number for rehabilitation/repair or widening of existing structures shall be the drawing number of the first sheet of the existing plans followed by an alphabetic character. The alphabetic character shall start with "A" for the first modification to the structure and increment for succeeding projects. This procedure allows for the original plans and all modifications to the structure to be archived together.

Retaining wall plans include walls attached to the structure and any walls associated with any structure. All types of walls are intended including cast-in-place and MSE.

Revisions:

April 2008 Added new article.

CONSULTANT QUALITY ASSURANCE/QUALITY CONTROL

The purpose of the QA/QC procedure is to improve the quality of the structural designs and plans. ITD Bridge Design expects consultants to fully check their own work. In general, review of designs and checking of plans, calculations, specifications, and estimates should be similar to that performed by the Bridge Section for their own work. Refer to ITD Bridge LRFD Manual Article 0.4.

Quality Assurance consists of the steps needed to verify quality. This should be a defined set of procedures, with measurable and verifiable actions.

Quality Control consists of the act of reviewing and checking the design, the calculations, and the plans. Quality control should be thorough, appropriate to the project, and documented.

Review consists of verifying general conformance of the design with project objectives. This includes general features of design, constructability, and presentation of the design.

Checking consists of detailed verification of design and details. This includes checking of the design calculations, the plans, the quantities, and the specifications.

Quality Control

Preliminary Design:

- Review major features of the project; structure type, constructability, environmental mitigation, cost.

- Check of the girder capacity, clearances, geometry.

Final Design:

- Review structure type and major features to verify that changes in the overall project have not invalidated previous project decisions; presentation and thoroughness of the plans.
- Check of design calculations, details of the design, and plans.

Design Check

A complete check of the structural design calculations shall be carried out by a Professional Engineer other than the Professional Engineer responsible for the design. This design check shall be carried out by another Consultant when the design Consultant does not have adequate in-house capabilities to provide this check.

The design check notes shall be stamped and submitted with the original design calculations.

Revisions:

July 2009 Added new article.

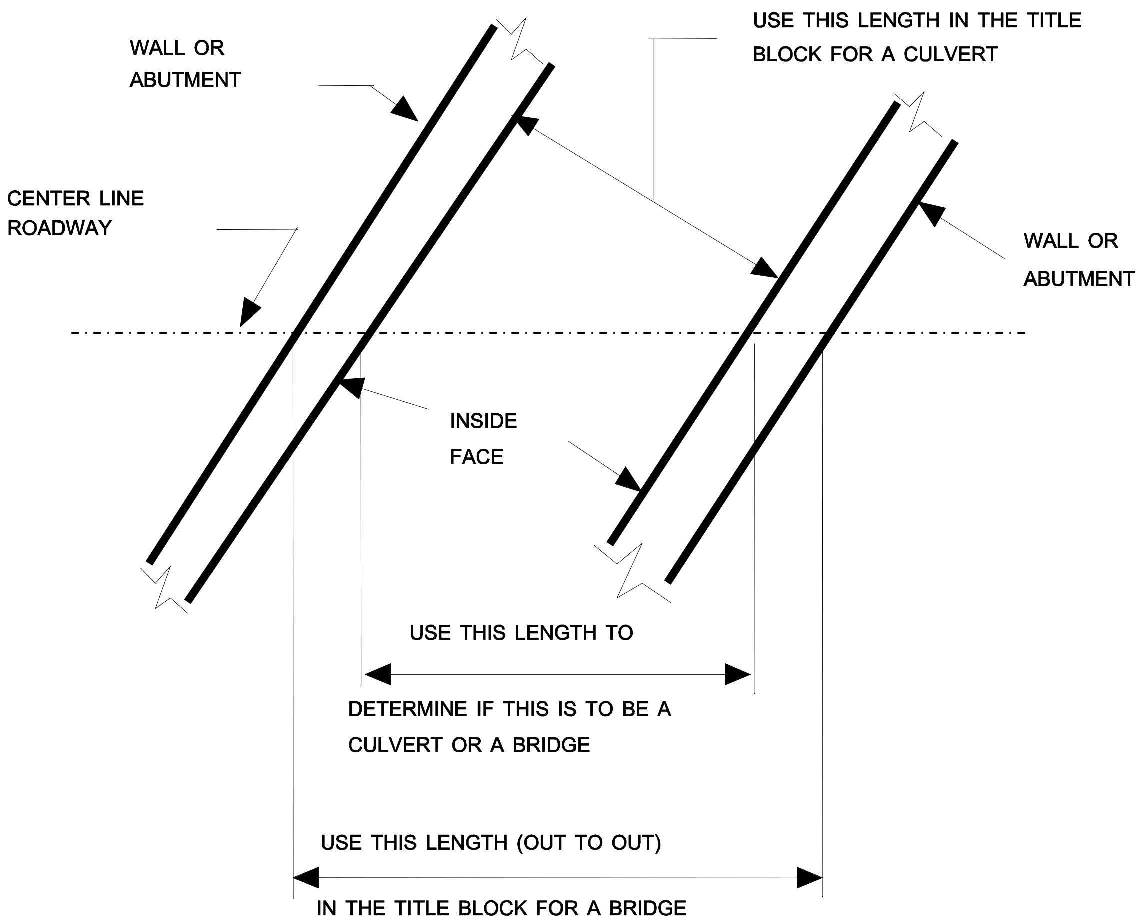
1.2 DEFINITIONS

DESIGNATION OF CULVERTS AND BRIDGES ON DRAWINGS
To determine whether a structure should be designated as a Culvert or a Bridge, the structure shall be measured along the centerline of roadway from the inside face of wall or abutment. If the measured length is 20 feet or less it shall be designated a culvert, if greater than 20 feet it shall be designated a bridge.

In drawings that are designated as "Culverts" the length shown in the title block shall be the clear span measured perpendicular to the walls.

In drawings that are designated as "Bridges" the length shown in the title block shall be the out to out distance of the abutments or walls measured along the centerline of roadway.

Commentary
The definition of “bridge” is in accordance with the Federal-Aid Policy Guide Title 23 CFR Section 650.403.



1.3.2. LIMIT STATES (RESISTANCE FACTORS)

For Extreme Event and Service Limit States the resistance factor φ shall be assumed as 1.0 unless shown otherwise.

For Strength Limit State the following articles apply:

Aluminum Structures	7.5.4 Resistance Factors
Buried Structures	12.5.5 Resistance Factors
Concrete Structures	5.5.4.2 Resistance Factors.doc
Earth Retaining Structures	11.5.6
Foundations	10.5.5 Resistance Factors
Steel Structures	6.5.4.2 Resistance Factors
Timber Structures	8.5.2.2 Resistance Factors

Revisions:
June 2006 Only articles in the ITD Bridge LRFD Manual use the .doc extension.

1.3.3 DUCTILITY

All Structures shall be designed to be ductile. The η_D factor shall be 1.0 unless measures in addition to those already in the code are used.

1.3.4 REDUNDANCY

All Structures shall be designed to be Redundant. The η_R factor shall be 1.0 unless measures in addition to those already in the code are used. Two girder and three girder bridges shall be classified as not redundant and the η_R factor shall be 1.05.

1.3.5 OPERATIONAL IMPORTANCE

The η_I factor shall be determined for each structure in accordance with the following guidelines:

- If the structure is on a route that services an emergency facility (hospital, firestation, etc.) or a major transportation facility and the detour for the emergency facility is greater than one mile the structure should be classified as Important.

- If the structure carries utilities (water, gas, and power) and these utilities would be considered essential in the case of a natural disaster or emergency, the structure should be considered Important.

If the above criteria apply, consult your Group Leader. The minimum factor shall be 1.0.

All bridges that are classified as critical under section 3.10.5 shall be considered Important.

Additional References ..\03 Load and Load Factors\3.10.5 Importance Categories.doc

Revisions:

April 2008 — Changed reference to 3.10.3 to 3.10.5 to agree with the 2008 Interims and deleted ADT & Detour Length Table.
Added minimum value.

PROTECTION OF USERS

When a pedestrian walkway is provided on a highway bridge the walkway shall be separated from the adjacent roadway by one of the following methods:

- When the design speed is less than 45 mph a raised walkway with a minimum height barrier curb of 6 inches shall be provided.
- When the design speed is 45 mph or greater, traffic or combination railing shall be provided.

All bridge railing shall meet the requirements of Section 13 of the AASHTO LRFD Bridge Design Specifications.

Commentary

The LRFD Bridge Design Specifications require walkways to be separated from the roadway with the use of a traffic or combination rail on high-speed urban expressways. The Definition for “Speed-Low/High” is in Article 13.2.

Revisions:

June 2006 Revised the low/high design speed to 45mph to conform to the 2006 Interims.

2.3.3 CLEARANCES

BRIDGES/CULVERTS OVER WATERWAYS OTHER THAN CANALS
For all closed bottom pipes with a clear span less than 12’, the ratio of the headwater to diameter during Q25 flow should be equal to or less than 1.25. In addition, the Q100 flow must pass through the pipe without overtopping the roadway.

For all closed bottom rectangular structures with a clear span less than 12’, the ratio of the headwater to height during Q50 flow should be equal to or less than 1.25. In addition, the Q100 flow must pass through the structure without overtopping the roadway.

All open bottom structures and all bridges/culverts with spans 12’ and greater and less than 20’ shall have a minimum 1’ of clearance above Q50 flow. In addition, the Q100 flow must pass through the structure without overtopping the roadway.

All bridges/culverts with a clear span of 20’ or greater shall have a minimum 2’ clearance above Q50 flow. In addition, the Q100 flow must pass beneath the lowest chord of the structure.

BRIDGES/CULVERTS OVER CANALS
All structures over canals shall have a minimum of 1’ clearance above the design flow and the maximum flow must pass beneath the lowest chord of the structure.

2.3.3.1 NAVIGATIONAL
Bridges over navigable waters shall meet the vertical clearances required by the Coast Guard.

2.3.3.2 HIGHWAY VERTICAL
All new bridges are to be designed for 17'-0" of vertical clearance. This clearance may be reduced with prior approval from the Roadway Design Engineer, Maintenance Engineer, and Bridge Engineer, but is not to be less than 16'-0".

During construction, as much vertical clearance as possible is desirable, with 14'-9" being the minimum desirable. The minimum legal vertical clearance is 14'-0". Check with the Permits Unit in Headquarters for restrictions for each particular site.

2.3.3.3 HIGHWAY HORIZONTAL
The structure width is generally controlled by the geometry of the approaching roadway. The required roadway widths are established in the *Roadway Design Manual,* Appendix C. Refer to sketches on page A2.6.

A. Bridges
The curb-curb width of new bridges shall be as follows:

No Sidewalk	
Concrete Parapet:	roadway width + 3'-4"
Curb Mount Metal Rail:	roadway width + 4'-7½"
Sidewalk	
Raised Sidewalk:	roadway width
At-grade Sidewalk:	roadway width + 3'-4"

B. Culverts
The curb-curb width of new culverts shall be as follows:

Clear Zone Provided	
No Guardrail:	roadway width + clear zone
Clear Zone Not Provided	
W-Beam/Thrie Beam:	roadway width + 10'-0"
Concrete Porta-rail:	roadway width + 10'-8"
Concrete Parapet:	roadway width + 3'-4"

C. Construction
Horizontal clearances should be verified with the District Design Engineer, District Traffic Engineer, and the Bridge Engineer.

2.3.3.4 RAILROAD OVERPASS

For bridges carrying the railroad over the highway, the vertical clearances for highway crossings shall apply.
For bridges carrying highways over the railroad, the minimum vertical clearance shall be 23'-4" from the top of rail at a point directly over the centerline of track.

For exceptions, e.g. widening of existing structures, the railroad and PUC approvals should be obtained prior to final layout.

The UPRR and BNRR/SANTA FE standards on page A2.3 have not been approved by FHWA and are considered guidelines only. The dimensions shown as minimums on the standards should be considered as maximums for the purpose of determining span lengths. Exceptions to the standards that will reduce the bridge cost should be considered in the preliminary design stage of the project. Any exceptions to the standards must be approved by the railroad Chief Engineer and approval should be obtained prior to the final layout.

Some of the items where exceptions should be considered on a project-by-project basis are:

- Ditch width
- Location of the railroad pole line to eliminate or reduce the distance between the pier and the ditch slope.
- Elimination of splash boards
- Use of 1½:1 slopes on slope paving

Revisions:

April 2008	Added Coast Guard clearance requirement for bridges. Modified waterway clearance requirements with the concurrence of the Hydraulics Engineer. Added references to highway horizontal sketches in A2.6. Changes culvert width for concrete porta-rail to roadway width + 10'-8". Changed minimum railroad vertical clearance to 23'-4" to comply with BNSF/UPRR Guidelines for Railroad Grade Separation Projects, January 2007. Added references to railroad drawings in A2.3.

2.5.2.1.1 Materials

WEATHERING STEEL SELECTION CRITERIA
The purpose of the office standard is to provide guidelines to the engineer for the proper application of uncoated weathering steels and to make recommendations for maintenance to ensure continued performance of the steel. This criteria is taken from the FHWA Technical Advisory "Uncoated Weathering Steel in Structures", T5140.22, dated October 3, 1989.

DESIGN
Selection Criteria

1. Environmental considerations where weathering steel may not be used.
   - High rainfall, high humidity or persistent fog.
   - Industrial areas where chemical fumes may drift onto the structure. If the sulfur trioxide level is greater than 2.1 mg/100 cm^2/day average, weathering steel shall not be used. This would apply near fertilizer plants, such as the FMC Plant in Pocatello, Potlatch Plant in Lewiston, etc.
2. Geometrics and location
   - Grade Separations - The so-called "tunnel effect" is produced by the combination of narrow depressed roadway sections between vertical retaining walls, narrow shoulders, bridges with minimum vertical clearances and deep abutments adjacent to the shoulders as are found at many urban/suburban grade separations. These roadway/bridge geometrics combine to prevent roadway spray from being dissipated by air currents and can result in excessive salt in the spray being deposited on the bridge steel. Figure 1 shown below is representative of situations where use of uncoated weathering steel should be avoided where winter deicing salt use is significant.

   NOTE: There is no evidence of salt spray causing excessive corrosion in cases of narrow bridges with wing walls parallel to the over crossing road.

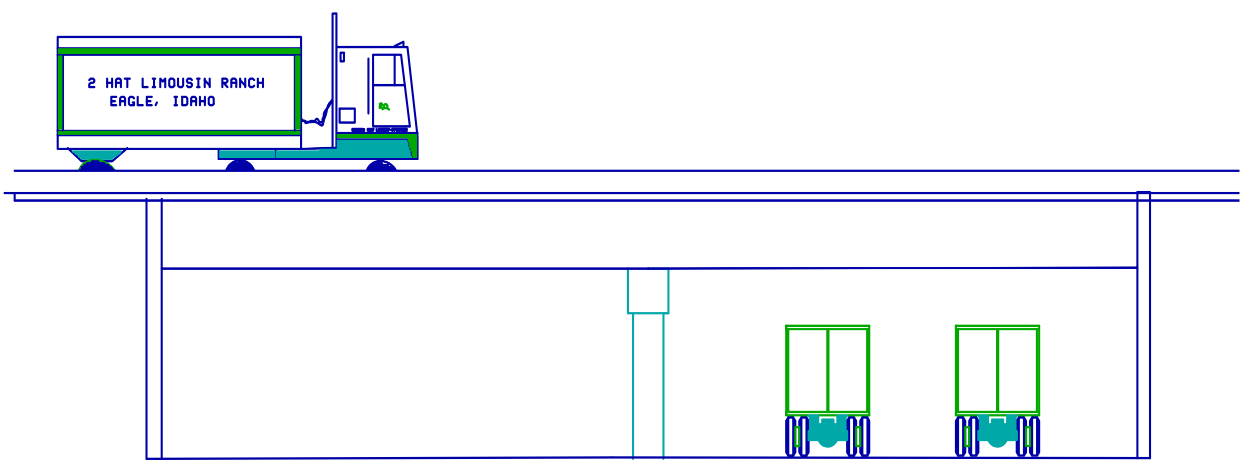


Figure 1 DEPRESSED ROADWAY TUNNEL EFFECT

   - Low Level Water Crossings - Sufficient clearance over bodies of water must be maintained so that spray or condensation or water vapor does not result in prolonged periods of wetness of the steel. Clearance to bottom flange of at least 10 feet over sheltered, stagnant water and at least 8 feet over running water is recommended.
3. Method of Selection
   - The Bridge Engineer, in cooperation with the Group Leaders, will make the selection. The selection will be based on stress, economics, environmental considerations, and aesthetics.
   - The selection will be made at the preliminary design stage.

<u>Design Details</u>

When weathering steel is used, the following design details should be utilized.

1. Controlling Roadway Drainage

   This is the first line of defense against localized corrosion by eliminating the exposure of the steel to contact with drainage from the roadway above, especially in areas where roadway salts are used.

   a. Joints:
      - To the extent possible, bridge joints should be eliminated. Virtually every bridge with joints has problems (corrosion, rideability, maintenance) attributable to the joint.
      - Extensive experience has shown that obtaining a permanent water-tight bridge joint is an elusive goal. Therefore, when joints are necessary, the assumption should be that the joints will leak and that drainage will contact the steel. Therefore, all steel within a minimum distance of 1 1/2 times the depth of the girder from the joint should be coated. In addition, Finger Joints shall not be used with weathering steel bridges unless given prior approval of the Bridge Engineer.
      - Drip bars on the top and bottom of the lower flanges can be effective in intercepting drainage and preventing it from running long distances along the flange and causing corrosion of the uncoated steel. However, welding of any attachment to the tension flange should be considered only after a thorough analysis of the impact of the attachment on fatigue life of the member.

   b. Drains
      - The spacing between drainage scuppers should be maximized in accordance with established hydrologic and hydraulic design. The FHWA Publication HEC 21, "Bridge Deck Drainage Guidelines", provides sound recommendations in this regard. Refer to Bridge Design Manual Article 2.6.6 for a design procedure. As scupper spacing increases, the volume of water required to pass through each scupper increases, thus creating velocities high enough to flush outlets clogged by deposits from low volume rainfalls.
      - Scupper downspouts should be designed and placed such that drainage will not contact the steel surface. However, details used to connect scuppers to drain pipes have often created more problems than they have prevented. Do not use flat runs of piping and elbows which clog or connections that separate. Careful detailing is critical.
      - Scupper drain pipes should not be routed through closed box sections where leakage inside of the box is possible, and may go undetected for long periods of time.

2. Other Features

   a. Water Traps
      - All details must be designed to provide natural drainage. Small copes in corners of plates or small drain holes are easily plugged, and should not be relied on to provide drainage.

   b. Box Sections
      - Box sections that are too small to provide for adequate visual inspection and access for maintenance personnel should be painted and weep holes to allow proper drainage and circulation of air should be provided.
      - Larger boxes should be detailed to minimize the entrance of water, debris and dirt that can promote corrosion. They must also provide for natural drainage of water that may enter and adequate access for inspection, cleaning and maintenance when necessary. Precautions should include:
        1. Locked covers or screens over access holes to prevent the entry of animals and birds or unauthorized personnel. Covers over manholes should be on hinges and provided with a lock to allow easy access by inspection personnel.
        2. Provision of positive drainage and adequate ventilation to minimize the wetting of the interior surfaces from water or condensation.
        3. Painting of the inside of the box members should be considered. Weep holes should also be provided to allow for proper drainage and air circulation.

   c. Concrete Surfaces

      After passing over uncoated weathering steel, drainage leaves dark, non-uniform and often unsightly stains on concrete surfaces. This problem can be mitigated, if desired, by using one or more of the following approaches:
      - Wrapping the piers and abutments during construction to minimize staining while the steel is open to rainfall.
      - Allowing/requiring the contractor to remove staining with a commercial solvent after completion of construction.

- Applying epoxy or some other material to coat and/or seal the concrete surfaces against staining.

d. Overlapping surfaces
If water is allowed to flow over overlapping joints, capillary action can draw the water into the joint and cause "rust-pack" to form. Therefore, the contact surfaces of overlapping joints must be protected from intrusion of rainfall and runoff. This applies to non-slip-critical bolted joints as well as to overlapped joints such as those in tapered high mast lighting poles. The faying surfaces should be painted or sealed to prevent the capillary penetration. In slip-critical bolted splices, "rust-pack" should not occur when the bolts are spaced as per AASHTO specifications.

e. Fascia Girders
There is no evidence that coating the entire fascia girder will add to the service life of an otherwise uncoated bridge. Therefore do not paint exterior girder except in the area of expansion joints.

MAINTENANCE
Effective inspection and maintenance programs are essential to ensure that all bridges reach their intended service life. This is especially true in the case of uncoated weathering steel bridges. The following maintenance actions should be done routinely:

<u>Inspection</u>
Implement inspection procedures that recognize the unique nature of uncoated weathering steel and the conditions resulting from excessive corrosion damage. Develop inspection guidelines that highlight the structural features to be inspected and also illustrate the difference between the desired oxide coating and excessive rust scaling. Measurements of the oxide coating thickness should be taken during each bridge inspection and recorded in the bridge inspection reports.

<u>Controlling Roadway Drainage</u> To the extent feasible the following should be done:

- Divert approach roadway drainage away from the bridge structure.
- Clean troughs of open (finger) joints and reseal "watertight" deck joints.
- Maintain deck drainage systems (scuppers, troughs, etc.) in order to divert deck drainage away from the superstructure steel and substructure units.
- Periodically clean and repaint all steel within a minimum distance of 1 1/2 times the depth of the girder from bridge joints.

<u>Other Maintenance</u>

- Remove dirt, debris and other deposits that hold moisture and maintain a wet surface condition on the steel. In some situations, hosing down a bridge to remove debris and contaminants may be practical and effective. Some agencies have a regularly scheduled program to hose down their bridges.
- Maintain screens over access holes in box sections to prevent entrance by animals and birds.
- Remove growth of nearby vegetation that prevents the natural drying of surfaces wet by rain, spray or other sources of moisture.

<u>Commentary</u>
FHWA Technical Advisory T5140.22 can be found at http://www.fhwa.dot.gov/legsregs/directives/techadvs/t514022.htm

HEC-21 can be found at http://www.fhwa.dot.gov/bridge/hec21.pdf

2.5.2.3 MAINTAINABILITY

Bearing Replacement

New bridges with replaceable bearings should have provisions shown on the plans for jacking the superstructure. Refer to Article 3.4.3 for jacking forces.

The minimum space for the jacks should be 10” in height and 11” in diameter.

An additional jacking stiffener in front of the bearings may be required on steel girder bridges. Concrete girder bridges should provide room to either jack on the girder in front of the bearings or jack on the diaphragm beside the bearings.

2.5.2.4 RIDEABILITY

Approach Slabs

Application

Approach slabs shall be used when the roadway approaches are concrete pavement or an integral abutment is used. When the approaches are asphalt pavement and the abutment is not an integral type, the designer shall determine the need for an approach slab with the concurrence of the Group Leader. If the approach slab is not constructed when the bridge is built then it will probably never be required. For this reason, do not provide a paving notch in the abutment or the dowel bars for an approach slab if one is not required for the original design.

Design Notes

The 12" thickness assumes a 10' unsupported span. This meets the requirements of AASHTO Article 2.5.2.6.3 for simple spans.

Dead loads include the weight of the approach slab, concrete parapets, and 2⅜" future wearing surface.

The design is based upon Strength 1 and Service 1.

Standard Drawing Notes

The Standard Drawing shall be modified to meet the details for each specific project. Only the details that apply should be shown. If any of the design parameters are exceeded, the design should be checked.

2.5.2.5 UTILITIES

UTILITIES INSTALLED WITH NEW CONSTRUCTION

General
Utility facilities should be allowed on structures in accordance with the ITD Utility Accommodations Policy. If utilities are allowed on the structure, the bridge designer shall determine the method of attachment and the location.

Bridge plans shall include all hardware specifications and details for the utilities installed during original construction. All materials and workmanship shall be in accordance with the current edition of the Idaho Transportation Department Standard Specifications for Highway Construction. Water, sewer, and gas lines shall only be installed during original construction.

Provisions for future telephone, power, and cable utility attachments shall include installation of sleeves from the roadway shoulder through the abutment backwall, intermediate diaphragms, and pier diaphragms. Threaded inserts shall also be provided in the bottom of the deck slab for future utility hangars.

Design Criteria
All utilities and utility supports shall be designed according to the AASHTO LRFD Bridge Design Specification to resist Strength and Extreme Event Limits States. This includes and not limited to dead load, expansion, surge, and earthquake forces.

Location
Utilities should be located, if possible, such that a rupture or support failure will not result in damage to the bridge, the surrounding area, or be a hazard to traffic. The utility installation shall be located so as to minimize the effect on the appearance of the structure and shall not affect bridge maintenance or bridge inspection activities. For new construction, the utility should be installed between girders. For attachment to an existing bridge, the utilities may be installed on the cantilevered portion of the deck. Utilities shall not be attached above the bridge deck nor attached to the railings or posts. Utilities and supports must not extend below the bottom of the superstructure. When appurtenances such as air release valves are required, care should be taken to provide adequate space.

Termination at Bridge Ends
Utility conduit and encasements shall extend beyond the end of the bridge/approach slab and shall extend to the roadway shoulder using acceptable bends or radius sweeps. Conduits and encasements installed for future use shall have the ends capped. The details shall be shown on the bridge plans and coordinated through the Utilities Engineer.

Expansion Joints
The utilities shall be designed with a suitable expansion system as required to prevent longitudinal forces from being transferred to bridge members.

Water mains generally remain a constant temperature and are anchored in the ground at the abutments. However, the bridge will move with temperature changes and seismic forces. Pipe support systems must be designed to allow for the bridge movements. For short bridges, this generally means the bridge will move and the utility will not since it is anchored at the abutments. For long bridges that require pipe expansion joints, design must carefully locate pipe expansion joints and the corresponding longitudinal load-carrying support.

Electrical conduits that use PVC should have an expansion device for every 100’ of pipe due to the higher coefficient of expansion. If more than two joints are specified, a cable or expansion limiting device is required to keep the ends from separating.

Pipelines for Fluids and Gases
The pipeline material shall be appropriate to handle the capacity and loads required, with due care taken to protect the structure and traffic. All pipelines carrying fluids and gases shall be encased the length of the structure in an acceptable encasement pipe and braced for lateral loading. The space between the carrying pipe and the encasement sleeve shall be effectively vented and drained beyond the structure at each end and at high points. Generally, a sleeve approximately 3” larger than the outside diameter of the encasement pipe shall be used to pass the utility through concrete diaphragms or abutments.

Pipelines transporting fluids and gases shall provide shut-off valves, either manual or automatic, at or near each end of the structure to provide a means of control in case of an emergency.

All pipe systems under pressure shall state the maximum operating and test pressure on the plans.

When exposed steel utility lines and supports are attached to a painted structure, they shall be painted in accordance with ITD Standard Specifications and the final coat shall match the bridge color. Any painted surfaces damaged during construction shall be cleaned and painted as directed by the Engineer. Any paint splatters shall be removed from the bridge.

Volatile Fluids
The utility company shall make provisions to electrically insulate pipelines carrying volatiles from its support.

Water and Sewer Lines
Water and Sewer lines should be steel pipe. Transverse support or bracing shall be provided for all water lines to carry Strength and Extreme Event Lateral Loading.

In box girders (closed cell), a rupture of a water line will generally flood a cell before emergency response can shut down the water main. This will be designed for an Extreme Event II load case, where the weight of water is a dead load (DC). Additional weep holes or open grating should be considered to offset this Extreme Event.

Water or sewer lines to be placed lower than adjacent bridge footings shall be encased if failure can cause undermining of the footing.

Telephone, Power, and Cable Conduit
All conduits shall be rigid. Conduits shall be galvanized steel pipe, Schedule 40 (minimum) PVC pipe of a UL approved type, or an approved equal. All metal fittings shall be galvanized in accordance with AASHTO M-111 or M-232 (ASTM A-123 or A-153 respectively). PVC pipe may be used with suitable consideration for deflection, placement of expansion fittings, and of freezing water within the conduits.

Utility Supports
Where such conduit is buried in concrete curbs or barriers or has continuous support, such support is considered to be adequate. Where hangers or brackets support conduit at intervals, the distance between supports shall be small enough to avoid excessive sag between supports. Utility supports shall be designed so that any loads imposed by the utility installation do not overstress the conduit, supports, bridge structure, or bridge members.

Designs shall provide longitudinal and transverse support for loads from gravity, earthquakes, temperature, inertia, etc. It is especially important to provide transverse and longitudinal support for single rod and other similar inserts that cannot resist moment.

Attachment of conduits or brackets to non-prestressed concrete members shall be with epoxy bond anchors. There should be a minimum of 3” edge distance to the centerline of bolt holes in concrete. Drilling into prestressed concrete members for utility attachments shall not be allowed. All welding must be approved and welding across main members will not be permitted. All steel in utility supports, including fastenings and anchorages, shall be galvanized in accordance with AASHTO M-111 or M-232 (ASTM A-123 or A-153 respectively).

The following types of supports are generally used for various utilities. Selection of a particular support type should be based on the needs of the installation and the best economy.
<u>Concrete Embedment</u>
This is the best structural support condition and offers maximum protection to the utility. Its cost may be high for larger conduit and the conduit cannot be replaced.

<u>Pipe Hangers</u>
Utility lines should be suspended by means of cast-in-place anchors, whenever possible. This is the most common type of support for utilities to be hung under the bridge deck. This allows the use of standard cast-in-place inserts and is very

flexible in terms of expansion requirements. Refer to Bridge LRFD Manual pages B2.4A and B2.4B for suggested details. To avoid complete failure of the utility hanger system by failure of one hanger for heavy pipes over traffic (10” water main or larger), the Extreme Limit State should be used assuming one hangar has failed. Vertical Anvil Insert, Figure 282 or similar inserts, will not provide resistance to longitudinal forces. Transverse supports must be provided by a second hanger extending from a girder or by a brace against the girder. Where PVC conduit is to be supported by hangers or pedestals at intervals, the distance between supports shall be small enough to avoid excessive sag of the conduit. Refer to Article A2.5 for recommended support spacing for PVC pipe.

UTILITIES INSTALLED ON EXISTING BRIDGES

General

Requests to install utilities on existing structures should follow the procedures outlined in the ITD Guide for Utility Management. Utilities shall not be allowed to attach to a highway structures until approved by ITD. Attachment details should be shown on the existing bridge plan sheets that can be obtained from the ITD Bridge Section. Design for utilities attached to existing structures should follow the same requirement as utilities installed with new construction. Any existing utilities on the same side of the structure as the proposed utility should be shown on the plans. The utility company shall be responsible for calculating design stresses in the utility and design of the support system. All calculations shall be on 8½”x11” paper and stamped by an engineer licensed in Idaho. Plans shall be either 11”x17” or 22”x34” sheets and stamped by an engineer licensed in Idaho.

Pipelines for Fluids and Gases

Bridges are not designed to carry the extra weight of future pipelines for fluids and gases. Adding these pipelines to an existing bridge will normally produce an overstress in the members. Design calculations are required to verify that no overstress occurs in the members and the live load carrying capacity of the bridge is not reduced.

Revisions:

June 2006 Added new article.

2.5.2.6.2 CRITERIA FOR DEFLECTION

Structures shall be designed according to the provisions of this article.

2.5.2.6.3 CRITERIA FOR SPAN -TO-DEPTH RATIOS

Flexural members of bridge structures shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect the strength or serviceability of the structure at service load plus impact.

The minimum span to depth ratios are recommended unless computation of deflection indicates that lesser depths may be used without adverse effects.

The span length of members that are not built integrally with their supports shall be considered the clear span plus the depth of the member but need not exceed the distance between centers of supports.

The span length of continuous members shall be considered at the faces of support. When fillets making an angle of 45 degrees or more with the axis of the member are built monolithic with the member and support, the face of support shall be considered at a section where the combined depth of the member and fillet is at least 1.5 times the thickness of the member.

2.5.3 CONSTRUCTIBILITY

All provisions of this article shall apply. Refer also to Article 6.10.3.2 for Steel Structures.

2.6.6 ROADWAY DRAINAGE

I. BASIC CONCEPTS
In the design of bridge drainage facilities, the aim is to keep the traveled way free from hazardous flooding except infrequent, excessively heavy rainstorms. Roadway drainage inlets must be provided to remove water upstream of the structure.

To remove the storm water from the bridge deck, the standard drain inlet shall be used. The deck is used to carry the storm water to these drain inlets. The drains are placed according to hydraulic principles and located next to the curbing. The collected water should be dropped over the median or slope paving. Flexible sealers are used at expansion joints to prevent undesirable leakage through the deck. As a minimum, place one Type I Deck Drain upstream (approximately 10 ft), of any expansion joint to reduce leakage onto the substructure. The intent is to remove the gutter flow ahead of the expansion joint except for larger storms that will infrequently splash over the drain and wet the expansion joint.

The preferred drain is the Type 1. Use the type 2 drain when a larger opening is required by the design procedure. The Type 3 drain should only be used with voided slabs, box beams, and deck bulb tee type structures.

For further reference, see Hydraulic Engineering Circular No. 21, May 1993, “Design of Bridge Deck Drainage.”

II. DECK DRAIN LOCATION

1. Avoid locations over Highway or Railroad travel way as piping will be required.

2. Locate over medians, water or slope paving. If over a median, provide a suitable splash pad or downspout to eliminate unsightly erosion.

3. Low points. Usually these will include at least one and frequently all four corners of the bridge deck. In each case the drain shall be placed as close to the corner of the deck as practicable.

4. In the case of bridges where the approach grade is downward toward the bridge, removal of runoff must be handled in the roadway plans. Runoff exiting the structure must be handled similarly. Assurances to this effect should be obtained from the appropriate area engineer. If received verbally, record it in writing and file it in the subject files. Note on **Situation & Layout** sheet for roadway plans to pick up runoff and the volume of runoff involved.

5. Flat areas will require further investigation. Check if bridge can be sloped.

6. Trash, gravel, and ice are the main consideration in designing drainage piping systems.

III. DESIGN PROCEDURE
Refer to Bridge Design Manual Appendix 2.1

Commentary

HEC-21 can be found at http://www.fhwa.dot.gov/bridge/hec21.pdf

Refer to NCHRP 67 for designing drainage piping systems.

Bridge Deck Drain Design Procedure

DEFINITION OF SYMBOLS

A = contributing drainage area (acres)

C = Runoff coefficient (ratio of impervious to pervious drainage area). Use 0.9

d = water film depth Use 0.006 ft.

E = inlet interception efficiency

$$E = R_f * E_0$$

E_o = Ratio of frontal flow to total gutter flow

$$E_0 = 1 - \left[1 - \frac{W_d}{T}\right]^{2.67}$$

G1 = Incoming tangent grade on a vertical curve. Ft/ft

G2 = Outging tangent grade on a vertical curve. Ft/ft

i = design rainfall intensity (in/hr) Use the value from the Rational Method and compare to the Avoidance of Hydroplaning and Driver Vision Impairment.

Rational Method

1. Select the rainfall intensity zone from the map.
2. Make a trial selection of duration and compute the trial intensity.

$$t_c = t_0 + t_g \qquad t_0 = 0.93 * \frac{(W_p * n)^{0.6}}{(C * i)^{0.4} * S^{0.3}} \qquad t_g = 484 * \frac{S_x * T^2}{C * i * W_p}$$

3. Using the trial intensity value, compute the total time of concentration
4. The design intensity is the value when the trial duration equals the calculated total time of concentration.

Avoidance of Hydroplaning

$$i = \left[\frac{64904.4}{C * n}\right]\left[\frac{S_x}{(S_x^2 + S^2)^{0.25}}\right]\left[\frac{d^{1.67}}{(W_p - T)}\right] \text{ (in/hr.)}$$

Driver Vision Impairment

i = 4.0 in/hr

IDF = intensity-duration-frequency curve. See Figure 1 for the Rainfall Intensity Zone. The inlet design is based upon a 25-year frequency. The intensity for a given duration can be computed from the following equations:

Zone 1	duration < 60 minutes	$i = 10^{0.894094 - 0.55912 * \log(duration)}$
	duration > 60 minutes	$i = 10^{1.132115 - 0.69398 * \log(duration)}$
Zone 2	duration < 60 minutes	$i = 10^{1.132076 - 0.60121 * \log(duration)}$
	duration > 60 minutes	$i = 10^{1.375147 - 0.75708 * \log(duration)}$

K = inlet interception coefficient on vertical curve bridges

$$K = 1 - (1 - E) * \left(\frac{S_u}{S}\right)^{0.5}$$

L_c = distance between deck drains after the first inlet (ft)

Drains on tangent grade

$$L_c = \frac{43560 * Q_f * E}{C * i * W_p}$$

Drains on flat grade

$$L_c = \frac{1312 * S_x^{1.44} * T^{2.11}}{(n * C * i * W_p)^{0.67}}$$

L_0 = distance to first deck drain (ft)

$$L_0 = \frac{43560 * Q_f}{C * i * W_p}$$

L_D = length of deck drain parallel to gutter flow (ft)

Type 1 Drain = 0.78 ft.
Type 2 Drain = 0.79 ft.

L_{vc} = length of vertical curve (ft)

n = Manning's friction coefficient (0.016)

P = total inlet perimeter for flat grade bridges (ft)

$$P = \frac{(C * i * W_p)^{0.33} * T^{0.61}}{102.5 * S_x^{0.06} * n^{0.67}}$$

Q_f = full gutter flow at design spread (cfs)

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67}$$

Q_R = flow calculated with rational formula (cfs)

$$Q_R = C * i * A$$

R_f = frontal flow interception efficiency

$$R_f = 1 - 0.09 * (V - V_0)$$

S = grade of bridge deck at a given drain (ft/ft)

Drains on vertical curve

$$S = G1 + \frac{(G2 - G1) * X}{L_{vc}}$$

S_u = longitudinal grade for upstream inlet on vertical curve bridge

S_x = cross slope of deck (ft/ft)

T = design spread (ft) The design spread is calculated assuming that the gutter flow can spread across the full shoulder width on bridges with not more than 2 traffic lanes. For bridges with more than 2 traffic lanes, the gutter flow can spread across the full shoulder width and half of the adjacent traffic lane width.

V = gutter flow velocity (ft/sec)

$$V = \frac{1.12}{n} * S_x^{0.67} * S^{0.5} * T^{0.67}$$

V_o = Velocity of flow at which splash-over first occurs over a grate (ft/s.)

Type 1 Drain

$$V_0 = 10^{0.50299 * \log(L_d) + 0.759359}$$

Type 2 Drain

$$V_0 = 10^{0.654078 * \log(L_d) + 0.474699}$$

W_p = width of pavement contributing to runoff (ft) equals the width of deck from the face of curb to high point of crown.

W_D = width of deck drain at right angle to gutter flow (ft)

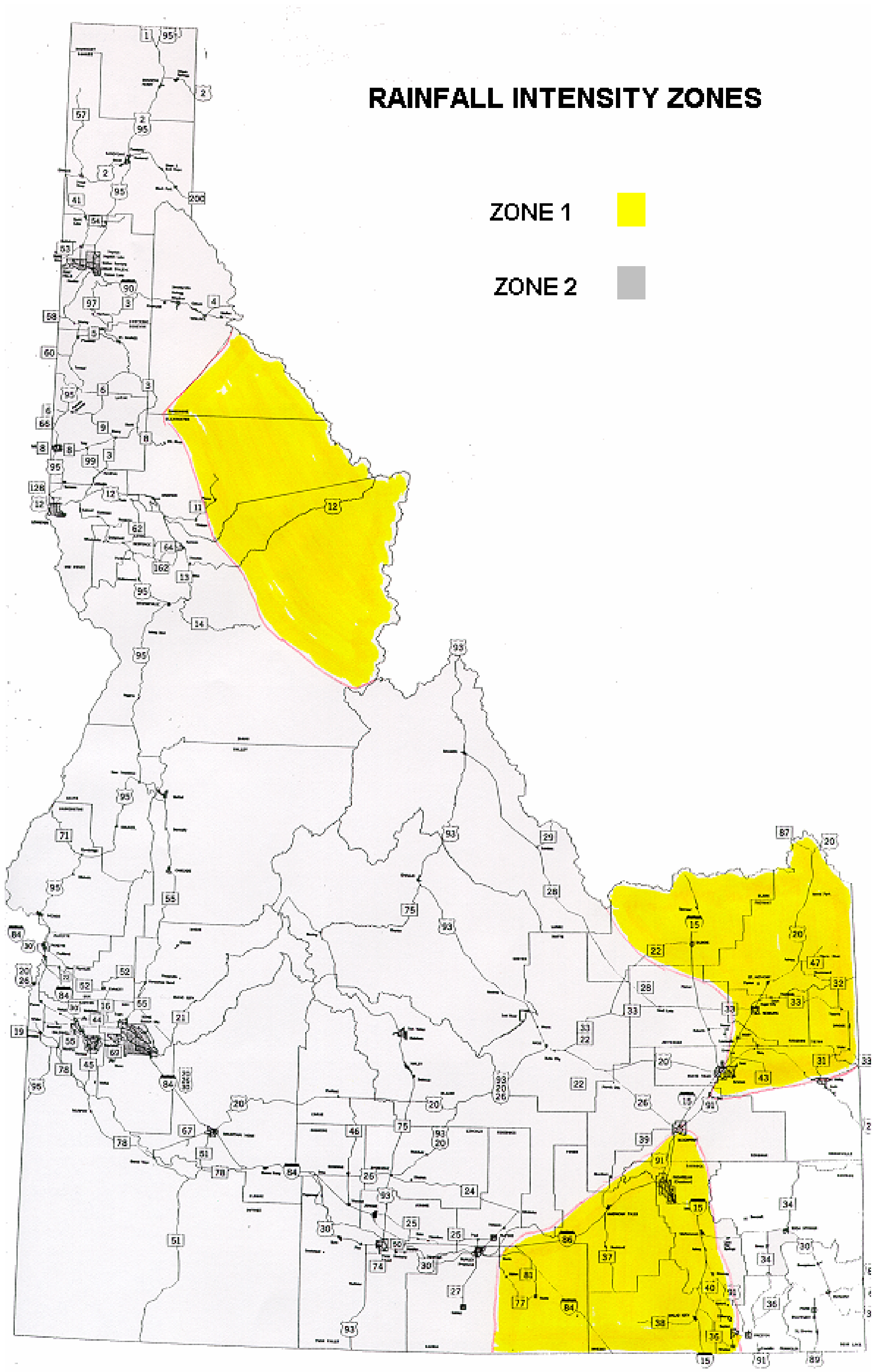
Type 1 Drain = 0.448 ft
Type 2 Drain = varies

X = distance from the PC of a vertical curve to the inlet (ft)

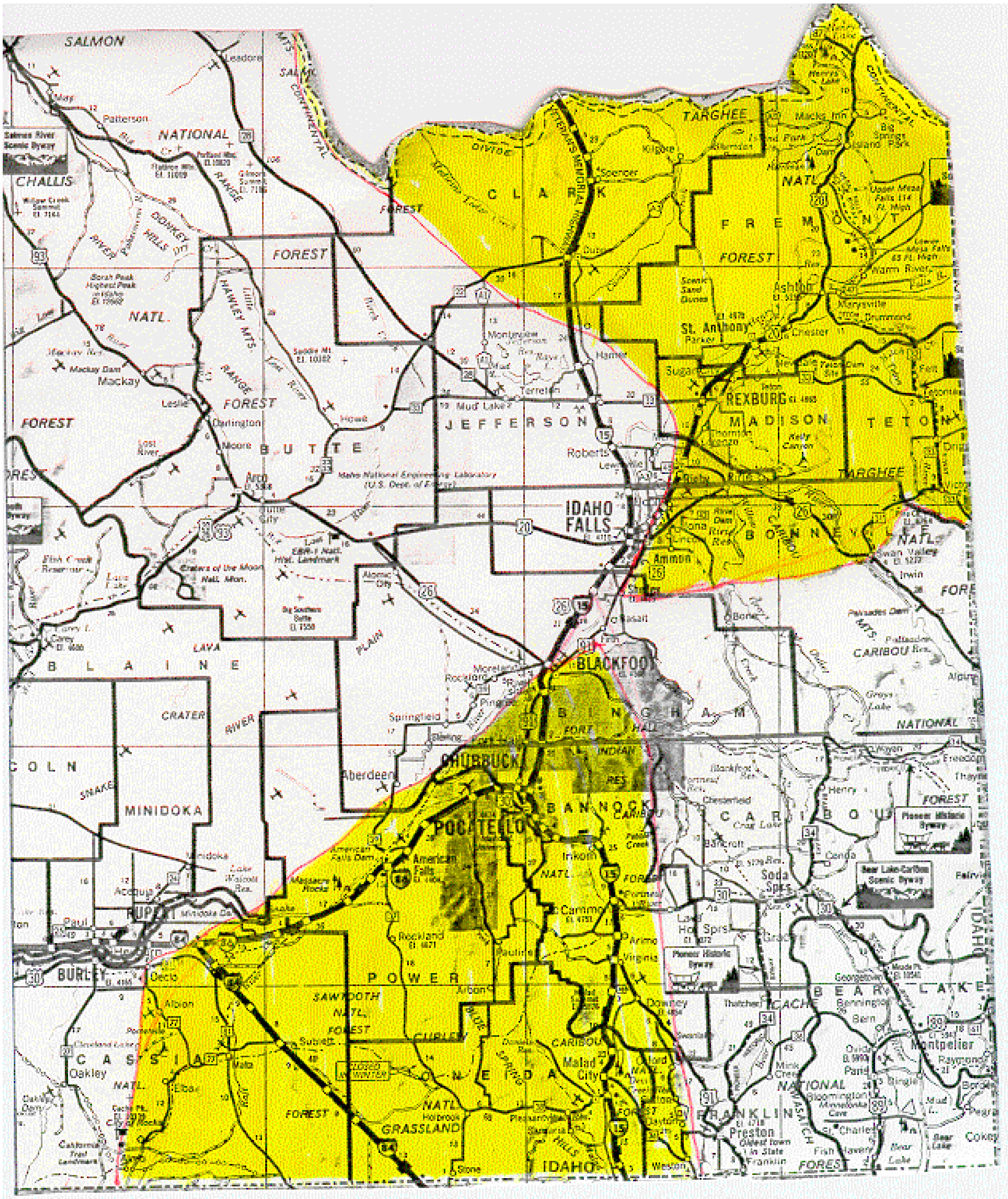
X_T = distance from PC of a vertical curve to the turning point (ft)

$$X_t = \frac{-G1 * L}{(G2 - G1)}$$

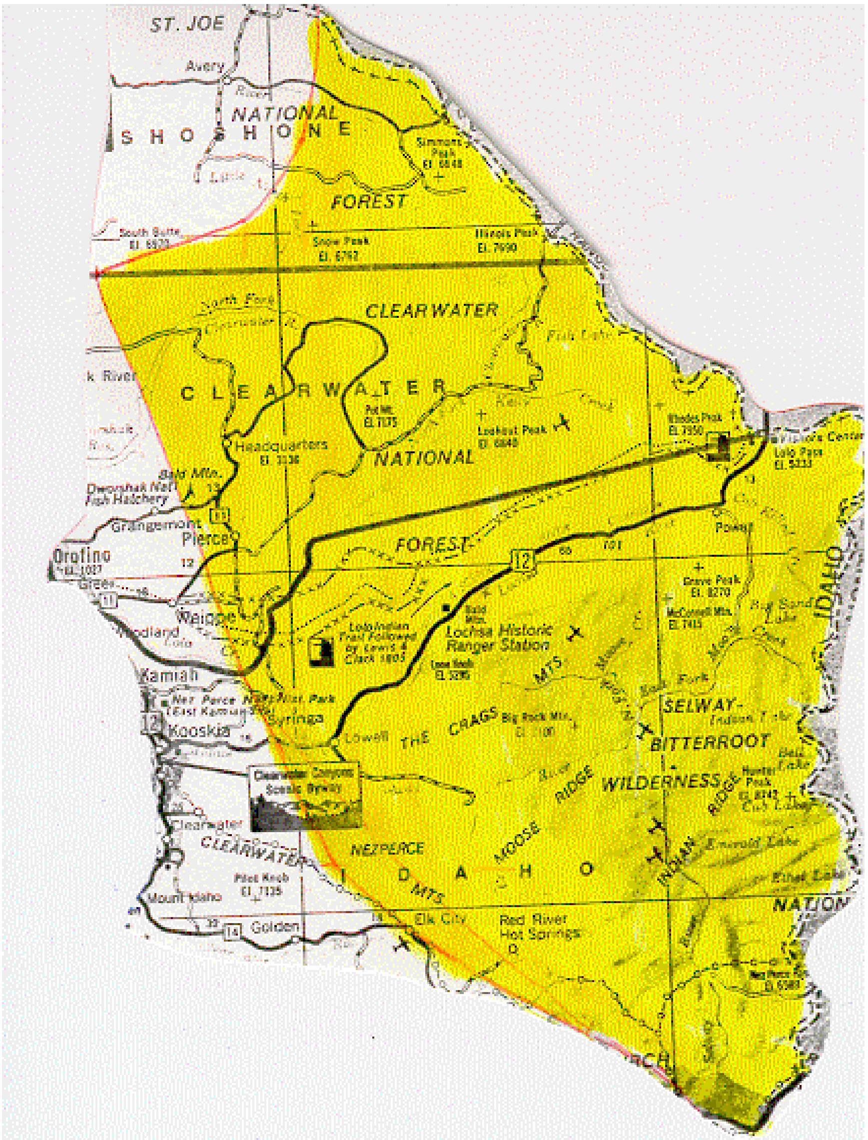
Figure 1



ZONE 2 – INSET 1



ZONE 2 – INSET 2



DESIGN PROCEDURE FOR CONSTANT GRADE BRIDGES

If the bridge grade is less than 0.3 percent, then the procedure for flat bridges should be followed.

1. Determine the rainfall intensity.
    - Calculate the intensity using the Rational Method
    - Calculate the intensity using the Avoidance of Hydroplaning Method
    - Intensity due to Vision Impairment is 4.00 in/hr.
    - Use the Rational Method value for the design intensity
    - Compare the Rational Method value to the Avoidance of Hydroplaning and Vision Impairment values
2. Calculate the full gutter flow, Q_f, at the design spread, T.
3. Calculate the distance to the first inlet, L_0, from the upslope end of the bridge.
4. Determine if inlets are needed on the bridge.
    - If L_0 is greater than the total bridge length, no inlets are required.
    - If L_0 is less than the total bridge length, calculate the spacing between inlets, L_c.
5. Space inlets at L_c until the end of the bridge is reached.
6. Design the bridge end treatment.

    Inlets beyond the end of the bridge should be sized for a flow. Energy dissipation devices should be provided, if necessary, at the toe to prevent erosive velocities.
    - Where debris is a problem, assume a percentage of blockage of the drains.
    - Assume a duration time and design spread width.
    - Calculate the intensity using the Rational Method.
    - Compute t_c and compare to the assumed duration time. Iterate until the assumed and computed values are equal.
    - Compute the design flow.
    - Compute the spread.
    - Iterate until the assumed duration and spread values are equal to the computed values.

DESIGN PROCEDURE FOR FLAT GRADE BRIDGES

Bridges with vertical curves are nearly flat at their low or high points. For bridges with grades less than 0.3 percent at the high or low points of vertical curves or on a constant grade, check the required inlet spacing assuming the bridge is flat. Use the flat spacing if it is less than the spacing required for a constant grade.

1. Determine the rainfall intensity using the Rational Method.
    - Use a duration of 5 minutes
    - Determine the Rainfall Intensity Zone for the bridge site and compute the intensity
2. Calculate the constant inlet spacing, L_c.
3. Determine if inlets are needed on the bridge.
    - If L_c is greater than the total bridge length, no inlets are required.
4. If inlets are needed, calculate the required inlet perimeter, P.
5. Design the bridge end treatment.

DESIGN PROCEDURE FOR VERTICAL CURVE BRIDGES

1. Compute the distance from the turning point of the vertical curve to each end of the bridge.
2. Determine the rainfall intensity using the Rational Method.
    - Assume a duration and compute the intensity.
    - Compute the time of concentration, t_c, by summing the gutter and overland flow times, t_0 and t_g.
    - Iterate until t_c equals the assumed duration.
3. Determine the spacing to the first inlet.
    - Assume a distance from the turning point to the first inlet and compute the slope at that point.
    - Compute the gutter flow, Q_f., at the design spread.
    - Compute the distance to the first inlet, L_0, letting K=1.
    - Iterate the process until the trial and computed values are equal.
    - If L_0 is greater than the distance from the turning point to the end of bridge, than inlets are not required.
4. Determine the spacing to the next inlet, if required.
    - Select a trial spacing
    - Compute the local slope, S.
    - Calculate the gutter flow, Qf.

- Compute the inlet efficiency, E.
- Compute the interception coefficient, K.
- Compute the inlet spacing and compare to the trial value.
- Iterate the process until the trial and computed values are equal.

5. Repeat step 4 until the sum of the inlet spacing is greater than the distance from the turning point to the end of bridge.
6. Repeat steps 3, 4, & 5 for the drains on the other side of the turning point.
7. Design bridge end treatment.

EXAMPLE 1 - TANGENT GRADE BRIDGE WITH NO INLETS REQUIRED

EXCEL or MATHCAD programs can be used to complete the design process. *Drains on tangent grade(.xls or .mcd)* is on the X: drive in the LRFD Design Aids, Section 2 folder.

Given:	500 ft bridge	S = 0.03	S_x = 0.02
	W_p = 18 ft	T = 10 ft	n = 0.016
	C = 0.9	Zone 2	d = 0.006 ft

Step 1. Calculate the Design Rainfall Intensity

a. Rational Method

Assume duration of 44 minutes

$$i = 10^{1.132076 - 0.60121*\log(duration)} = 10^{1.132076 - 0.60121*\log(44)} = 1.39\ in/hr$$

$$t_0 = 0.93*\frac{(W_p*n)^{0.6}}{(C*i)^{0.4}*S^{0.3}} = 0.93*\frac{(18*0.016)^{0.6}}{(0.9*1.39)^{0.4}*0.03^{0.3}} = 1.15\ \text{min}utes$$

$$t_g = 484*\frac{S_x*T^2}{C*i*W_p} = 484*\frac{0.02*10^2}{0.9*1.39*18} = 42.95\ \text{min}$$

$$t_c = t_0 + t_g = 1.15 + 42.95 = 44.10\ \text{min}$$

The assumed time of 44 minutes is approx. equal to the computed time of 44.10 minutes. Therefore, use i = 1.39 in/hr.

b. Avoidance of Hydroplaning

$$i = \left[\frac{64904.4}{C*n}\right]\left[\frac{S_x}{(S_x^2+S^2)^{0.25}}\right]\left[\frac{d^{1.67}}{(W_p - T)}\right] = \frac{64904.4}{0.9*0.016}*\frac{0.02}{(0.02^2+0.03^2)^{0.25}}*\frac{0.006^{1.67}}{(18-10)} = 11.56\ in/hr$$

c. Avoidance of Vision Impairment

i = 4.00 in/hr

d. Design rainfall intensity = 1.39 in/hr

Step 2. Compute full gutter flow based on the design spread of 10'.

$$Q_f = \frac{0.56}{n}*S_x^{1.67}*S^{0.5}*T^{2.67} = \frac{0.56}{0.016}*0.02^{1.67}*0.03^{0.5}*10^{2.67} = 4.12\ cfs$$

Step 3. Starting at the upslope end of the bridge, compute the distance to the first inlet.

$$L_0 = \frac{43560*Q_f}{C*i*W_p} = \frac{43560*4.12}{0.9*1.39*18} = 7969.94\ ft$$

Step 4. Since L_0 is greater than the total bridge length (500 ft), no inlets are required.

Step 5. Design the bridge end treatment.

- Since no drains are required, the assumed blockage = 0%
- Assumed duration = 4.5 minutes Assumed spread = 6 feet

$$i = 10^{1.132076 - 0.60121*\log(duration)} = 10^{1.132076 - 0.60121*\log(4.5)} = 5.49\ in/hr$$

$$t_0 = 0.93*\frac{(W_p*n)^{0.6}}{(C*i)^{0.4}*S^{0.3}} = 0.93*\frac{(18*0.016)^{0.6}}{(0.9*5.49)^{0.4}*0.03^{0.3}} = 0.67\ \text{min}$$

$$t_g = 484*\frac{S_x*T^2}{C*i*W_p} = 484*\frac{0.02*6^2}{0.9*5.49*18} = 3.92\ \text{min}$$

$$t_c = t_0 + t_g = 0.67 + 3.92 = 4.59\ \text{min}$$

4.59 minutes are approx. equal to the assumed value of 4.5 minutes.

- Compute the design flow with the percent blockage

$$Q = C*i*A - [(blockage)*E*(number\ of\ drains)*V]$$

$$A = \frac{W_p*(length\ of\ bridge)}{43560} = \frac{18*500}{43560} = 0.207\ acres$$

$$Q = 0.9*5.49*0.207 - [0] = 1.02\ cfs$$

- Check the design spread with the new computed flow. Compute the spread by solving the gutter flow equation for T.

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67}$$

$$T = \left(\frac{Q_f * n}{0.56 * S_X^{1.67} * S^{0.5}} \right)^{\frac{1}{2.67}} = \left(\frac{1.02 * 0.016}{0.56 * 0.02^{1.67} * 0.03^{0.5}} \right)^{\frac{1}{2.67}} = 5.93 \ ft$$

This is approximately equal to the assumed value of 6 feet.

- Select an inlet that will handle 1.02 cfs and provide a pipe or paved ditch to convey the design flow from the drain to the toe of the embankment. Provide energy dissipation, if necessary, at the toe to achieve nonerosive velocities.

EXAMPLE 2 - TANGENT GRADE BRIDGE WITH INLETS REQUIRED

EXCEL or MATHCAD programs can be used to complete the design process. *Drains on tangent grade(.xls or .mcd)* is on the X: drive in the LRFD Design Aids, Section 2 folder.

Given:	2000 ft bridge	S = 0.01	$S_x = 0.02$
	$W_p = 34$ ft	T = 10 ft	n = 0.016
	C = 0.9	Zone 2	d = 0.006 ft
	$W_D = 1.0$ ft	$L_D = 1.50$ ft	

Step 1. Calculate the Design Rainfall Intensity

- Rational Method

  Assume duration of 12.25 minutes

$$i = 10^{1.132076 - 0.60121*\log(duration)} = 10^{1.132076 - 0.60121*\log(12.25)} = 3.00\ in/hr$$

$$t_0 = 0.93*\frac{(W_p*n)^{0.6}}{(C*i)^{0.4}*S^{0.3}} = 0.93*\frac{(34*0.016)^{0.6}}{(0.9*3.00)^{0.4}*0.01^{0.3}} = 1.73\ \text{min}\,utes$$

$$t_g = 484*\frac{S_x*T^2}{C*i*W_p} = 484*\frac{0.02*10^2}{0.9*3.*34} = 10.53\ \text{min}$$

$$t_c = t_0 + t_g = 1.73 + 10.53 = 12.25\ \text{min}$$

  The assumed time of 12.25 minutes is equal to the computed time of 12.25 minutes.
  Therefore, use i = 3.00 in/hr.

- Avoidance of Hydroplaning

$$i = \left[\frac{64904.4}{C*n}\right]\left[\frac{S_x}{(S_x^2+S^2)^{0.25}}\right]\left[\frac{d^{1.67}}{(W_p - T)}\right] = \frac{64904.4}{0.9*0.016}*\frac{0.02}{(0.02^2+0.01^2)^{0.25}}*\frac{0.006^{1.67}}{(34-10)} = 4.89\ in/hr$$

- Avoidance of Vision Impairment

  i = 4.00 in/hr
- Design rainfall intensity = 3.00 in/hr

Step 2. Compute full gutter flow based on the design spread of 10'.

$$Q_f = \frac{0.56}{n}*S_x^{1.67}*S^{0.5}*T^{2.67} = \frac{0.56}{0.016}*0.02^{1.67}*0.01^{0.5}*10^{2.67} = 2.38\ cfs$$

Step 3. Starting at the upslope end of the bridge, compute the distance to the first inlet.

$$L_0 = \frac{43560*Q_f}{C*i*W_p} = \frac{43560*2.38}{0.9*3.00*34} = 1127.92\ ft$$

Step 4. Since L_0 is less than the total bridge length (2000 ft), inlets are required.

Step 5. Space inlets at L_c until the end of the bridge is reached.

- Compute the frontal flow ratio, E_0.

$$E_0 = 1 - \left[1 - \frac{W_d}{T}\right]^{2.67} = 1 - \left(1 - \frac{1.0}{10}\right)^{2.67} = 0.245$$

- Compute the gutter velocity.

$$V = \frac{1.12}{n}*S_x^{0.67}*S^{0.5}*T^{0.67} = \frac{1.12}{0.016}*0.02^{0.67}*0.01^{0.5}*10^{0.67} = 2.38\ fps$$

- Compute the splash-over velocity.

$$V_0 = 10^{0.654078*\log(L_d)+0.474699} = 10^{0.654078*\log(1.50)+0.474699} = 3.89\ fps$$

- Compute the frontal flow intercept efficiency $R_f = 1 - 0.09*(V - V_0) = 1 - 0.09*(2.38 - 3.89) = 1.136$

  Maximum $R_f = 1.0$
- Compute the inlet efficiency $E = R_f * E_0 = 1.0*0.245 = 0.245$
- Compute the constant spacing between the remainder of the inlets.

$$L_c = \frac{43560*Q_f*E}{C*i*W_p} = \frac{43560*2.38*0.245}{0.9*3.00*34} = 276.57\ ft$$

- Compute the number of drains per side

$$number = \frac{(bridge\ length) - L_0}{L_c} = \frac{2000 - 1129.33}{276.57} = 3\ drains$$

Step 6. Design the bridge end treatment.

- Since drains are required, assume blockage = 50%
- Assumed duration = 15.5 minutes   Assumed spread = 10.6 feet

$$i = 10^{1.132076 - 0.60121 * \log(duration)} = 10^{1.132076 - 0.60121 * \log(15.5)} = 2.61\ in/hr$$

$$t_0 = 0.93 * \frac{(W_p * n)^{0.6}}{(C * i)^{0.4} * S^{0.3}} = 0.93 * \frac{(34 * 0.016)^{0.6}}{(0.9 * 2.61)^{0.4} * 0.01^{0.3}} = 1.83 \min$$

$$t_g = 484 * \frac{S_x * T^2}{C * i * W_p} = 484 * \frac{0.02 * 10.6^2}{0.9 * 2.61 * 34} = 13.62 \min$$

$$t_c = t_0 + t_g = 1.83 + 13.62 = 15.45 \min$$

15.45 minutes are approx. equal to the assumed value of 15.5 minutes.

- Compute the design flow with the percent blockage

$$Q = C * i * A - [(blockage) * E * (number\ of\ drains) * V]$$

$$A = \frac{W_p * (length\ of\ bridge)}{43560} = \frac{34 * 2000}{43560} = 1.56\ acres$$

$$Q = 0.9 * 3.21 * 1.56 - [0.50 * 0.245 * 3 * 2.38] = 2.79\ cfs$$

Check the design spread with the new computed flow. Compute the spread by solving the gutter flow equation for T.

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67}$$

$$T = \left(\frac{Q_f * n}{0.56 * S_X^{1.67} * S^{0.5}} \right)^{\frac{1}{2.67}} = \left(\frac{2.79 * 0.016}{0.56 * 0.02^{1.67} * 0.01^{0.5}} \right)^{\frac{1}{2.67}} = 10.61\ ft$$

This is equal to the assumed value of 10.6 feet.

- Select an inlet that will handle 2.79 cfs and provide a pipe or paved ditch to convey the design flow from the drain to the toe of the embankment. Provide energy dissipation, if necessary, at the toe to achieve nonerosive velocities.

EXAMPLE 3 – FLAT GRADE BRIDGE

EXCEL or MATHCAD programs can be used to complete the design process. *Drains on flat grade(.xls or .mcd)* is on the X: drive in the LRFD Design Aids, Section 2 folder.

Given:	4000 ft bridge	T = 10 ft	$S_x = 0.02$
	S = 0	n = 0.016	C = 0.9
	$W_p = 34$ ft	Rainfall Zone = 2	$t_c = 5$ minutes

Step 1. Determine the rainfall intensity.

- Rational Method

$$i = 10^{1.132076-0.60121*\log(duration)} = 10^{1.132076-0.60121*\log(5)} = 5.15\,in/hr$$

- Avoidance of Hydroplaning

$$i = \left[\frac{64904.4}{C*n}\right]\left[\frac{S_x}{\left(S_x^2+S^2\right)^{0.25}}\right]\left[\frac{d^{1.67}}{\left(W_p - T\right)}\right] = \frac{64904.4}{0.9*0.016} * \frac{0.02}{\left(0.02^2+0\right)^{0.25}} * \frac{0.006^{1.67}}{(34-10)} = 5.17\ in/hr$$

- Driver Vision Impairment   i = 4.00 in/hr
- Design intensity = 5.15 in/hr

Step 2. Compute the inlet spacing

$$L_c = \frac{1312*S_x^{1.44}*T^{2.11}}{\left(n*C*i*W_p\right)^{0.67}} = \frac{1312*0.02^{1.44}*10^{2.11}}{(0.016*0.9*5.15*34)^{0.67}} = 325.32\ ft$$

Step 3. Since L_c is less than the bridge length (4000 ft), inlets are needed.

Step 4. Compute the total inlet perimeter

$$P = \frac{\left(C*i*W_p\right)^{0.33}*T^{0.61}}{102.5*S_x^{0.06}*n^{0.67}} = \frac{(0.9*5.15*34)^{0.33}*10^{0.61}}{102.5*0.02^{0.06}*0.016^{0.67}} = 4.26\ ft$$

If the inlet is adjacent to the curb, then the sum of the other 3 sides should equal the computed inlet perimeter. A curb opening should equal the computed inlet perimeter.

Solutions.

- Number of inlets required on each side of the deck.

$$Number\ of\ drains = \frac{L_b - L_c}{L_c} = 11.3 \qquad \text{Use } N_D = 12$$

$$Drain\ Spacing = \frac{L_b}{N_D + 1} = 307.69\ ft$$

- Type 2 Drain
  12 drains spaced at 307.69 feet along the length of bridge.

$$Width\ of\ drain = \frac{4.26-0.79}{2} = 1.74\ ft$$

- Type 1 Drain
  12 locations spaced at 307.69 feet along the length of bridge.
  $P_1 = 2*0.448 + 0.78 = 1.676$
  Number of drains required at each location = 4.26 / 1.676 = 2.6   Use 3

EXAMPLE 4 – VERTICAL CURVE BRIDGE

EXCEL or MATHCAD programs can be used to complete the design process. *Drains on vertical curve(.xls or .mcd)* is on the X: drive in the LRFD Design Aids, Section 2 folder.

Given:	3000 ft bridge	$S_x = 0.02$	T = 10 ft
	n = 0.016	C = 0.9	W_p = 34 ft
	d = 0.006 ft	W_D = 1.0 ft	L_D = 1.5 ft
	Zone 2	G1 = 0.01	G2 = -0.01
	Length of vertical curve = 3000 ft		
	PC station = 10+00	Begin bridge station = 10+00	

Step 1. Compute the distance from the turning point of the vertical curve to each end of the bridge.

- Locate the distance of the turning point from the PC.

$$X_t = \frac{-G1 * L}{(G2 - G1)} = \frac{-0.01 * 3000}{(-0.01 - 0.01)} = 1500 \ ft$$

- Compute the distance from the turning point to each end of the bridge.
  Since the length of bridge and vertical curve are equal,
  $L_{e1} = L_{e2}$ = 1500 ft

Step 2. Determine the rainfall intensity (turning point to end of bridge)

- Rational Method S = absolute value (G2) = 0.01
  Assume duration of 12.25 minutes

$$i = 10^{1.132076 - 0.60121 * \log(duration)} = 10^{1.132076 - 0.60121 * \log(12.25)} = 3.01 \ in / hr$$

$$t_g = 484 * \frac{S_x * T^2}{C * i * W_p} = 484 * \frac{0.02 * 10^2}{0.9 * 3.01 * 34} = 10.53 \text{ min}$$

$$t_0 = 0.93 * \frac{(W_p * n)^{0.6}}{(C * i)^{0.4} * S^{0.3}} = 0.93 * \frac{(34 * 0.016)^{0.6}}{(0.9 * 3.01)^{0.4} * 0.01^{0.3}} = 1.73 \text{ min}$$

$$t_c = t_0 + t_g = 1.73 + 10.53 = 12.25 \text{ min}$$

The assumed time of 12.25 minutes is equal to the computed time. Therefore, use i = 3.01 in/hr.

- Avoidance of Hydroplaning

$$i = \left[\frac{64904.4}{C * n}\right]\left[\frac{S_x}{(S_x^2 + S^2)^{0.25}}\right]\left[\frac{d^{1.67}}{(W_p - T)}\right] = \left(\frac{64904.4}{0.9 * 0.016}\right) * \frac{0.02}{(0.02^2 + 0.01^2)^{0.25}} * \frac{0.006^{1.67}}{(34 - 10)} = 4.89 \ in / hr$$

- Avoidance of Vision Impairment
  i = 4.00 in/hr
- Design rainfall intensity = 3.01 in/hr

Step 3. Assume the distance to the first inlet from the turning point to the end of bridge = 848 ft

Distance from PC to first inlet, X = 848 + 1500 = 2348 ft.

- Compute the slope at the assumed first inlet location

$$S = G1 + \frac{(G2 - G1) * X}{L_{vc}} = 0.01 + \frac{(-0.01 - 0.01) * 2348}{3000} = -0.00565$$

Use the absolute value of S. Therefore, S = 0.00565 ft/ft

Step 4. Compute the gutter flow at the design spread

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67} = \frac{0.56}{0.016} * 0.02^{1.67} * 0.00565^{0.5} * 10^{2.67} = 1.79 \ cfs$$

Step 5. Compute the distance to the first inlet and compare to the assumed value.

K = 1.0 for the first inlet

$$L_c = \frac{43560 * Q_f * K}{C * i * W_p} = \frac{43560 * 1.79 * 1.0}{0.9 * 3.01 * 34} = 848.07 \ ft$$

Assumed value of 848 ft equals computed value of 848.07 ft.

Inlets are required since the distance to the first inlet, 848 ft, is less than the distance from the turning point to the end of bridge, 1500 ft.

Step 6. Determine the spacing to the next inlet.

- Assume a distance to the next inlet and compute the slope at that point
  Assume 381 ft to the next drain
  X = 1500 + 848 + 381 = 2729 ft from the PC

$$S = G1 + \frac{(G2 - G1) * X}{L_{vc}} = 0.01 + \frac{(-0.01 - 0.01) * 2729}{3000} = -0.00819$$

Use the absolute value of S. Therefore, S = 0.00819.

- Compute the gutter flow at the design spread

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67} = \frac{0.56}{0.016} * 0.02^{1.67} * 0.00819^{0.5} * 10^{2.67} = 2.16\ cfs$$

- Compute the inlet efficiency, E.

$$E_0 = 1 - \left[1 - \frac{W_d}{T}\right]^{2.67} = 1 - \left(1 - \frac{1.0}{10}\right)^{1.67} = 0.245$$

$$V = \frac{1.12}{n} * S_x^{0.67} * S^{0.5} * T^{0.67} = \frac{1.12}{0.016} * 0.02^{0.67} * 0.00819^{0.5} * 10^{0.67} = 2.155\ fps$$

$$V_0 = 10^{0.654078 * \log(L_d) + 0.474699} = 10^{0.654078 * \log(1.5) + 0.474699} = 3.889\ fps$$

$$R_f = 1 - 0.09 * (V - V_0) = 1 - 0.09 * (2.155 - 3.889) = 1.156$$

Maximum R_f = 1.0

$$E = R_f * E_0 = 1.0 * 0.245 = 0.245$$

- Compute the interception coefficient, K.
  Su = 0.00565
  S = 0.00819

$$K = 1 - (1 - E) * \left(\frac{S_u}{S}\right)^{0.5} = 1 - \left[(1 - 0.245) * \left\{\frac{0.00565}{0.00819}\right\}^{0.5}\right] = 0.373$$

- Compute the inlet spacing and compare to the assumed value.

$$L_i = \frac{43560 * Q_f * K}{C * i * W_p} = \frac{43560 * 2.16 * 0.373}{0.9 * 3.01 * 34} = 380.72\ ft$$

The computed length of 380.72 ft equals the assumed value of 381 ft.

Step 7. Repeat step 6 for each inlet until the end of the bridge is reached.
Since the bridge is this example is symmetrical, the drain configuration will be the same from the turning point to both the begin/end of bridge. The following table summarizes the inlet spacing.

Spacing of inlets from the turning point to begin/end bridge				
Inlet	Spacing, Li Feet	Station	Slope, S Ft/ft	Gutter flow, Q cfs
	848.07			
1		3348.07	0.00565	1.79
	380.72			
2		3728.79	0.00819	2.16
	357.40			

Only 2 drains are required on the bridge. The 357.40 ft to the third drain puts it at station 4086.19, which is beyond the end of the bridge.

Step 8. Design bridge end treatment

- Since drains are required, assume blockage = 50%
- Assumed duration = 23.5 minutes Assumed spread = 11.75 feet

$$i = 10^{1.132076 - 0.60121 * \log(duration)} = 10^{1.132076 - 0.60121 * \log 23.5} = 2.03\ in/hr$$

$$t_0 = 0.93 * \frac{(W_p * n)^{0.6}}{(C * i)^{0.4} * S^{0.3}} = 0.93 * \frac{(34 * 0.016)^{0.6}}{(0.9 * 2.03)^{0.4} * 0.01^{0.3}} = 2.02 \text{ min}$$

$$t_g = 484 * \frac{S_x * T^2}{C * i * W_p} = 484 * \frac{0.02 * 11.75^2}{0.9 * 2.03 * 34} = 21.50 \text{ min}$$

$$t_c = t_0 + t_g = 2.02 + 21.50 = 23.52 \text{ min}$$

23.5 minutes are equal to the assumed value.

- Compute the design flow with the percent blockage

$$Q = C * i * A - [(blockage) * E * (number\ of\ drains) * V]$$

$$A = \frac{W_p * (length\ of\ bridge)}{43560} = \frac{34 * 3000}{43560} = 2.34\ acres$$

$$V = \frac{1.12}{n} * S_x^{0.67} * S^{0.5} * T^{0.67} = \frac{1.12}{0.016} * 0.02^{0.67} * 0.01^{0.5} * 11.75^{0.67} = 2.653\ fps$$

$$Q = 0.9 * 2.03 * 2.34 - [0.50 * 0.245 * 2 * 2.653] = 3.63\ cfs$$

- Check the design spread with the new computed flow. Compute the spread by solving the gutter flow equation for T.

$$Q_f = \frac{0.56}{n} * S_x^{1.67} * S^{0.5} * T^{2.67}$$

$$T = \left(\frac{Q_f * n}{0.56 * S_X^{1.67} * S^{0.5}} \right)^{\frac{1}{2.67}} = \left(\frac{3.63 * 0.016}{0.56 * 0.02^{1.67} * 0.01^{0.5}} \right)^{\frac{1}{2.67}} = 11.71\ ft$$

This is equal to the assumed value of 11.75 feet.

- Select an inlet that will handle 3.63 cfs and provide a pipe or paved ditch to convey the design flow from the drain to the toe of the embankment. Provide energy dissipation, if necessary, at the toe to achieve nonerosive velocities.

SLOPE PAVING

Slope protection under bridges shall generally be provided under the end spans of grade separations and interchange structures and at railroad overpasses. Slope protection shall be concrete or crushed rock. Consult with the District to determine which one shall be used.

Concrete slope paving shall be used in urban and suburban areas where vandalism could be a problem. Vandalism may be anticipated even if pedestrian traffic is prohibited under the bridge. Concrete slope paving may also be required by a duly constituted authority such as a railroad company. The maximum recommended slope for concrete slope paving is 2:1.

Crushed rock of approximately 4” maximum dimension may be used at locations where vandalism is not likely to be a problem. The District should verify that a material’s source for the rock is readily available.

Avoid unpaved areas between slope paving and sidewalks. Avoid conducting water drained from slope paving across the top of a sidewalk.

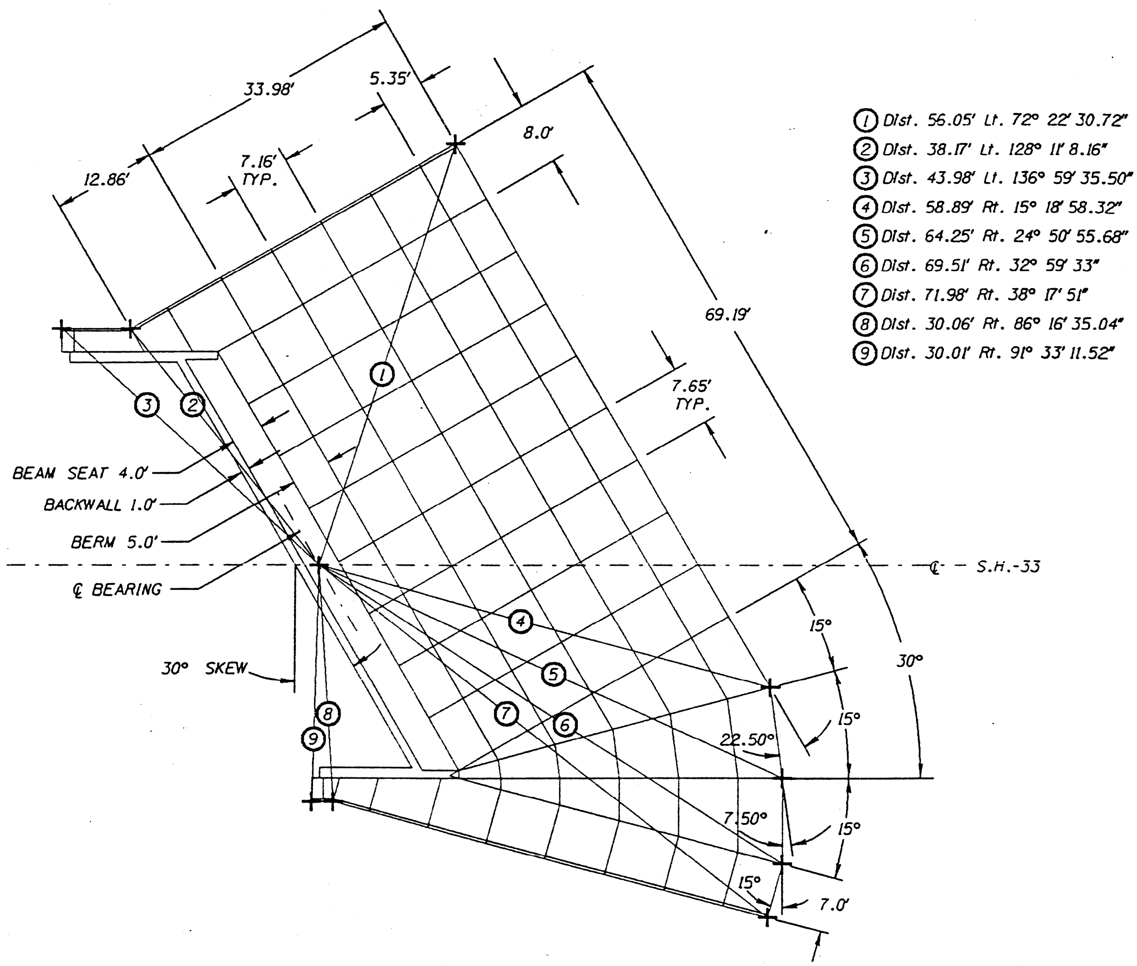
For dual structures over 15º skew, the end flared section shall be the same as for single structures over 15º skew.

The center section shown on the standard drawing is typical for all dual structures at any skew angle with medians not more than 76’ edge to edge of traveled lanes. For median strips greater than 76’ use two single structures.

Welded wire fabric may be deleted if fiber reinforced concrete is used. Modify the Standard Drawing details accordingly and add the following note to the Standard Drawing:

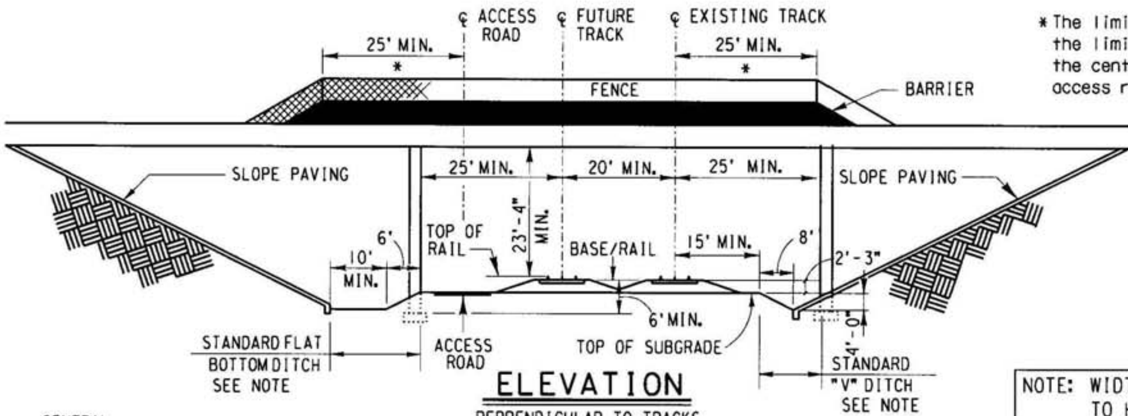
> Polypropylene fibrillated fibers, Fiber Mesh MD or equal, shall meet the material specifications of ASTM C-1116, Type 111 Section 4.1.3 (Synthetic Fiber Reinforced Concrete or Shotcrete). The weight of the fibers shall be 0.056 pcf.

Calculate the data shown in the following sketch and show on the drawing to make the field layout easier.



Revisions:

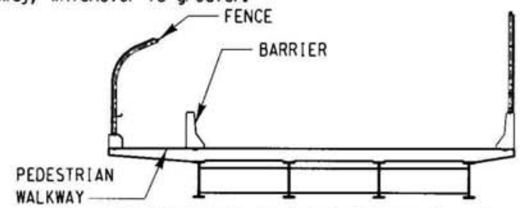
April 2008 Added instructions for location and type of slope paving.



ELEVATION

PERPENDICULAR TO TRACKS

* The limits of the fence with barrier rail shall extend to the limits of Railroad Right-of-Way or a minimum of 25 ft. beyond the centerline of the outermost existing track, future track or access roadway, whichever is greater.



OVERHEAD STRUCTURE CROSS SECTION WITH FENCE

NOTE: WIDTH AND HEIGHT SUBJECT TO HYDRAULIC REQUIREMENTS. SEE PLAN NO. 711100, SHEET 5.

GENERAL

Fence shall be provided as indicated on the cross sections and elevation view on both sides of the Overhead Structure in ALL new or modified structures.

Barrier rail for Overhead Structures, without walkways, that may be subject to snow removal shall be a minimum of 42 inches in height with a 4 foot wide shoulder or 30 inches in height with a 6 foot wide shoulder. See Plan No. 711100, Sheet 4.

Lights are to be installed on the underside of the Overhead Structure where shadows cast by the structure would interfere with Railroad operations.

Slope paving shall be provided where end slopes are equal to or exceed 2 horizontal to 1 vertical.

Falsework for construction of overhead structures shall comply with Railroad Requirements.

Demolition of existing Overhead Structures shall comply with Railroad Demolition Requirements. Temporary shoring shall be designed in accordance with Railroad Guidelines for Temporary Shoring.

Applicant shall be responsible for identification, location and protection of existing utilities.

Call the following numbers at least 48 hours prior to commencing work to determine location of fiber optics:
UPRR "Call Before You Dig", 1-800-336-9193
BNSF "Call Before You Dig", 1-800-533-2891.

CLEARANCES

Minimum vertical clearance shall be 23'-4" above the top of high rail within 25' of centerline of track. Additional clearance may be required for construction purposes or if sag of vertical curve must be adjusted or if future track raise for flood considerations or maintenance is probable.

Minimum horizontal clearances, measured at right angle from centerline of track, shall be as shown in elevation view.

For minimum construction clearances, see Plan No. 711100, Sheet 3.

PIERS

Piers shall be located outside Railroad Right-of-Way.

Pier protection walls shall be provided in accordance with AREMA Chapter 8, Part 2.1.5 for piers within 25 feet of the centerline of track.

Top of footings located within 25 feet from centerline of track shall be a minimum of 6 feet below base of rail and a minimum of 1 foot below flowline of ditch.

DRAINAGE

Drainage from the Overhead Structure shall be diverted away from and not discharged onto the tracks, roadbed and Railroad Right-of-Way.

At minimum, a standard "V"-shaped or flat-bottom ditch shall be provided on each side of the tracks as necessary.

Culverts may be installed in lieu of standard Railroad ditches when approved by Railroad Central Engineering. Maintenance of culverts will be at Applicant's expense.

FUTURE TRACKS AND ACCESS ROAD

Space is to be provided for one or more future tracks as required for long range planning or other operating requirements. Where provision is made for more than two tracks, space is to be provided for an access road on both sides of tracks.

REVISIONS		
DATE	LTR.	DESCRIPTION
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/		
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/		

DESIGN BY: RAF | DRAWN BY: KDM | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY — UNION PACIFIC

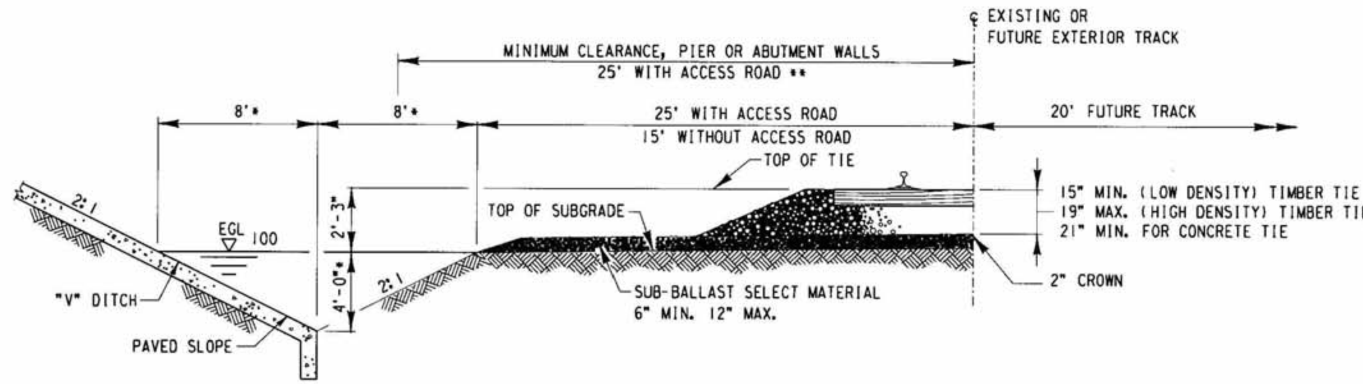
BRIDGE STANDARDS

GRADE SEPARATION GUIDELINES (OVERHEADS)

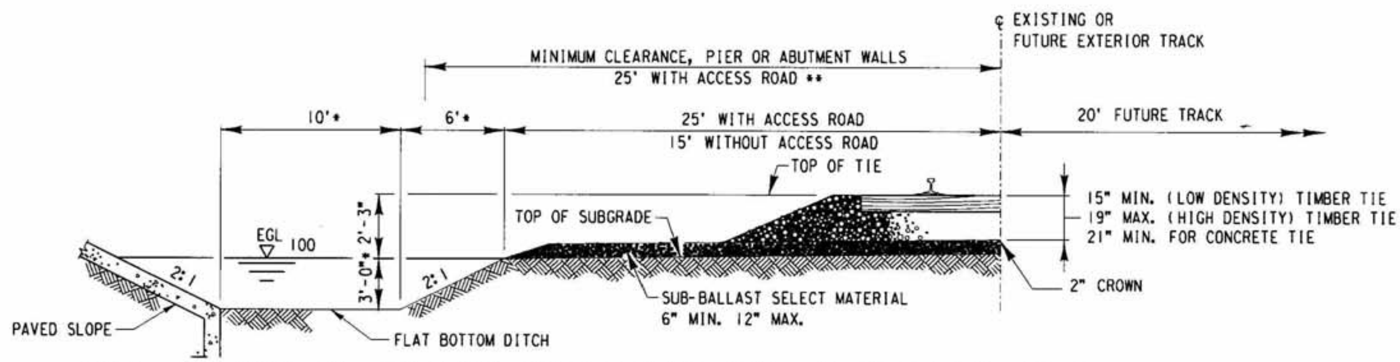
GENERAL OVERHEAD STRUCTURE DRAWING

FILE OWNER: UPRR | DATE: 1/24/07

PLAN NO.: 711100 | SHEET: 1



TYP. SECTION AT ABUTMENT SLOPES WITH STD. "V" DITCH



TYP. SECTION AT ABUTMENT SLOPES WITH STD. FLAT BOTTOM DITCH

NOTE:

MINIMUM DITCH SIZES ARE SHOWN. DITCH SIZE TO BE INCREASED AS REQUIRED BASED ON RR HYDRAULIC CRITERIA.

* WIDTH AND HEIGHT SUBJECT TO HYDRAULIC REQUIREMENTS.

** LOCATION OF PIER, BENT COLUMNS OR ABUTMENT WALLS SHOULD NOT INTERFERE WITH THE DRAINAGE IN THE AREA. IF MINIMUM STANDARD DITCHES ARE NOT PROVIDED IN THE LAYOUT, LONGITUDINAL CULVERTS SHALL BE PROVIDED THAT WILL HANDLE THE DRAINAGE AS REQUIRED BY THE HYDRAULIC STUDIES.

REVISIONS		
DATE	LTR.	DESCRIPTION
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/		

DESIGN BY: RAF | DRAWN BY: FJS | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY

UNION PACIFIC

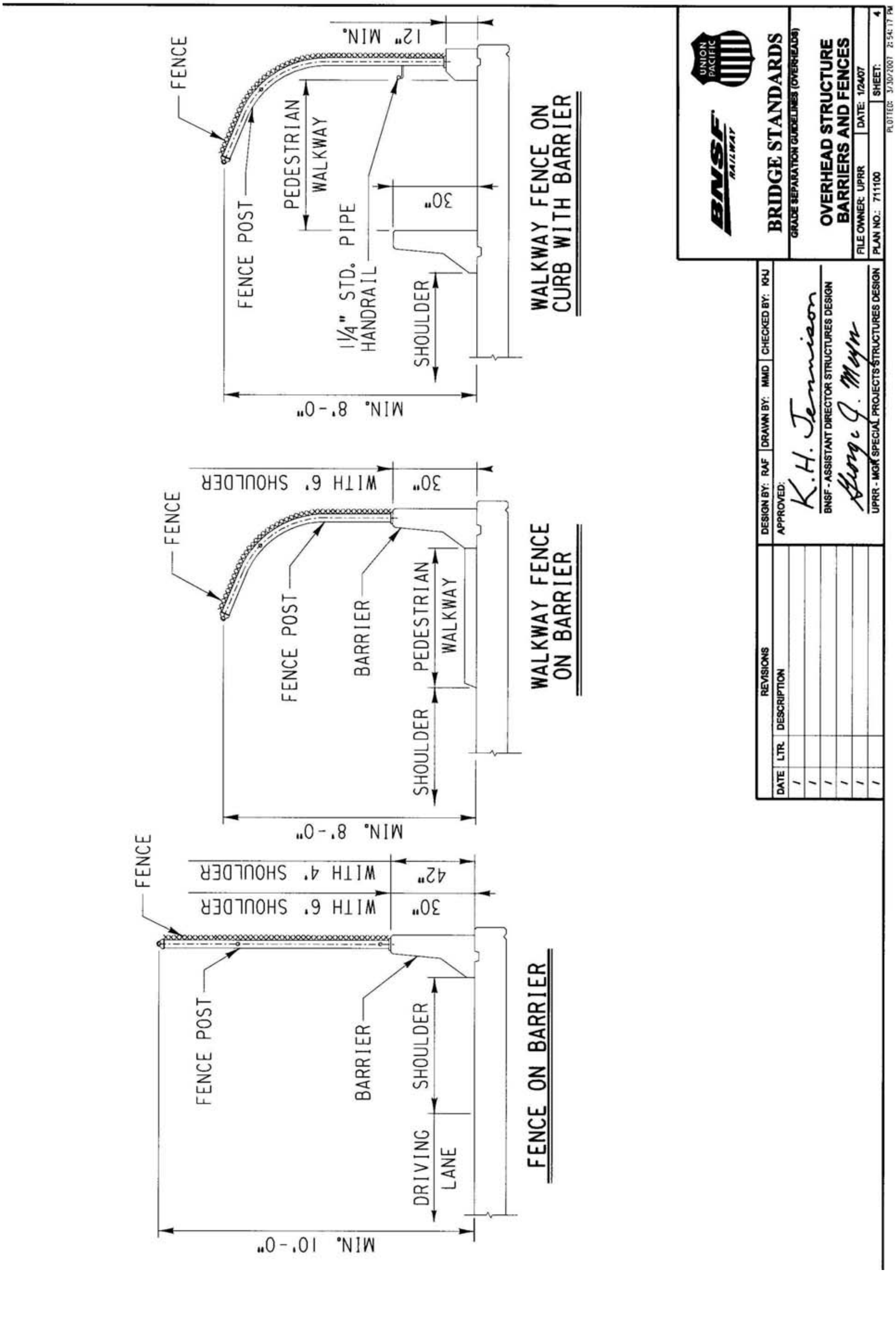
BRIDGE STANDARDS

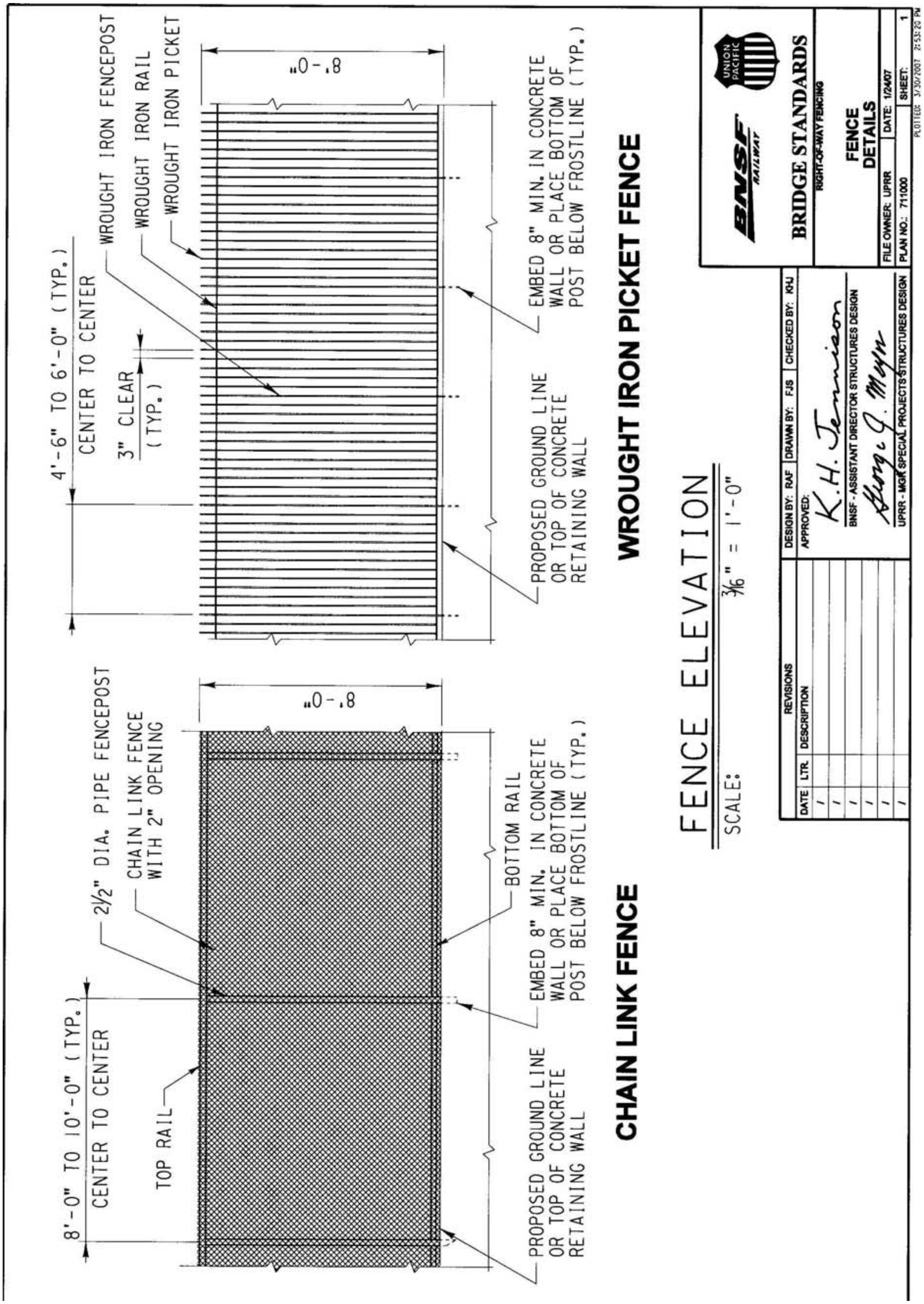
GRADE SEPARATION GUIDELINES (OVERHEADS)

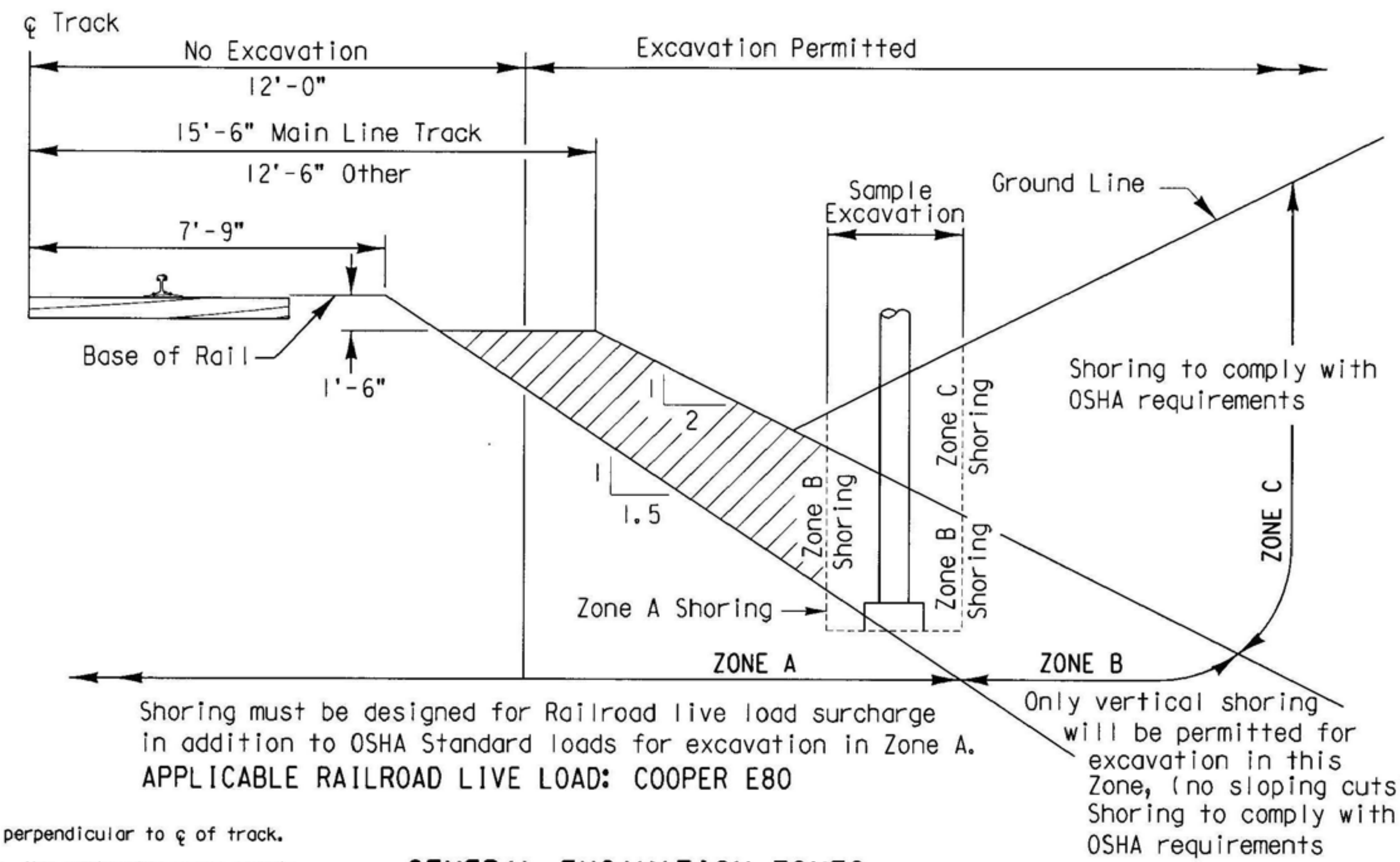
TYPICAL SECTIONS AT ABUTMENT SLOPES

FILE OWNER: UPRR | DATE: 1/24/07

PLAN NO.: 711100 | SHEET: 5


FENCE
FENCE POST
BARRIER
SHOULDER
DRIVING LANE
WITH 6' SHOULDER
30"
WITH 4' SHOULDER
42"
MIN. 10'-0"
FENCE ON BARRIER
FENCE
FENCE POST
BARRIER
PEDESTRIAN WALKWAY
SHOULDER
WITH 6' SHOULDER
30"
MIN. 8'-0"
WALKWAY FENCE ON BARRIER
FENCE
FENCE POST
PEDESTRIAN WALKWAY
1¼" STD. PIPE HANDRAIL
SHOULDER
30"
12" MIN.
MIN. 8'-0"
WALKWAY FENCE ON CURB WITH BARRIER
REVISIONS
DATE
LTR.
DESCRIPTION
DESIGN BY: RAF
DRAWN BY: MMD
CHECKED BY: KHJ
APPROVED:
K.H. Jennison
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN
BNSF RAILWAY
UNION PACIFIC
BRIDGE STANDARDS
GRADE SEPARATION GUIDELINES (OVERHEADS)
OVERHEAD STRUCTURE BARRIERS AND FENCES
FILE OWNER: UPRR
DATE: 1/24/07
PLAN NO.: 711100
SHEET: 4
PLOTTED: 3/30/2007 2:54:17 PM


FENCE ELEVATION
SCALE: 3/16" = 1'-0"
CHAIN LINK FENCE
8'-0" TO 10'-0" (TYP.)
CENTER TO CENTER
TOP RAIL
2½" DIA. PIPE FENCEPOST
CHAIN LINK FENCE
WITH 2" OPENING
8'-0"
BOTTOM RAIL
EMBED 8" MIN. IN CONCRETE
WALL OR PLACE BOTTOM OF
POST BELOW FROSTLINE (TYP.)
PROPOSED GROUND LINE
OR TOP OF CONCRETE
RETAINING WALL
WROUGHT IRON PICKET FENCE
4'-6" TO 6'-0" (TYP.)
CENTER TO CENTER
3" CLEAR
(TYP.)
WROUGHT IRON FENCEPOST
WROUGHT IRON RAIL
WROUGHT IRON PICKET
8'-0"
EMBED 8" MIN. IN CONCRETE
WALL OR PLACE BOTTOM OF
POST BELOW FROSTLINE (TYP.)
PROPOSED GROUND LINE
OR TOP OF CONCRETE
RETAINING WALL
REVISIONS
DATE
LTR.
DESCRIPTION
DESIGN BY: RAF
DRAWN BY: FJS
CHECKED BY: KHJ
APPROVED:
K.H. Jennison
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN
BNSF RAILWAY
UNION PACIFIC
BRIDGE STANDARDS
RIGHT-OF-WAY FENCING
FENCE
DETAILS
FILE OWNER: UPRR
DATE: 1/24/07
PLAN NO.: 711000
SHEET: 1
PLOTTED: 3/30/2007 2:53:20 PM



Shoring must be designed for Railroad live load surcharge in addition to OSHA Standard loads for excavation in Zone A.

APPLICABLE RAILROAD LIVE LOAD: COOPER E80

GENERAL NOTES:

All dimensions are measured perpendicular to ℄ of track.

Prior to commencing any work, the contractor shall submit for approval by the Railroad detailed plans indicating the nature and extent of the track protection shoring proposed. The contractor shall install the temporary shoring system per the approved plans. Design of the temporary shoring system to comply with **GUIDELINES FOR TEMPORARY SHORING.**

For excavations which encroach into zone A or B, shoring plans shall be accompanied by design calculations. Plans and calculations must be signed and stamped by a Professional Engineer registered in the state where the work will be performed.

GENERAL EXCAVATION ZONES

SCALE: (NOT TO SCALE)

REVISIONS		
DATE	LTR.	DESCRIPTION
5/03	1	FORMERLY UPRR C.E. 106613
/		
/		
/		
/		
/		

DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

9-1-04

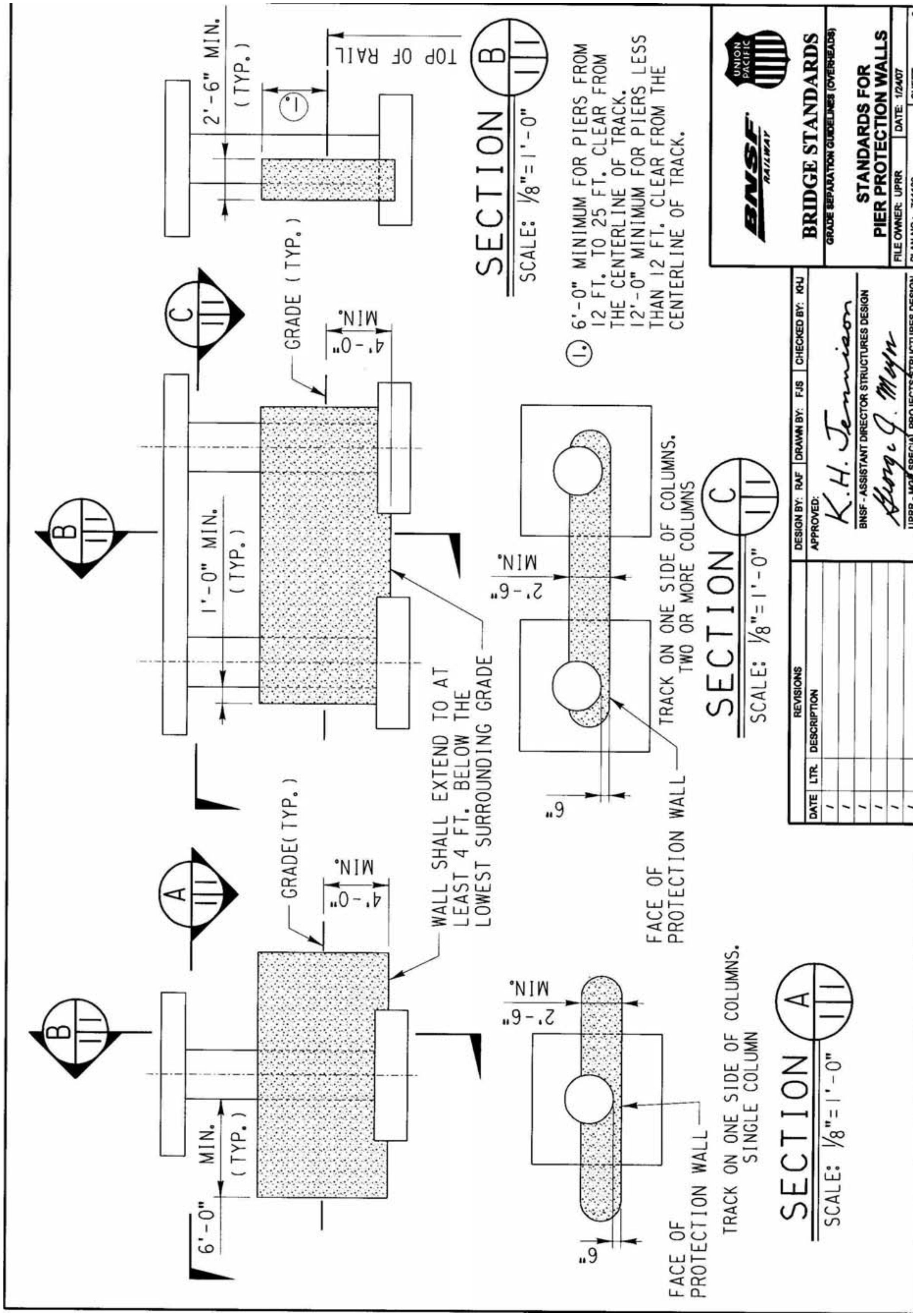
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

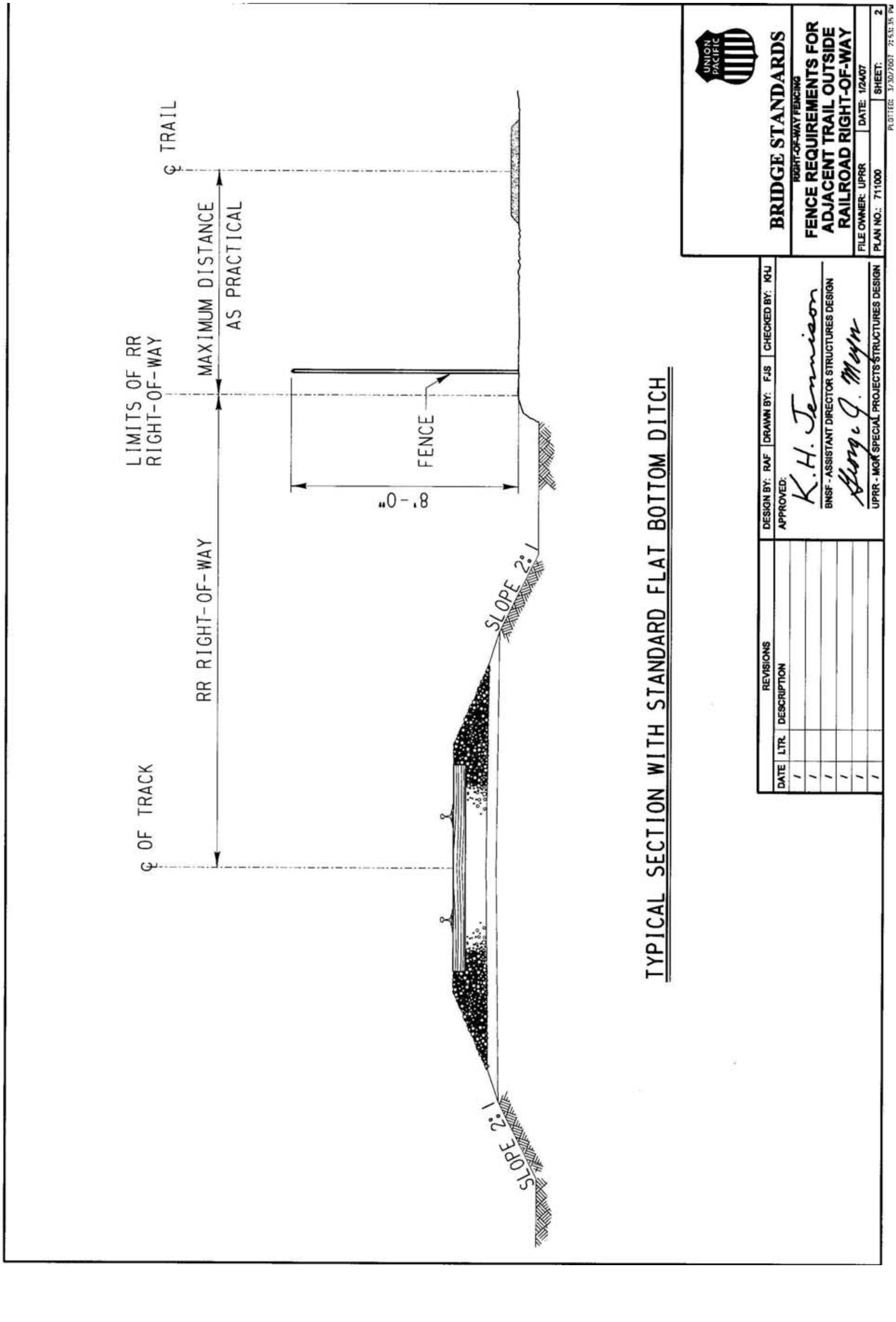
BRIDGE STANDARDS

GENERAL SHORING REQUIREMENTS

FILE OWNER: UPRR | DATE: 5-6-03

PLAN NO.: 710000 1 | SHEET: 1 OF 1


6'-0" MIN.
(TYP.)
GRADE (TYP.)
4'-0" MIN.
WALL SHALL EXTEND TO AT LEAST 4 FT. BELOW THE LOWEST SURROUNDING GRADE
FACE OF PROTECTION WALL
2'-6" MIN.
6"
PROTECTION WALL
TRACK ON ONE SIDE OF COLUMNS. SINGLE COLUMN
SECTION A
SCALE: 1/8"=1'-0"
1'-0" MIN.
(TYP.)
GRADE (TYP.)
4'-0" MIN.
2'-6" MIN.
6"
FACE OF PROTECTION WALL
TRACK ON ONE SIDE OF COLUMNS. TWO OR MORE COLUMNS
SECTION C
SCALE: 1/8"=1'-0"
2'-6" MIN.
(TYP.)
TOP OF RAIL
SECTION B
SCALE: 1/8"=1'-0"
1. 6'-0" MINIMUM FOR PIERS FROM 12 FT. TO 25 FT. CLEAR FROM THE CENTERLINE OF TRACK. 12'-0" MINIMUM FOR PIERS LESS THAN 12 FT. CLEAR FROM THE CENTERLINE OF TRACK.
BNSF RAILWAY
UNION PACIFIC
BRIDGE STANDARDS
GRADE SEPARATION GUIDELINES (OVERHEADS)
STANDARDS FOR PIER PROTECTION WALLS
FILE OWNER: UPRR
DATE: 1/24/07
PLAN NO.: 711100
SHEET: 6
DESIGN BY: RAF
DRAWN BY: FJS
CHECKED BY: KHJ
APPROVED:
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN
REVISIONS
DATE
LTR.
DESCRIPTION


℄ TRAIL
LIMITS OF RR
RIGHT-OF-WAY
MAXIMUM DISTANCE
AS PRACTICAL
FENCE
8'-0"
RR RIGHT-OF-WAY
℄ OF TRACK
SLOPE 2:1
SLOPE 2:1
TYPICAL SECTION WITH STANDARD FLAT BOTTOM DITCH
UNION PACIFIC
BRIDGE STANDARDS
RIGHT-OF-WAY FENCING
FENCE REQUIREMENTS FOR
ADJACENT TRAIL OUTSIDE
RAILROAD RIGHT-OF-WAY
FILE OWNER: UPRR
DATE: 1/24/07
PLAN NO.: 711000
SHEET: 2
DESIGN BY: RAF
DRAWN BY: FJS
CHECKED BY: KHJ
APPROVED:
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN
REVISIONS
DATE
LTR.
DESCRIPTION

Properties & Support Spacing for PVC Pipe

Where PVC conduit is to be supported by hangers or pedestals at intervals, the distance between supports shall be small enough to avoid excessive sag of the conduit.

Recommended support spacing and tabulated properties of PVC pipe are shown in the following Table.

SCHEDULE 40 PVC					
Nominal size	O.D. – inches	Average I.D. inches	Minimum Wall Thickness inches	Pounds per 100 ft	Support Spacing feet
¾”	1.050	0.804	0.113	22.6	4
1”	1.315	1.029	0.133	33.3	4.5
1¼”	1.660	1.367	0.140	45.0	5
1½”	1.9	1.590	0.145	53.7	5
2”	2.375	2.047	0.154	72.0	5
2½”	2.875	2.445	0.203	113.6	6
3	3.5	3.042	0.216	148.8	6
4	4.5	3.998	0.237	211.8	6.5
5	5.563	5.016	0.258	287.4	7
6	6.625	6.031	0.280	373.3	7.5
8	8.625	7.942	0.322	561.9	8
10	10.75	9.976	0.365	796.6	8.5
12	12.75	11.889	0.406	1053.4	9.5
SCHEDULE 80 PVC					
Nominal size	O.D. – inches	Average I.D. inches	Minimum Wall Thickness inches	Pounds per 100 ft	Support Spacing-feet
½”	0.840	0.526	0.147	21.3	4.5
¾”	1.050	0.722	0.154	28.9	4.5
1”	1.315	0.936	0.179	42.4	5
1¼”	1.660	1.255	0.191	58.6	5.5
1½”	1.900	1.476	0.200	71.1	5.5
2”	2.375	1.913	0.218	98.4	6
2½”	2.875	2.290	0.276	150.0	6.5
3	3.500	2.864	0.300	201.0	7
4	4.500	3.786	0.337	293.8	7.5
5	5.563	4.768	0.375	407.8	8
6	6.625	5.709	0.432	561.0	9
8	8.625	7.565	0.500	852.2	9.5

Spacing shown is set for a 100° maximum temperature.
The physical properties of PVC material are:

E = 410,000 psi
Tensile Strength = 7300 psi at 78°F
Working Stress in Bending = 4.0 ksi
Temperature coefficient = 0.035” per 100°/ft

Commentary

The Table was taken from the July 2005 WSDOT Bridge Design Manual. The values were obtained from ASTM D1785 and Harvel Plastics.

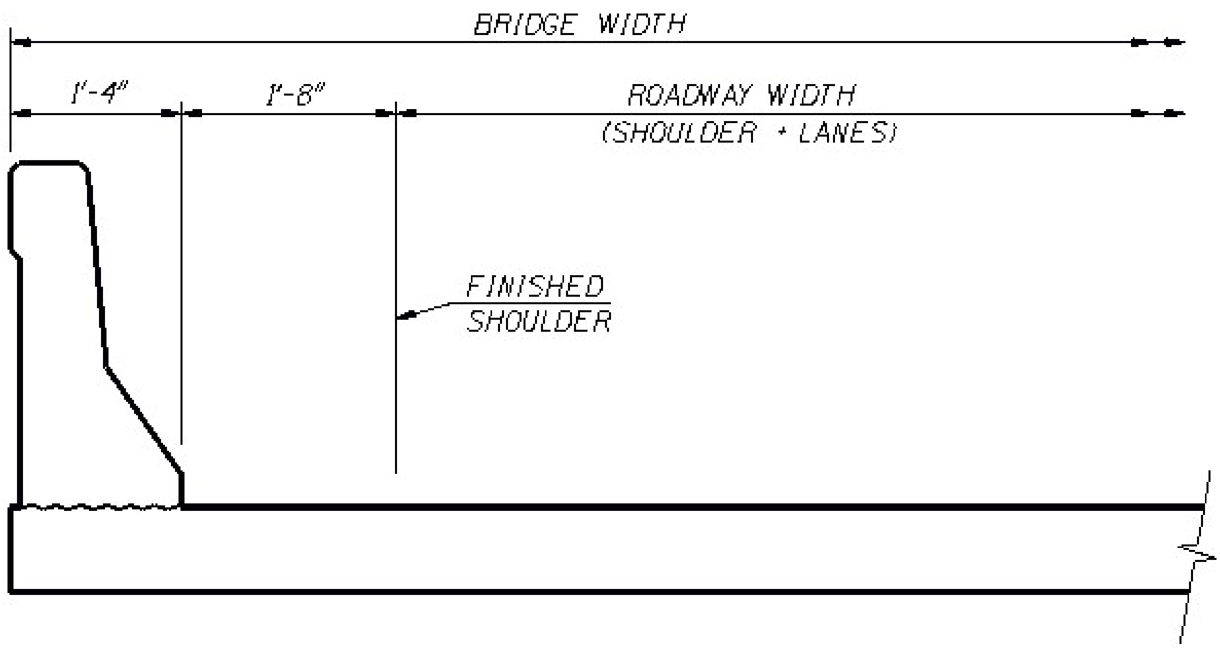
Revisions:

June 2006 Added new article.

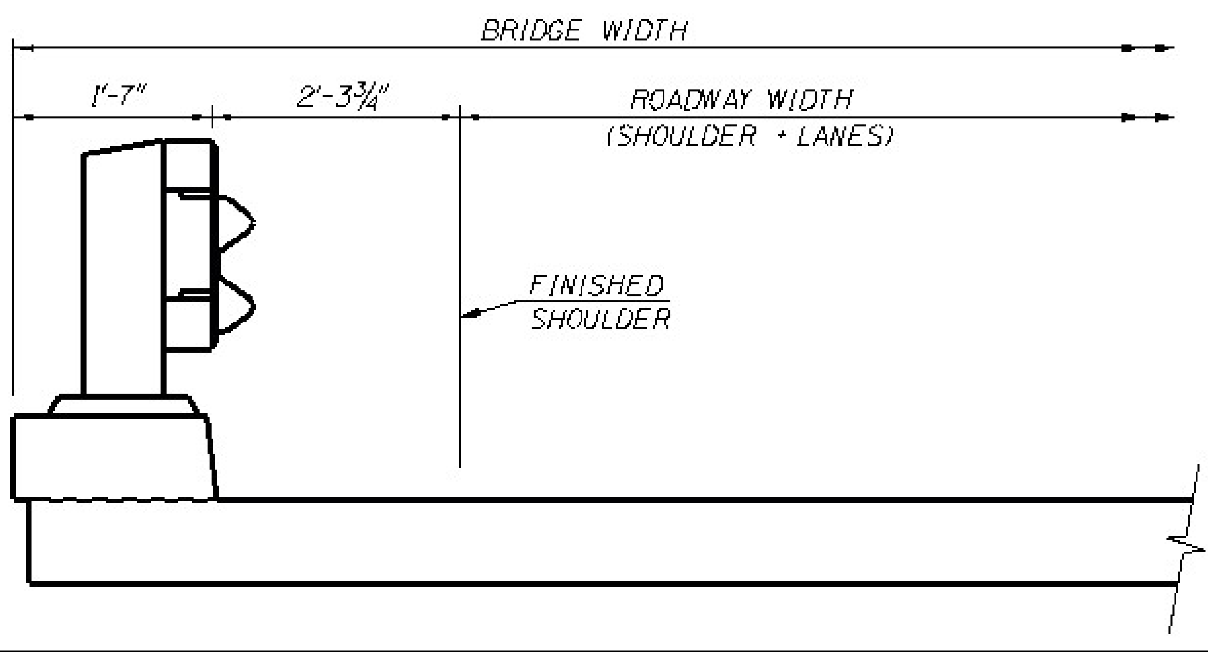
STRUCTURE WIDTH

BRIDGES – NO SIDEWALK

Concrete Parapet

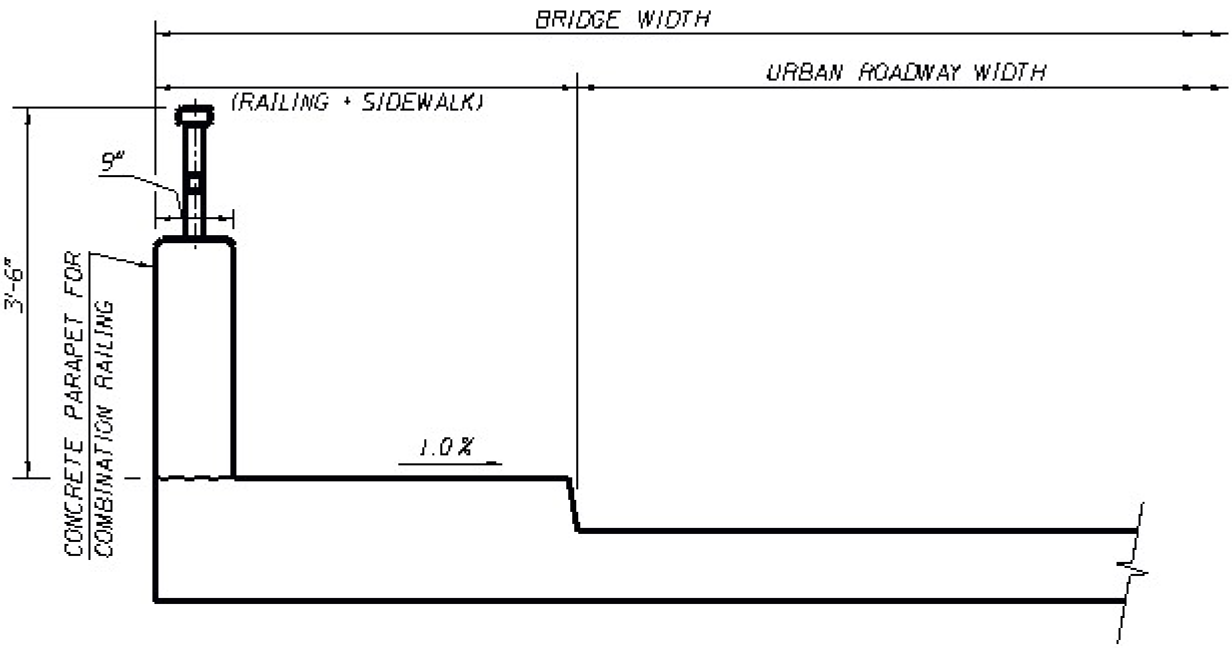


Two Tube Curb Mount Rail

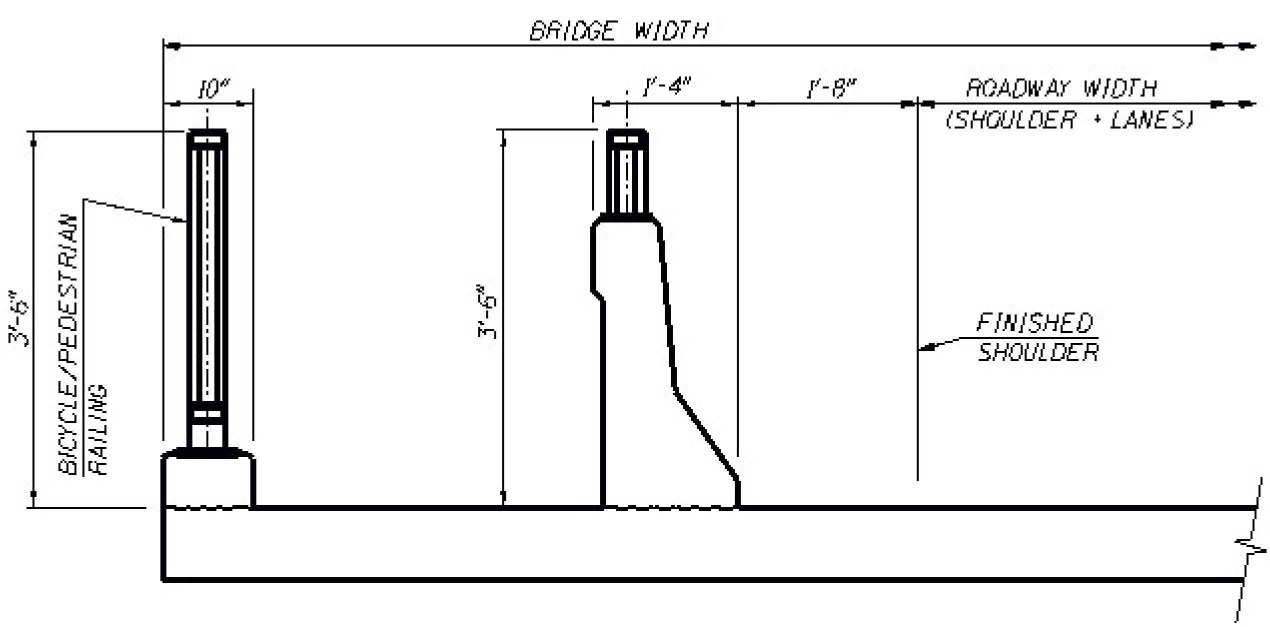


BRIDGES – RAISED SIDEWALK WITH SPEEDS 45mph AND LOWER

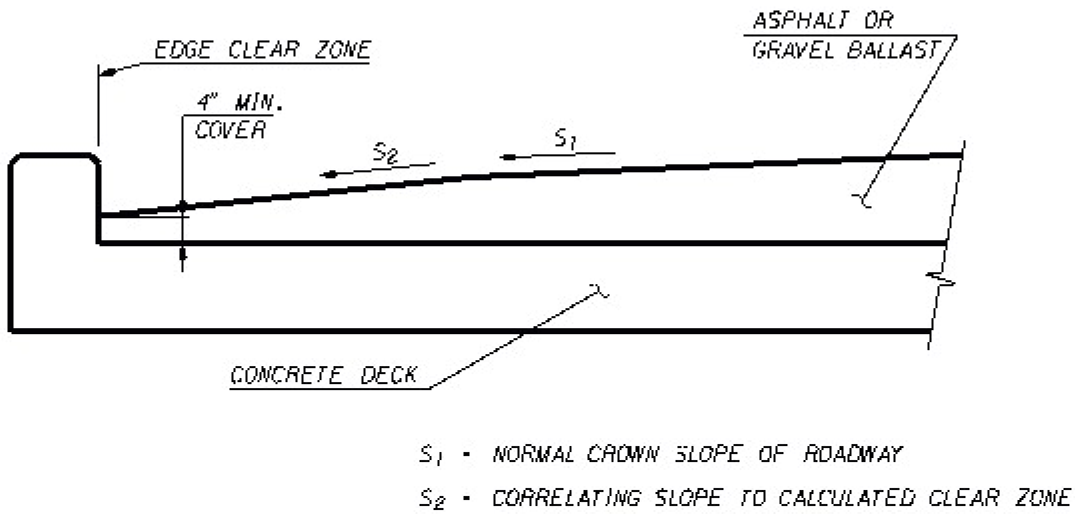
Combination Rail



BRIDGES – AT-GRADE SIDEWALK FOR SPEEDS OVER 45mph

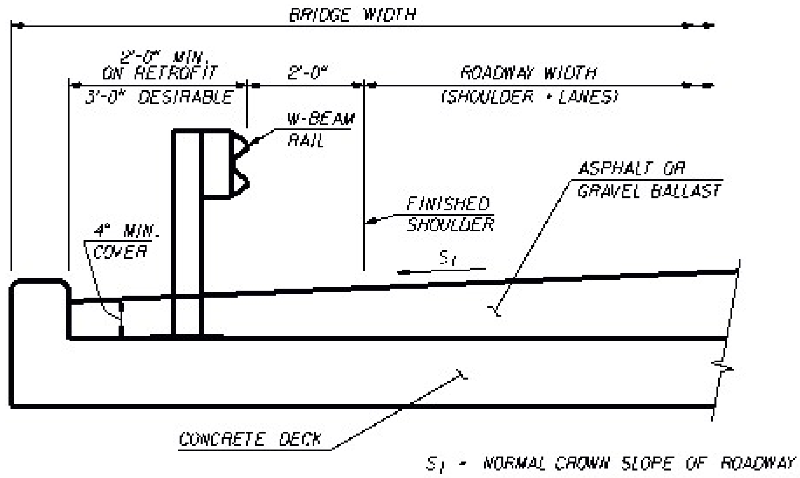


CULVERTS – CLEAR ZONE PROVIDED

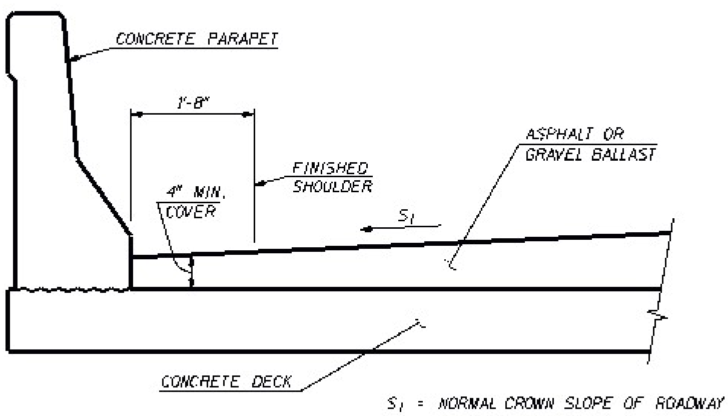


CULVERTS – CLEAR ZONE NOT PROVIDED

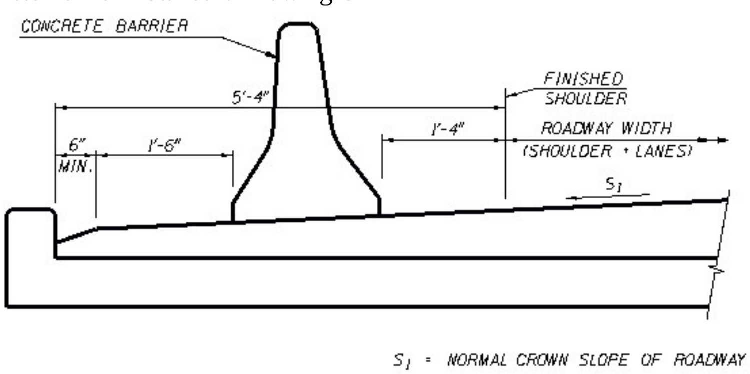
Test Level 3 Rail



Test Level 4 Rail



Concrete Barrier – Standard Drawing G-2-A



CHAPTER 2
STANDARD DRAWING REVISION LOG

B2.4A Utility Hangars for Prestressed Girders
June 2006 Added new standard drawing for attaching water, sewer, and gas lines to new bridges.

B2.4B Utility Hangars for Prestressed Girders
June 2006 Added new standard drawing for attaching power, and communication lines to new bridges

3.4.1 LOAD FACTORS

- γ_{EQ} shall be assumed as 0.0 for Extreme Event I Load Combination.
- γ_{SE} & γ_{TG} shall be taken as follows:

  0.0 for strength and extreme event limit states
  1.0 for service limit states if live load is not considered
  0.5 for service limit states with live load

3.5.1 DEAD LOADS

All structures whose deck slabs will be exposed to traffic shall be designed for a future wearing surface.

Box girder bridges (steel, conventional reinforced, post-tension) shall be designed for a future 1¼" concrete overlay to replace the original top 1" non-structural deck slab concrete. Use 3 psf for the value of the future wearing surface.

All other structures shall be designed for a future wearing surface of 28 psf .

3.6.1.1.2 MULTIPLE PRESENCE FACTORS

The factors specified in the Table shall not be applied in conjunction with approximate load distribution factors specified in Articles 4.6.2.2 and 4.6.2.3, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges, specified in Article 4.6.2.2d, are used

Multiple presence factors are based on the directional ADTT. If directional ADTT percentages are not available, they may be assumed to be 50% of the total ADTT.

Number of Loaded Lanes	MPF for design year ADTT>1000 in one direction	Adjusted MPF for design year (0.95) 100≤ADTT≤1000 in one direction	Adjusted MPF for design year (0.90) ADTT<100 in one direction
1	1.20	1.14	1.08
2	1.00	0.95	0.90
3	0.85	0.81	0.77
>3	0.65	0.62	0.59

Commentary:
The multiple presence factor adjustment is described in C3.6.1.1.2.

Revisions:
April 2008 Added Commentary and listed adjustment factors in the table.

3.6.1.2 DESIGN VEHICULAR LIVE LOAD

3.6.1.2.1 General

NEW CONSTRUCTION
Bridges and culverts, which carry traffic on Interstates & Ramps, US, State Primary, and State Secondary highways, shall be designed for HL93 loading using the LRFD Design Code.

Local and Off-System bridges shall be designed for HL93 loading using the LRFD Design Code.

MODIFICATION OF EXISTING STRUCTURES
Where economically feasible, the modifications to structures carrying US, State, Interstate, and Local traffic should be designed for HL93 loading using the LRFD Design Code. When HL93 loading will require extensive upgrading to other structural elements, the live load used shall be determined by the Bridge Engineer.

Commentary:
Refer to Article 0.0 Commentary.

Revisions:

April 2008	Deleted 17th Edition Design for Local and Off System bridges. Added Commentary.

3.6.1.3 APPLICATION OF VEHICULAR LIVE LOADS

3.6.1.3.1 General

All bridges will be designed for Strength I case using the controlling live load cases:

A. Lane Load + Design Tandem.
B. Lane Load + Design Truck.
C. Negative moment between points of contraflexure only.
    1. 90% of two Design trucks (with 14' axle spacing) going in the same direction a minimum of 50 feet apart (measured from the rear axle of the lead vehicle to the front axle of the trailing vehicle) + 90% lane load.
    2. Two Design tandems spaced 26' to 40' apart + lane load

3.6.1.3.2 Loading for Optional Live Load Deflection

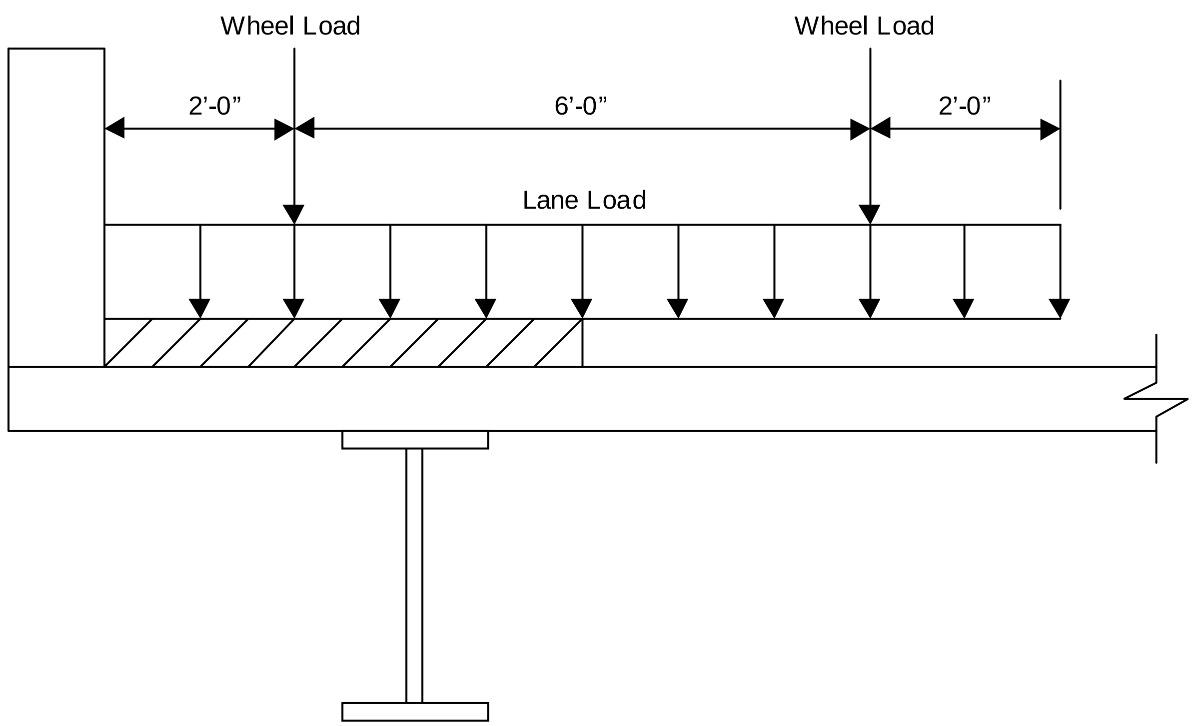
The live load deflection should be based on the provisions of this article.

3.6.1.6 PEDESTRIAN LOADS

For the Strength and Service Limit States, the pedestrian and vehicular live loading (HL-93) shall be placed as stated in the commentary of article 3.6.1.1.2.

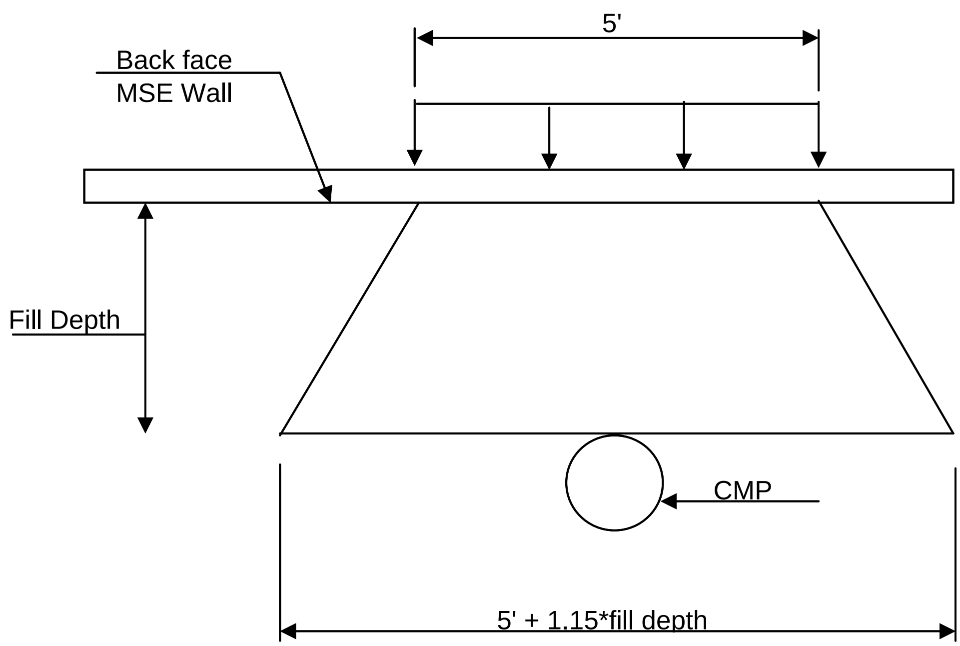
For the case of the vehicular live loading (HL-93) mounting the sidewalk, an Extreme Event-III load combination is to be used. The vehicular live loading is to consist of one loaded lane, with distribution factors determined by the lever rule, and on narrow bridges the rotation of the bridge cross section.

Extreme Event-III: γ_P*DC + γ_P*DW + 1.75*(LL+IM)
$\varphi = 1.0$
M.P.F. = 1.20



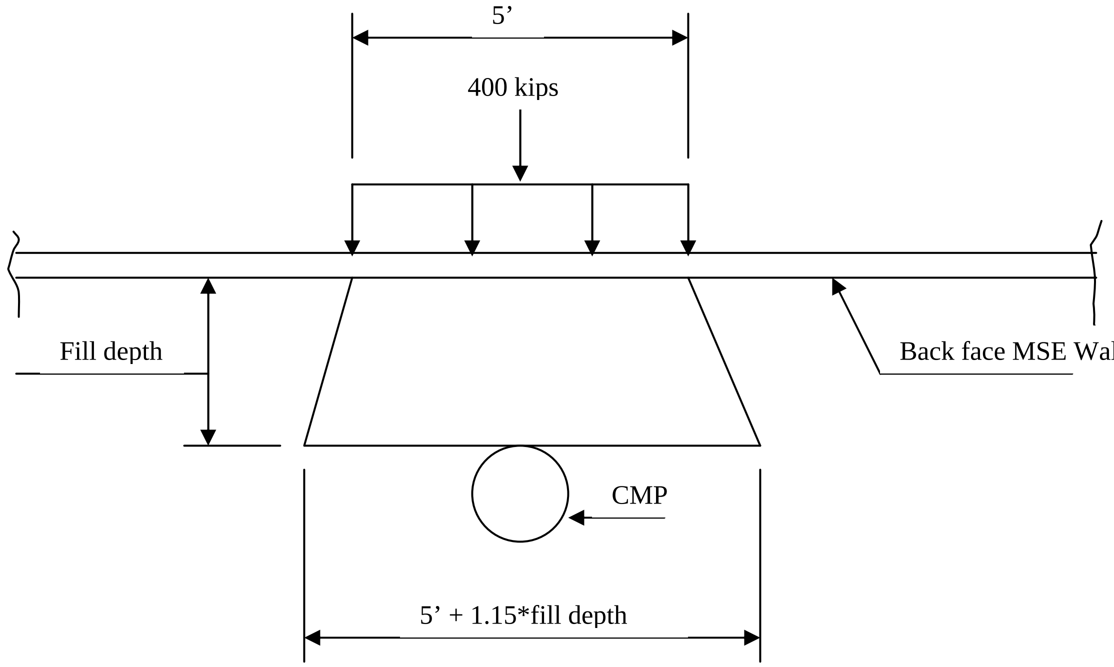
3.6.2 DYNAMIC LOAD ALLOWANCE

The Dynamic Allowance shall be applied only to the design truck or design tandem.

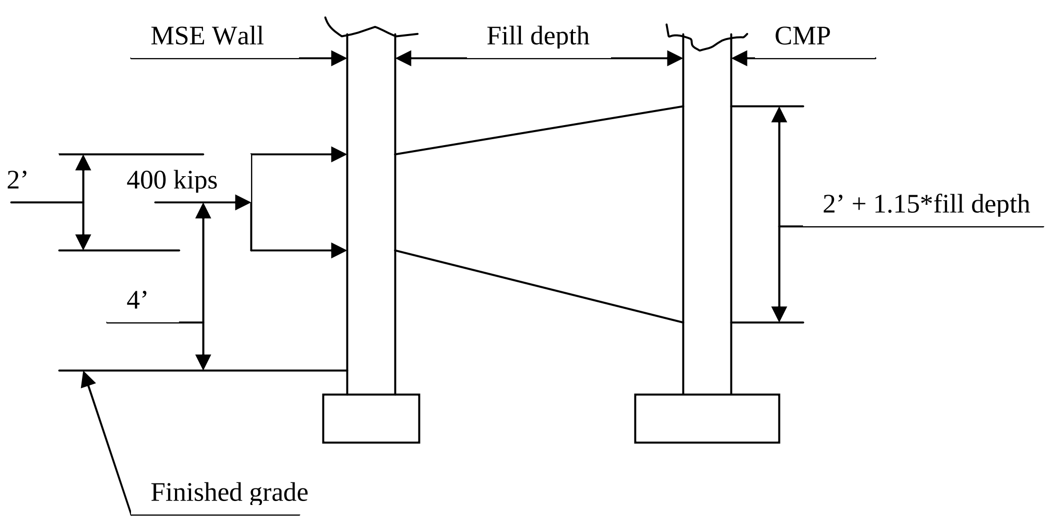

5'
Back face
MSE Wall
Fill Depth
CMP
5' + 1.15*fill depth

3.6.5.2 VEHICLE AND RAILWAY COLLISION WITH STRUCTURES

The provisions of this article shall also apply to columns of spill-through abutments contained within MSE wall fills. The 400-KIP collision force shall be assumed to act perpendicular to the face of the MSE wall at a distance of 4.0 FT above ground. Furthermore, the collision force shall be distributed over a MSE wall area not greater than 5.0 FT wide by 2.0 FT high and distributed through the fill in accordance with Article 3.6.1.2.6, where the fill depth shall be taken as the cover distance measured from the back face of the MSE wall panel to the face of the CMP pipe. The resulting pressure shall be assumed to act on the full diameter of the CMP and applied as a line load acting along the longitudinal axis of the column.



PLAN



ELEVATION

3.7.3 STREAM PRESSURE

The design high water elevation and velocity for the purposes of stream pressure calculations shall be based on Q_{100}.

For structures where the pier is aligned with the stream flow the lateral stream pressure applied to the side of the pier shall be based on an angle of 5° to allow for a change in the direction of flow over the life of the structure.

The average pressure of flowing water acting in the longitudinal or lateral direction on the substructure, p, shall be calculated using equation 3.7.3.1-1 or 3.7.3.2-1 respectively. The stream flow pressure distribution shall be triangular with 2•p located at the top of water elevation and zero pressure located at the flow line.

The stream flow forces shall be computed by the product of the stream flow pressure, taking into account the pressure distribution and the exposed pier area. In cases where the corresponding top of water elevation is above the low beam elevation, stream flow loading on the superstructure shall be investigated. The pressure acting on the superstructure shall be a uniform distribution equal to 2•p along the superstructure.

3.9.2.4.1 PIERS PARALLEL TO FLOW

The friction angle ϕ_f between the pier nose and the ice is defined by the following equation.

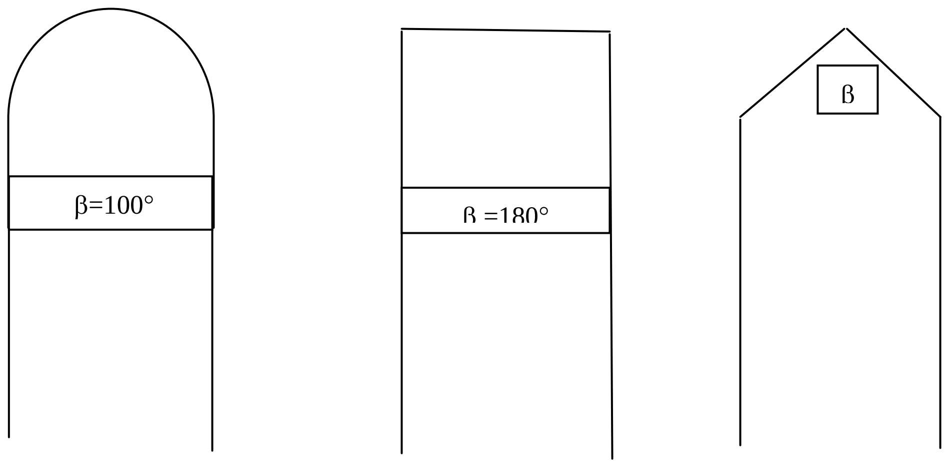
$$\phi_f = \tan^{-1} \mu$$

However, the coefficient of friction μ can not be established with great certainty. The Alberta Research Council uses $\mu = 0.18$ and the Alyeska Pipeline Co uses $\mu = 0.10$.

For most design cases $\phi_f = 0.0$ (which is conservative) should be used unless the loads become unrealistic in which case the following values should be $\mu = 0.10$ and $\phi_f = 5.7°$.

Changing the nose geometry β will have a bigger effect than changing ϕ_f .

β is defined as follows:



Note when β= 180 F_t = 0.

3.10.2 ACCELERATION COEFFICIENT

The velocity-related acceleration coefficient, A_v, for Idaho is shown on the following page. The contour spacing is not linear, but roughly logarithmic. This should be taken into account when interpolating for A_v values.

3.10.2 SEISMIC HAZARD

A site specific procedure shall be used if the site is within 6 miles of an active fault. The Geotechnical Engineer should be contacted to determine the response spectra. Site Specific Procedures are specified in Article 3.10.2.2.

Active fault locations for Idaho are shown on the following page.

Revisions:

April 2008	Revised article to agree with 2008 Interims. Revised map to only show active fault locations.

3.10.5 IMPORTANCE CATEGORIES

If the route that the structure is on services an emergency facility (hospital, firestation, etc.) or a major transportation facility and the detour for the emergency facility is greater than one mile the structure should be classified as critical.

If the structure carries utilities (water, gas, and power) and these utilities would be considered essential in the case of an earthquake or other emergency, the structure should be considered Critical.

All bridges that are classified as critical under section 3.10.5 shall be considered Important for η_I purposes.

Additional References ..\01 Introduction\1.3.5 Operational Importance.doc

Revisions:
April 2008 Changed reference from 3.10.3 to 3.10.5 to agree with the 2008 Interims.

3.12.2.1 TEMPERATURE RANGE

All of Idaho can be assumed to be in the cold climate region. The base construction temperature for bridge structures may be assumed to be 60 degrees Fahrenheit.

The base construction temperature is the assumed temperature at which the structure is initially built. All temperature related movements should be determined based on the difference between the base temperature and the limits of the temperature ranges listed in Article 3.12.2.1.

The design base temperature of 60 degrees is the average of the Mean Daily Temperatures during the construction season of April through October for the following three cities:

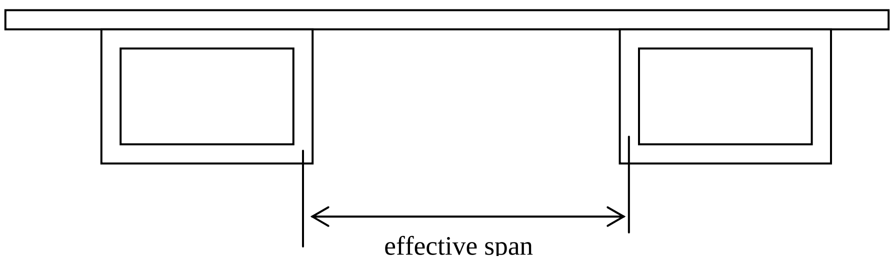
Mean Daily Temperatures from NWS historic climate summaries

	April	May	June	July	August	September	October
Coeur d' Alene	46	55	62	69	68	59	49
Boise	49	58	66	74	73	63	52
Idaho Falls	44	53	61	68	67	57	46

4.6.2.1.6 CALCULATION OF FORCE EFFECTS

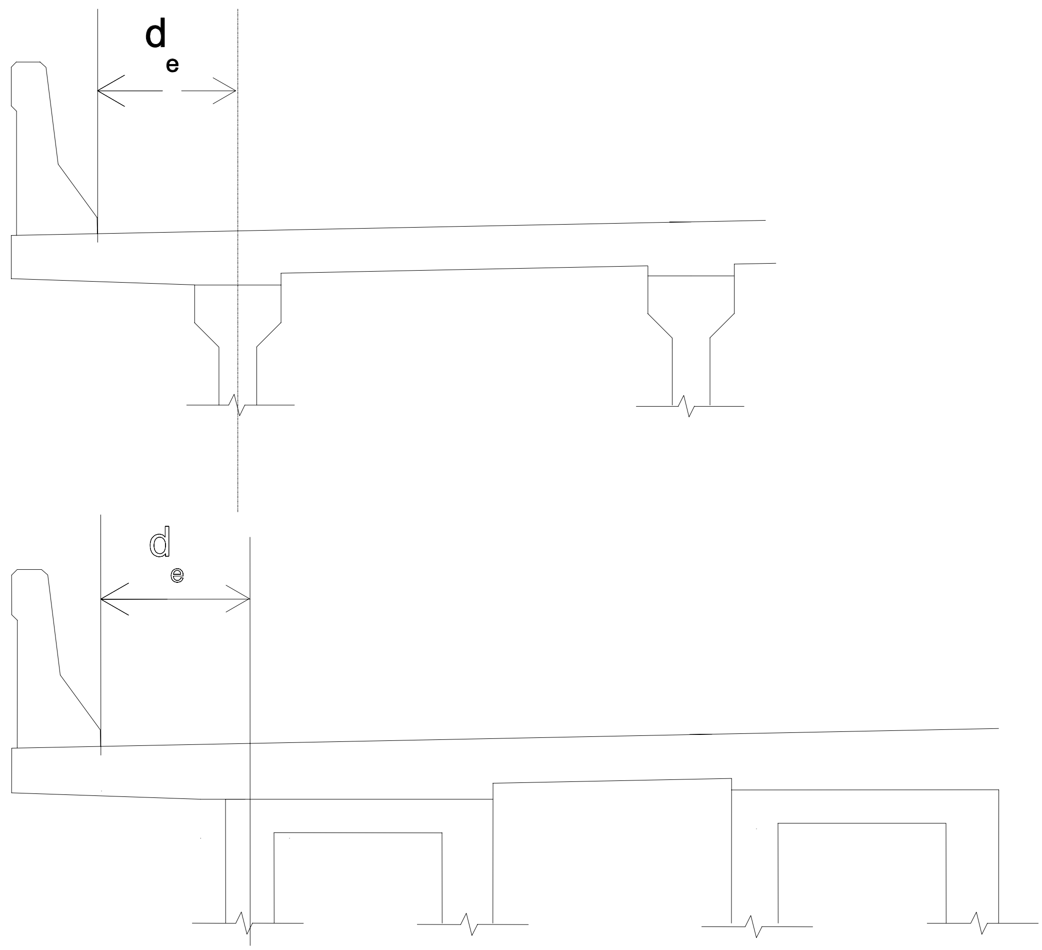
The effective span length for deck design for precast spread box beams shall be as shown below.

(Note minimum will be 4’-0”).



4.6.2.2.1 APPLICATION

The term d_e shall be defined as follows:



4.7.4 ANALYSIS FOR EARTHQUAKE LOADS

4.7.4.1 General

The soil spring constants for piles, drilled shafts, or deep foundations can be modeled using the procedure in the FHWA Manual “Seismic Design of Highway Bridge Foundations” Report No. FHWA/RD-86/101.

Existing structures that require major rehabilitation and are in Seismic Zones 2, 3, and 4 shall be evaluated for seismic retrofit in accordance with the FHWA Manual “Seismic Retrofitting Guidelines for Highway Bridges” Report No. FHWA/RD-83/007.

Revisions:
April 2008 Deleted requirement for multi-mode analysis.

5.4 MATERIAL PROPERTIES

The designer is responsible for selecting the proper class of concrete for the degree of exposure and intended placement. The classes of concrete specified should meet the requirements of Section 502 of the Standard Specifications.

SCHEDULE OF CONCRETE
Concrete placement is divided into schedules for structures that have definite superstructure and substructure elements; e.g., prestressed girder bridges. Unscheduled concrete is used for culverts, retaining walls, etc.

- Schedule Number 1 concrete is intended for placement in the substructure. Typical placement is below the beam seats at abutment and piers and at wing walls.
- Schedule Number 2 concrete is intended for placement in the superstructure. Typical placement is above the beam seats.

CLASS OF CONCRETE
The class of concrete is specified in 100 psi 28-day strength. Refer to Bridge LRFD Manual Article 5.12 for additional requirements on classes of deck concrete.

Class	Strength-psi	
Seal	NA	Use for underwater placement for sealing cofferdams
15	1500	Use for a leveling course
30	3000	Not recommended for structural applications.
40	4000	Recommended for most structural members.
40A	Intended for use when high entrained air and low slump are necessary for extreme exposure and wear. Typical applications are deck slabs, curbs, and parapets.	
40B	Intended for use when air entrained concrete is desirable for substructure elements. Typical applications are abutments, piers, pier caps, columns, and wing walls.	

The properties for structural concrete placed underwater shall be specified by a Special Provision.

ALKALI SILICA REACTIVITY
All coarse and fine aggregate for concrete shall be tested for ASR according to the Standard Specifications subsection 703.02 & 703.03. Aggregates found to be potentially reactive shall require mitigating measures. The mitigative additives may be fly ash, lithium or other additives in any combination. The proposed mix design shall be tested with the mitigative additives.

BID ITEMS
Concrete should be specified as follows:

- Class 40A, Class 40B, and Seal Concrete bid items should be used.
- Do not specify concrete classes with fly ash.

Commentary
Fly ash can not be produced that conforms to ITD Specifications and specifying fly ash on the contract plans will require the Resident Engineer to write a change order deleting it. The contractor may add fly ash to enhance the mix design at his option.

Revisions:

June 2006	Fly ash concrete paragraph deleted and replaced with ASR paragraph to conform to the 2004 ITD Standard Specifications.
April 2008	Added Seal, Class 15, and Underwater structural concrete to Class of Concrete

5.4.3 REINFORCING STEEL

5.4.3.3 Special Applications

The maximum length of #3 and #4 bars shall be 40'. The maximum length of #5 through #18 bars shall be 60'.

5.5.4.2 RESISTANCE FACTORS

Strength Limit State resistance factors shall be as follows:

Strength Limit States	Resistance Factor
Flexure and tension of reinforced concrete	0.90
Flexure and tension of prestressed concrete	1.00
Shear and torsion normal weight concrete lightweight concrete	 0.90 0.70
Axial Compression with spirals or ties	0.75
Bearing on concrete	0.70
Compression in Strut and Tie Models	0.70
Compression in anchorage zones normal weight concrete lightweight concrete	 0.80 0.65
Tension in steel in anchorage zones	1.00
Resistance During Pile Driving	1.00
Compression members with flexure Factored axial load $\geq 0.10 f'_c A_g$ $0.10 f'_c A_g \geq$ Factored axial load > 0 Factored axial load = 0	 0.75 Linear transition 0.90
Partially prestressed members $PPR = A_{ps}f_{py}/(A_{ps}f_{py}+A_s f_y)$	0.90+0.10(PPR)
Extreme Event Limit States	
Extreme Event for Seismic Zones 3 & 4 Compression members with flexure Factored axial load $\geq 0.20 f'_c A_g$ $0.20 f'_c A_g \geq$ Factored axial load > 0 Factored axial load = 0	 0.50 Linear transition 0.90
All other cases	1.00

For Service and Extreme Event Limit States see 1.3.2 Limit States (Resistance Factors)

See article 5.10.11.4.1b for concrete columns in Seismic Zones 3 & 4.

5.7.1 ASSUMPTIONS FOR SERVICE & FATIGUE LIMIT STATES

The following are ITD's preferred assumptions that differ from the LRFD Code:

- The modular ratio, n, should be equal to E_s/E_c and not rounded.
- An effective modular ratio of n should be used for permanent loads and prestress.

Commentary

The computations require more effort to differentiate n and 2n, and the use of 2n for creep effects appears to increase f_s only 1-2%.

5.7.3.3.2 MINIMUM REINFORCEMENT

The cracking moment, Mcr, is defined in Article 5.7.3.6.2. The gross moment of inertia, Ig, is defined in Article 5.3.

5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

The exposure factor, $\gamma_{e,}$ shall be defined as follows:

Class 1 Exposure γ_e = 1.0 kips/inch

Components exposed only to the effects of weather such as pier caps and columns.

Class 2 Exposure γ_e = 0.75 kips/inch

Components exposed to earth fill or submerged in water such as abutments, wings, and footings. Also Deck slabs not designed in accordance with Article 9.7.2.

Revisions:
June 2006 Article revised to comply with the AASHTO 2005 Interims.

5.7.3.6.2 DEFLECTION and CAMBER

The ultimate purpose in computing deflections due to prestressing and dead load is to produce a structure which will conform with the profile grade after the effects of immediate dead load deflection and prestressing and subsequent deflection due to creep have taken place.

In prestressed, precast girders the pretensioned steel strands induce upward deflection or camber. This camber is lessened by the weight of the girder, but not negated. Since concrete is subject to creep, the camber increases with time.

The camber strip is the relatively thin layer of concrete between the underside of the bridge deck and the top of the girder. The purpose of the camber strip is to compensate for the difference between the profile of the underside of the deck along the girder line and the girder itself. The camber strip shall be of sufficient thickness so that the top of the girder does not protrude into the slab.

The thickness of the deck at centerline of bearing shall be shown on the plans. The thickness shall include the following:

- Basic deck thickness
- Camber strip thickness
- Superelevation allowance
- Vertical curve effects
- Horizontal curve effects
- Fabrication/Construction tolerances

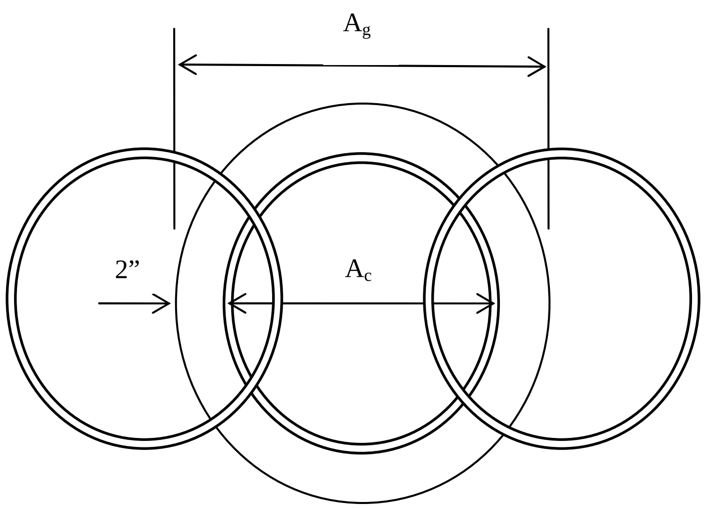
Deflections and camber are assumed to be parabolic curves and shall be computed according to ITD Research Report 98-16-01, Camber Growth In Prestressed Concrete Bridge Girders. The design procedure is outlined in Appendix A, Article A5.6.

Deck Forms
At the time of setting deck forms, the primary concern is that the tops of the girders not extend into the deck and interfere with the reinforcing steel.

After the girders are set, the tops of the girders are profiled. Finish grades for the top of the deck are generated by adding the correction for the slab dead load deflection. The bottom of deck form elevation is computed by subtracting the deck thickness. The top of girder profile should be lower than the bottom of deck form elevation. If not, the profile grade may require adjustment.

5.7.4.6 SPIRALS AND TIES

For calculation of the ratio of spiral reinforcement required by Equation 5.7.4.6-1 for a single column with interlocking spirals, the core area, Ac, shall be based on the out to out core area for one single spiral of the interlocking spirals. The gross area, Ag, shall be based on a theoretical round column having the core area as previously defined and having a theoretical cover of 2” all around the core.



Commentary: From Oregon DOT Bridge Engineering Section Office Practice Manual 2003, Section 5.1.7.5

5.8.3.5 LONGITUDINAL REINFORCEMENT

In equation 5.8.3.5-2 for the required tensile reinforcement at the inside edge of the bearing area of simple end supports to the section of critical shear, the values of V_u, V_s, V_p, and θ calculated at the section of critical shear shall be used. Where there is an embedded steel bearing plate in the girder, the inside edge of the bearing area shall be taken as the inside edge of the embedded steel plate and the theoretical crack is assumed to propagate from the inside edge of the embedded steel plate at an angle of θ, where θ is calculated at the section of critical shear. Refer to Figure C5.8.3.5-1.

For prestressed girders, if the prestressing strands alone are not sufficient to resist the required tensile force, then the embedded steel plate can be lengthened slightly, mild steel can be utilized, or a combination of the two. Any lack of full development in either prestressing strands or the mild reinforcing shall be accounted for. Check development of mild reinforcing from the end of the girder to the point where the reinforcing bars cross the theoretical crack.

At other sections (critical section, 0.1L, 0.2L, etc.), the tensile capacity of the tensile reinforcement on the flexural tension side of the member shall be calculated using equation 5.8.3.5-1. The values of M_u, N_u, V_u, V_s, V_p, and θ shall be the values that are calculated for the section under consideration.

Commentary: At sections other than at the end, the values of M_u, N_u, V_u, V_s, V_p, and θ are assumed to apply over a bandwidth that is centered on the section under consideration. At the end, the bandwidth is taken as the distance from the inside edge of the bearing area to the section of critical shear. The code is not clear regarding what values of M_u, N_u, V_u, V_s, V_p, and θ apply over the bandwidth at the end. The commentary allows using V_u, V_s, V_p, and θ calculated at the section of critical shear over this bandwidth (Refer to C5.8.3.5.). M_u is taken as 0.0 ft-kips over this bandwidth. For uniformity within the Bridge Section, we have chosen to use the method in the commentary.

5.9.4.1.2 TENSION STRESSES

Temporary tensile stresses in prestress concrete before losses in all areas shall be limited to:

$$0.0948\sqrt{f'ci} \leq 0.2ksi$$

Reinforcing bars shall not be used to resist the tensile force in the concrete.

5.9.4.2.2 TENSION STRESSES

Tensile stresses in the top slab of post-tension box girders in all areas shall be limited to 0 ksi tension. Tensile stresses in the bottom slab of post-tension box girders in all areas shall be limited to $0.0948\sqrt{f'c}$.

Revisions:
April 2008 Added allowable tension in bottom slab of box girders.

5.9.5 LOSS OF PRESTRESS

Elastic Losses or Gains, Δf_{pES}

Δf_{pES} is the sum of all the losses or gains to the strand stress due to elastic shortening or extension caused by either internal (prestressing) or external (gravity) loads applied to the concrete section.

Δf_{pES} should be calculated using either the iteration of Equation 5.9.5.2.3a-1 or directly solving with Equation C5.9.5.2.3a-1.

Long–Term Losses, Δf_{pLT}

Δf_{pLT} is the sum of losses due to long-term shrinkage and creep of the concrete, and relaxation of the prestressing steel.

There are 3 methods listed in the AASHTO LRFD code for computation of long-term losses:

1. The approximate estimate according to the provisions of Article 5.9.5.3 and Equation 5.9.5.3-1.

2. The approximate estimate according to the provisions of Article 5.9.5.3 and Table 5.9.5.3-1.

3. The refined estimates according to the provisions of Article 5.9.5.4.

Method 1 is an approximate method that gives reasonably good results (as compared to Method 3) for conventional prestressed girders with a composite cast-in-place deck; however this method can be unconservative for girders without a cast-in-place deck.

Method 2 is the least accurate method and should not be used for final design.

Method 3, the refined method, presumably results in the most accurate estimate of long-term losses for all prestressed girder types with or without a cast-in-place deck, however it requires significantly more effort than either Method 1 or 2. For typical bridge construction with a conventional prestressed girder and a cast-in-place deck, Method 3 will not yield significantly different or better results as compared to Method 1. It is the Bridge Section's opinion that for conventional prestressed girders with cast-in-place deck the presumed level of accuracy achieved by using Method 3 is seldom warranted or even possible because the material properties that effect creep and shrinkage and the times for various load application are either unknown or beyond the control of the bridge designer.

In general, Method 1 may be used for typical ITD bridge construction that involves precast, pretensioned members with a composite cast-in-place slab. Method 3 may be used for all prestressed girder types but must be used for precast, pretensioned members without a composite slab (such as deck bulb tees, side-by-side voided slab sections, etc) the use of Method 1 in this case would be unconservative. Method 2, if used at all, should be limited to preliminary designs only, not for final design.

The term "construction staging" as used in C5.9.5.3 is understood to refer to a member that is constructed in stages rather than a bridge that is constructed in stages. Examples of members that are constructed in stages are segmental construction and post-tensioned spliced precast girders.

Design Assumptions for Method 3 (refined estimate of losses)

It is ITD policy to use 55% humidity for all locations in Idaho.

When using the refined estimate of losses (Method 3) there are some assumptions that need to be made concerning the age of the girder concrete (ageing starts at the end of the cure period) at transfer, at deck placement and at final time. Also the age of the deck concrete is also required for the determination of losses or gains due to loads applied to the composite section (such as rail loads and wearing surfaces). Because these ages are not usually known during the design process some conservative assumptions should be made.

The sooner a cast-in-place deck can be placed on prestressed girders the lower the long term prestress loss will be. Consequently the design should be based on the deck being placed at the latest practical time; this will result in the greatest long term losses and thereby produce a more conservative design. Therefore, if no better information is known, it is ITD policy to assume the following:

For Prestressed Girders with Composite cast-in-place Decks,

Age of girder at transfer = 0.75 days (a smaller number will result in greater prestress loss)
Age of girder at deck placement = 270 days (a larger number will result in greater prestress loss)
Age of deck at composite loading = 1 day (a smaller number will result in greater prestress loss)
Final time = 3650 days (a smaller number will result in less prestress loss, a larger number has little effect)

For Prestressed Girders without cast-in-place Decks,

Age of girder at transfer = 0.75 days (a smaller number will result in greater prestress loss)
Age of girder at installation = 15 days (a smaller number will result in greater prestress loss)
Final time = 3650 days (a smaller number will result in less prestress loss, a larger number has little effect)

The assumed values above were chosen to be conservative in most cases. If the designer has information to more accurately predict the number of days for the various conditions listed above different design values may be used in consultation with the Group Leaders.

Commentary:

The total loss of prestress, Δf_{pT}, is defined as the difference in the stress in prestressing strands immediately before transfer (the jacking stress) and the effective stress in the prestressing strands after all losses, f_{pe}. The total loss of prestress, Δf_{pT}, is further defined as the sum of elastic losses and long-term (also called time-dependent) losses. For typical pretensioned, precast members with low-relaxation strand, the above definitions can be re-stated as the following equations:

$f_{pe} = (0.75 * 270 \text{ ksi}) - \Delta f_{pT}$ Where 0.75 * 270 ksi is the jacking stress.

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ (Eqn. 5.9.5.1-1) Where Δf_{pES} is the elastic loss and Δf_{pLT} is the long-term loss.

Δf_{pES} , elastic losses or gains, occur instantaneously at the time of load application. While Δf_{pES} consists of many different components, one for each load, there are two general categories. The first category is the elastic loss that occurs immediately after transfer of the prestress force to the girder (loss due to prestress combined with gain due to the member self weight). The second category is the loss or gain due to additional loads, such as non-composite dead loads, composite dead loads and live loads that are applied after transfer. It is ITD's policy to ignore the effects of this second category for simplicity and the fact that it is conservative to due so because in most cases this effect would result in a gain or increase in prestressing force.

Δf_{pLT} , long term losses, occur due to long-term shrinkage and creep of concrete, and relaxation of the prestressing steel.

Sign Convention

When loads or prestressing forces cause a member to shorten or the strands are allowed to "relax", the tension stress in the strands decreases and the strands undergo a loss. A loss is considered to be a positive quantity. When loads cause the strands to elongate, the tension stress in the strand increases and the strands undergo a gain. A gain is considered to be a negative quantity. For typical construction, the losses will be greater than the gains, and the sum of losses and gains will result in an overall loss. Δf_{pT} will typically be a positive quantity that is subtracted from the jacking stress.

Section Properties

The AASHTO LRFD code allows the bridge designer to use either gross or transformed section properties when designing prestressed girders. Gross section properties are the properties of the girder section based only on the gross concrete area without regard to the area of prestressing steel or other reinforcement. The advantages of gross section properties are that they are simple to compute and they do not vary along the girder length with the eccentricity of harped strands.

Transformed section properties are those properties that take into account the effective area of the prestressing steel as well as any other longitudinal reinforcement in the girder based on the steel-to-concrete modular ratio. The use of transformed section properties does have the advantage of implicitly accounting for all elastic losses or gains due to applied loads including prestress, dead load and live load. The use of transformed section properties is technically more correct and should yield a more accurate estimate of stresses in both the prestressing steel and the concrete.

When gross section properties are used to analyze prestressed girders the typical analysis assumes that the prestressing force in the girder remains constant during all loading conditions. However this is not actually true since the prestressing strands are bonded to the concrete and if plane sections remain plane the stress in the strands (and therefore the prestressing force) varies with girder loading. Gross section analysis does not account for this change in prestress force directly. Therefore, to account for this effect the prestressing force has to be adjusted by calculating the elastic losses or gains and subtracting or adding them to the initial force.

It should be noted that if transformed section properties are used for the structural analysis of the girder there is no need to consider elastic losses or gains in the prestressing strands, the change in the strand stress is already accounted for implicitly because of the adjusted section properties. This effect can be understood with the simple case of a 10" by 10" rectangular concrete section with 1.5 square inches of prestressing steel located at the center of the section with a steel-to-concrete modular ratio (Es/Ec) of 7.0. Assuming the force at transfer is 0.75*270 ksi the initial prestress force on the section is 0.75*270*1.5 = 303.75 kips. If transformed properties are used the section area becomes 10*10 + 1.5(7.0 – 1) = 109 sq. in., dividing the total initial force by this area results in a compressive stress over the total transformed area of 303.75/109 = 2.787 ksi. Since the transformed area of the steel is part of the total area that the force is applied to then subtracting this compressive stress times the modular ratio from the original tension stress in the steel gives the net stress in the steel, 0.75*270 – 2.787*7.0 = 182.99 ksi. Now if gross section properties had been used for the analysis then to find the effective stress in the steel the elastic losses must be calculated with Equation C5.9.5.2.3a-1, the result for this example is a loss of 19.24 ksi. The effective stress in the steel is then, 0.75*270 – 19.24 = 183.26 ksi. The effective stress from the gross section analysis is essentially the same, well within the level of accuracy of the design material properties.

ITD has historically used gross section properties for prestressed girder design, ignoring all gains in the prestressing force due to added dead load and live load. As a result we have a good history of crack free prestressed girder performance in Idaho. It is ITD policy to continue to use gross section properties ignoring added dead load and live load elastic gains.

Humidity

The average annual humidity in Idaho can vary from 70% in the far north to as low as 55% in the southwest part of the state. Since concrete shrinks more in a low humidity environment resulting in greater prestress loss it is conservative to assume a low humidity in designing prestressed girders, therefore it is ITD policy to use 55% humidity in design unless the designer has better information for a particular bridge location.

Revisions:
April 2008 Added new article.

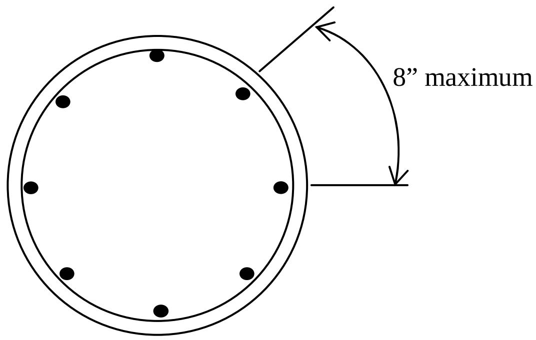
5.10.10.1 FACTORED BURSTING RESISTANCE - PRETENSIONED ANCHORAGE ZONES

In this section, the prestressing force at transfer is to be taken as the prestressing force *just after* transfer. This means that the loss due to elastic shortening should be subtracted out.

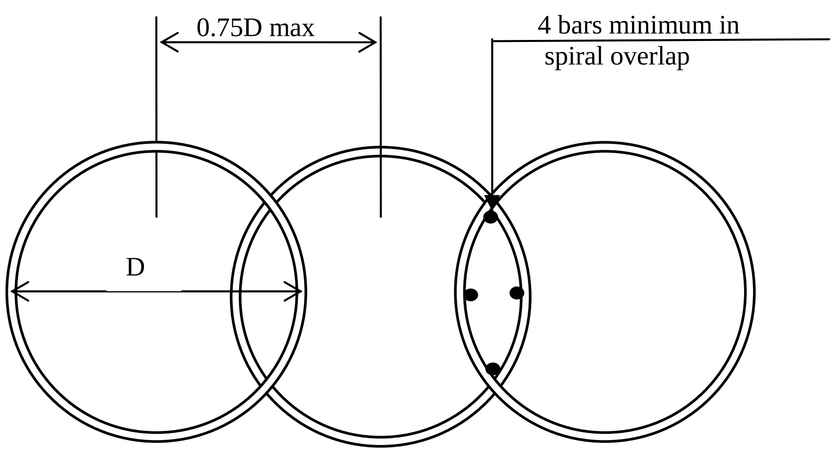
For example, for low-lax strands the stress in the strand equals 0.75*fpu – Δfp_{ES} where Δfp_{ES} is computed per section 5.9.5.2.3.

5.10.11.4.1d TRANSVERSE REINFORCEMENT FOR CONFINEMENT AT PLASTIC HINGES

The commentary for this article specifies a maximum spacing of 8” center-center of longitudinal bars.



Interlocking spirals shall be used for non-circular columns. Reinforcement details are shown in the following sketch.



5.12 DURABILITY

5.12.1 General

All State and Locally sponsored projects shall be required to provide a deck protection system in the design and construction on all new concrete bridge decks, regardless of the winter roadway maintenance salt policy. Both mats of reinforcing steel should be considered for epoxy coating in deck slabs that will be carrying high volumes of traffic and will be subjected to frequent winter salting applications. Since both high traffic and frequent salting will occur primarily in urban areas, all structures located in urban areas shall be evaluated for a dual protection system.

A Single Deck Protection System is the minimum acceptable deck protection for decks exposed to traffic on all interstate, state, or county highways unless additional deck protection is required. The single deck protection system shall meet the requirements listed below. The type of deck protection system shall be shown or noted on the Situation and Layout Drawing.

EXPOSED DECK SLAB

The deck slab is considered exposed to traffic when the distance between the finished grade and the top of the concrete deck is less than 4 inches between the paved roadway shoulders.

SINGLE DECK PROTECTION SYSTEM

- The concrete deck shall have an 8-inch minimum thickness of Class 40A concrete, which includes a ½ inch expendable wearing surface that is considered as added dead load and not having structural capabilities.
- The top mat of reinforcement shall have 2½ inches of cover.
- All reinforcement within 4 inches of surfaces exposed to traffic shall be epoxy coated, including concrete parapets.

DUAL DECK PROTECTION SYSTEM

A dual deck protection system shall be utilized for all structures requiring special construction techniques or that have been classified as major or unusual bridges. Any structure that will require shoring for removal and repair of the deck (e.g. CIP box girders, CIP tee beams, CIP slab bridges) shall have a dual protection system. Deck slabs on box girder bridges are difficult and costly to repair unless the deck is designed so a portion of the deck can be removed without requiring shoring, such as the Type 1 dual deck protection system.

The dual deck protection system shall meet the requirements listed for one of the 3 following types. The type selected will require the approval of the Bridge Design Engineer prior to incorporating that system in the design.

TYPE 1

- The concrete deck shall have a 7½ inch minimum thickness of Class 40A concrete, which does not include the 1½ inch of replaceable wearing surface that is considered as added dead load and not having structural capabilities.
- The top mat of reinforcement shall have 1¾ inches initial cover (before scarification of ¼ inches).
- The deck shall be designed so that the top 1½ inches can be removed without requiring shoring while maintaining traffic on a portion of the deck. The replaceable wearing surface of 1½ inches shall be latex modified concrete or micro silica modified concrete.
- All reinforcement within 4 inches of surfaces exposed to traffic shall be epoxy coated, including concrete parapets.

TYPE 2

- The concrete deck shall have an 8½-inch minimum thickness of Class 40A concrete, which includes a 1-inch expendable wearing surface that is considered as added dead load and not having structural capabilities.
- The top mat of reinforcement shall have 3 inches of cover.
- All reinforcement within 4 inches of surfaces exposed to traffic shall be epoxy coated, including concrete parapets.

TYPE 3

- Bridges using precast prestressed boxes or slabs as the deck to support traffic shall use the Type 3 Dual Deck Protection System. The concrete class and member sizes for precast, prestressed deck members shall be determined by design.
- The top mat of reinforcement shall have 2½ inches of cover.
- The top surface of precast beams that act as the bridge deck shall have an asphalt overlay of 1¾ inches with an interlayer waterproofing membrane. Prior to placement of the membrane, all joints between precast units that exceed ¼ inch difference in elevation shall be over grouted to accommodate hand finishing. The grouted slopes

shall not be greater than 1:12.
- All reinforcement within 4 inches of surfaces exposed to traffic shall be epoxy coated, including concrete parapets.

BURIED DECK SLABS

The deck slab is considered buried when the distance between the finished grade and the top of the concrete deck is greater than 4 inches between the paved roadway shoulders. This generally applies to box culverts and stifflegs where roadway ballast is carried over the top slab.

UNDER ROADWAY WITH LESS THAN 2 FEET FILL
- The deck slab shall be Class 40A concrete with a waterproof membrane applied as specified in Section 511 of the Standard Specifications.
- The top mat of reinforcement shall have 2½ inches of cover.
- Both mats of reinforcement in the deck slab shall be non-epoxy coated bars.

OUTSIDE ROADWAY OR FILL EXCEEDS 2 FEET
- The deck slab shall be Class 40B concrete.
- The top mat of reinforcement shall have 2 inches of cover.
- Both mats of reinforcement in the deck slab shall be non-epoxy coated bars.

ELECTRICAL CONNECTION TO BRIDGE DECK REINFORCEMENT

The following is standard practice for locating electrical test connection points on the edges of new concrete bridge decks. CADDS cell on B5.7 is available for use on the plans.

PLAIN & EPOXY COATED REINFORCEMENT

Each deck section shorter than 250’ should have two connection points. These connections should be located at the extreme ends of the deck section, within 3’ of the ends of the deck section. The connections should be located on opposite sides of the deck.

On decks greater than 250’, electrical connection points should be evenly spaced, but no more than 250’ apart. Locations should alternate between opposite edges of the deck. Spacing should be adjusted so that there is a connection point within 3’ of each end of the deck section.

A deck section is defined as a portion of deck where the longitudinal reinforcement is continuous throughout. The section ends at any point where there is a transverse joint if the longitudinal reinforcement does not pass through the joint. At a construction joint, where longitudinal steel is carried through the joint, the deck on both sides of the joint is part of a single, continuous deck section for electrical testing purposes.

CATHODIC PROTECTION

On decks designed for cathodic protection, connection points shall be installed in the same locations as for ordinary reinforcement. In addition, some means must be provided to break the cathodic protection circuit so that readings can be taken with the protective current shut off.

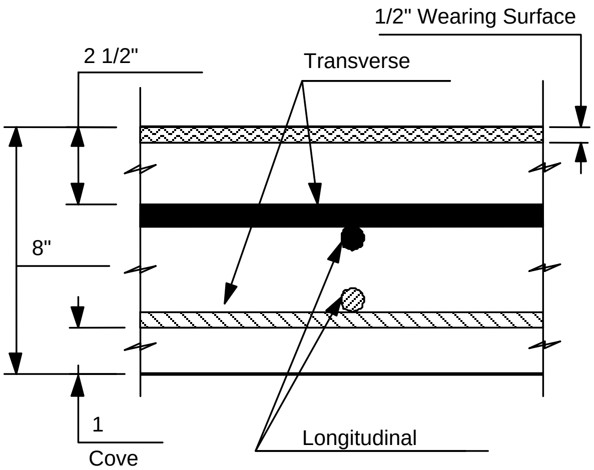
PRECAST PRESTRESSED SLABS

Electrical connection points are not required for precast, prestressed slab bridges and deck bulb-tee bridges.

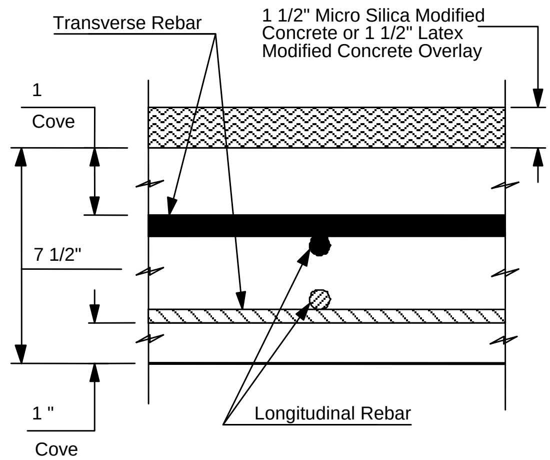
Revisions:

June 2006	Added paragraph for Electrical Connections to Bridge Decks that was in the ITD Bridge Metric/English Manuals.

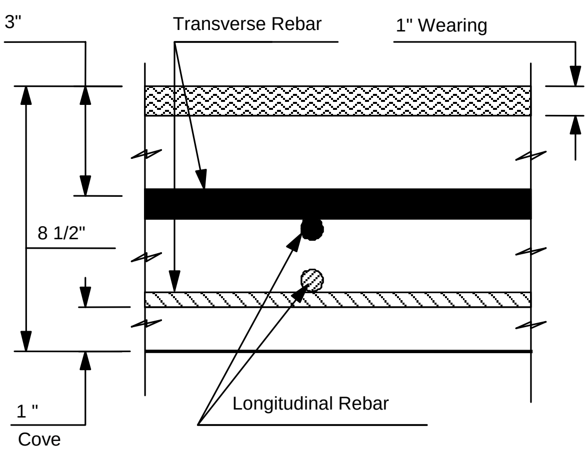
DECK PROTECTION SYSTEM DETAILS FOR NEW CONCRETE BRIDGE DECKS



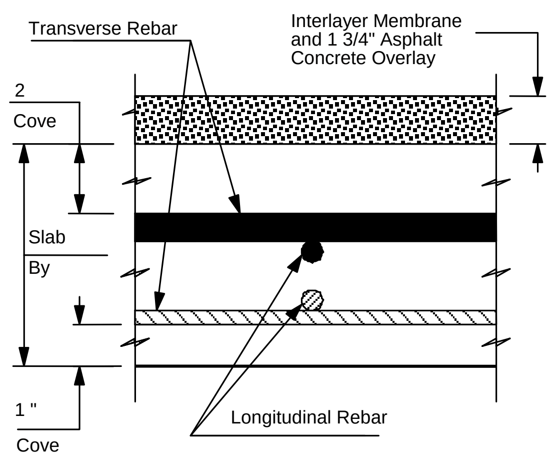
SINGLE PROTECTION SYSTEM



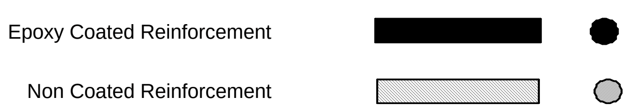
TYPE I
DUAL PROTECTION SYSTEM



TYPE 2
DUAL PROTECTION SYSTEM



TYPE 3
DUAL PROTECTION SYSTEM



5.13.2.2 DIAPHRAGMS

AASHTO & Bulb Tee Girders
Permanent concrete diaphragms or end beams shall be placed at the ends of each span. Intermediate concrete diaphragms shall be placed as follows:

Span Length	Diaphragm Location
40’ – 80’	Midspan
80’ – 120’	⅓ points
> 120’	¼ points

Deep Bulb Tee Girders
66 inch and deeper bulb tee girders shall have temporary diaphragms installed between the exterior and first interior girder. These diaphragms shall be placed midway between all permanent diaphragms as well as the end beams before pouring the permanent diaphragms and end beams. Temporary diaphragms shall be removed after the deck overhang brackets have been removed.

Expansion Ends
To facilitate jacking for future repairs at simply-supported ends of precast, prestressed girder bridges, the end diaphragms shall be designed for a minimum clearance of 10 inches above the level of the beam seats on piers and abutments. No extra reinforcement is required beyond that provided for normal dead, live and impact loads.

Continuous Ends
The end of girders in bridges designed without expansion joints shall be cast into full-depth end diaphragms.

5.14.1.2.5 CONCRETE STRENGTH

This provision of the code shall not be used without the prior approval of the Bridge Engineer and Materials Engineer.

Commentary

Since most of the precast elements that are used are steam cured and the mix designs given in the ITD Standard Construction Specifications has replaced cement with fly-ash at a greater rate than one to one, the values as listed in this specification will not be correct.

5.14.1.2.7 SIMPLE SPAN PRESTRESSED GIRDERS MADE CONTINUOUS

Negative Moments

The following design criteria shall apply for the design of the negative moment steel in the deck.

Case 1 Girders Designed As Simple Spans & Reinforcement Added To Control Cracking.

(This is the typical design case for new bridges)

- The top longitudinal deck reinforcement over the piers, in addition to the standard empirical design reinforcement, shall be #6 bars at 12" spaced between the standard top deck bars. The full area of the additional reinforcement shall extend either direction from the bearing centerlines a distance of at least 10 feet or 15% of the longer span whichever is greater. At least half the area of the additional reinforcement shall extend to a distance from the pier of at least 15 ft or 20% of the longer span whichever is greater.
- The bottom longitudinal deck reinforcement shall be per the empirical design method, Article 9.7.2.

Case 2 Girders Designed Fully Continuous For Live Load

(This case may be used for new bridges where Case 1 is not feasible)

- The weight of the parapet may be distributed evenly to all the girders.
- The maximum negative moment may be taken at the face of the support.
- Longitudinal reinforcement shall be anchored in accordance with Article 5.14.1.2.7b.
- The section shall meet Strength 1 limit state requirements as follows:

  $1.25*DC_{noncomposite}$, acting as simply supported and

  $1.25*DC_{composite} + 1.5*DW + 1.75*LL$, acting as a continuous beam.
- Both layers of steel in the deck may be used in resisting the negative moments.
- Only structures with fully effective construction joints at the piers, per Article 5.14.1.2.7c, shall be designed as continuous for the Service III limit state. This may be accomplished by placing the pier diaphragm concrete and the negative moment regions of the deck first and the positive moment regions only after the pier diaphragm concrete has reached 100% of its design strength.

Case 3 Girders Designed As Simple Spans And The Deck Is Replaced To Eliminate Joints.

(This case is used for existing bridges that are retrofitted)

- No design is required. Use #5 bars in the top mat at approximately 12" spacing over the piers and carry them at least a development length past the face of the support but not beyond the 1/4 pt.

Commentary

Case 1 – The additional reinforcement is based on providing enough steel in the deck over the piers to limit the top deck steel stress to 36 ksi, the maximum allowed for serviceability. This is based on the assumption that all girders act together as a single unit, as in deflection calculations, and that the distance between the centerlines of bearings for each of the spans on a bent is at least 24 inches. The 24 inches between bearings is assumed to act as a short span between the main spans with a moment of inertia of a cracked section. The additional deck steel along with the steel provided in the empirical design method may not supply enough negative moment capacity to resist the full negative moments that would be realized in continuous girder design, however it will provide enough restraint to limit the steel stress to 36 ksi under service loads for all prestressed girder depths up to 72".

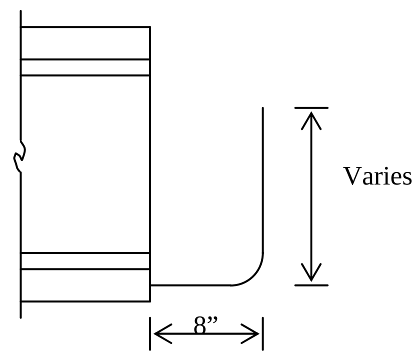
Case 2 – If the negative moment areas of the deck are placed first it may be assumed that the provisions of Article 5.14.1.2.7c are met. However, the deck reinforcement over the piers will then need to be designed for the negative moments at the piers generated by the concrete placed in the positive moment areas of the spans in addition to the composite loads. Otherwise the calculations of Article 5.14.1.2.7c will need to be provided in order to utilize continuity for Service III.

5.14.1.4 SIMPLE SPAN PRESTRESSED GIRDERS MADE CONTINUOUS

The following design criteria shall apply for the design of simple span prestressed girder bridges constructed with a continuous concrete deck (no expansion joints at the piers).

Case 1 Girders Designed As Simple Spans & Top Deck Reinforcement Added To Control Cracking.
(This is the typical design case for new bridges)

- The top longitudinal deck reinforcement over the piers, in addition to the standard empirical design reinforcement, shall be #6 bars at 12" spaced between the standard top deck bars. The full area of the additional reinforcement shall extend either direction from the bearing centerlines a distance of at least 10 feet or 15% of the longer span whichever is greater. At least half the area of the additional reinforcement shall extend to a distance from the pier of at least 15 ft or 20% of the longer span whichever is greater.
- The bottom longitudinal deck reinforcement shall be per the empirical design method, Article 9.7.2.
- A positive moment connection at the piers shall be provided. This shall entail extending the required number of prestressing strand beyond the end of each girder 8" horizontally then bent vertical and anchored in the pier diaphragm. See details on standard drawing for Prestressed AASHTO Girder Details B5.2D or Prestressed Bulb Tee Girder Details B5.3O.



Case 2 Girders Designed Fully Continuous For Live Load
(This case may be used for new bridges where Case 1 is not feasible and approved by the Bridge Engineer)

- The weight of the parapet may be distributed evenly to all the girders.
- The negative moment reinforcement shall be determined based on the maximum negative moment taken at the face of the support.
- Longitudinal reinforcement shall be anchored in accordance with Article 5.14.1.2.7b.
- The section shall meet Strength 1 limit state requirements as follows:
  $1.25*DC_{noncomposite}$, acting as simply supported and
  $1.25*DC_{composite} + 1.5*DW + 1.75*LL$, acting as a continuous beam.
- Both layers of steel in the deck may be used in resisting the negative moments.
- Only structures with fully effective construction joints at the piers, per Article 5.14.1.4.3, shall be designed as continuous for the Service III limit state. The requirements of Article 5.14.1.4.3, may be considered satisfied if the girder age is at least 90 days at the time the continuity diaphragm is placed.
- A positive moment connection at the piers shall be provided with a minimum capacity, Mn, of $1.2M_{cr}$ in accordance with NCHRP Report 519.

Case 3 Girders Designed As Simple Spans And The Deck Is Replaced To Eliminate Joints.
(This case is used for existing bridges that are retrofitted)

- No design is required. Use #5 bars in the top mat at approximately 12" spacing over the piers and carry them at least a development length beyond the centerline of bearing of the girders but not beyond the 1/4 pt. of the span.

Commentary

Case 1 – The additional reinforcement in the top of the deck is based on providing enough steel over the piers to meet the requirements for control of cracking in the deck per Article 5.7.3.4 assuming the total Service 1 live load is distributed equally to all girders. This additional deck steel may not supply enough negative moment capacity to resist the full negative moments that would be required in a continuous girder design for Strength 1 loading, however it will provide enough reinforcement to limit the steel stress requirements for crack control for spans up to 130' and girder depths up to 73.5". Girders not specifically designed for continuous action should still be provided with a positive moment connection in order to prevent possible cracking in the diaphragm and also to securely anchor the girders in the diaphragm. The number of prestressing strands that are extended into the pier diaphragm is based on the area of steel required to provide a moment capacity, M_n, sufficient to resist $0.6M_{cr}$ for the composite girder section in positive bending based on $M_{cr} = Sf_r$ where S is the composite section modulus and f_r is the modulus of rupture equal to $0.24f`_c$ where $f`_c$ is the strength of the diaphragm concrete. ITD's maximum slab thickness and the maximum effective composite slab width per Article 4.6.2.6.1 were used to establish the number of strands required for the various girder depths as well as development length. However, the number of strand and the development length shown on the standard drawing may be considered adequate.

Case 2 – Since the effects due to creep and shrinkage are highly dependent on conditions beyond the control or knowledge of the designer, it is more practical to allow the girders to age for 90 days before continuity is established in order to negate the effects of creep and shrinkage rather than attempting to calculate these effects based on assumed conditions and material properties that can vary significantly.

Revisions:

June 2006 Revised Article to conform to approved 2006 Ballot Item #13.

REBAR AREAS AND SPACING

SPACING	#3	#4	#5	#6	#7	#8	#9	#10	#11
4	0.33	0.60	0.93	1.32	1.80	2.37	3.00	3.81	4.68
4.5	0.29	0.53	0.83	1.17	1.60	2.11	2.67	3.39	4.16
5	0.26	0.48	0.74	1.06	1.44	1.90	2.40	3.05	3.74
5.5	0.24	0.44	0.68	0.96	1.31	1.72	2.18	2.77	3.40
6	0.22	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
6.5	0.20	0.37	0.57	0.81	1.11	1.46	1.85	2.34	2.88
7	0.19	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67
7.5	0.18	0.32	0.50	0.70	0.96	1.26	1.60	2.03	2.50
8	0.17	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34
8.5	0.16	0.28	0.44	0.62	0.85	1.12	1.41	1.79	2.20
9	0.15	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08
9.5	0.14	0.25	0.39	0.56	0.76	1.00	1.26	1.60	1.97
10	0.13	0.24	0.37	0.53	0.72	0.95	1.20	1.52	1.87
10.5	0.13	0.23	0.35	0.50	0.69	0.90	1.14	1.45	1.78
11	0.12	0.22	0.34	0.48	0.65	0.86	1.09	1.39	1.70
11.5		0.21	0.32	0.46	0.63	0.82	1.04	1.33	1.63
12		0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56
13		0.18	0.29	0.41	0.55	0.73	0.92	1.17	1.44
14		0.17	0.27	0.38	0.51	0.68	0.86	1.09	1.34
15		0.16	0.25	0.35	0.48	0.63	0.80	1.02	1.25
16		0.15	0.23	0.33	0.45	0.59	0.75	0.95	1.17
17		0.14	0.22	0.31	0.42	0.56	0.71	0.90	1.10
18		0.13	0.21	0.29	0.40	0.53	0.67	0.85	1.04

TENSION DEVELOPMENT LENGTH
Grade 60 reinforcement; 4000 psi concrete
5.11.2.1.1 TENSION DEVELOPMENT LENGTH

MODIFICATION FACTORS FOR OTHER PLAIN BARS

(1)	Lateral c-c spacing >= 6" & cover along spacing >= 3"	0.80
(2)	Enclosed by spiral with diam > 0.25" & pitch <= 4"	0.75

PLAIN BARS - OTHER

	3	4	5	6	7	8	9	10	11	14	18
	Development Length - inches										
Basic Ldb	12	12	15	18	23	30	38	48	59	81	105
(1)	12	12	12	15	18	24	30	39	47	65	84
(2)	12	12	12	14	17	23	29	36	44	61	79
(1) & (2)	12	12	12	12	14	18	23	29	36	49	63

MODIFICATION FACTORS FOR TOP PLAIN BARS

(1)	Lateral c-c spacing >= 6" & cover along spacing >= 3"	0.80
(2)	Enclosed by spiral with diam > 0.25" & pitch <= 4"	0.75
	Top Bar	1.40

PLAIN BARS - TOP

	3	4	5	6	7	8	9	10	11	14	18
	Development Length - inches										
Basic Ldb	13	17	21	26	32	42	53	67	82	114	147
(1)	12	14	17	21	26	34	42	54	66	91	118
(2)	12	13	16	19	24	32	40	51	62	86	111
(1) & (2)	12	12	13	16	19	25	32	41	50	69	89

TENSION DEVELOPMENT LENGTH
Grade 60 reinforcement; 4000 psi concrete

MODIFICATION FACTORS - OTHER EPOXY BARS

(1)	Cover <= 3*Db	1.50
(2)	Cover > 3*Db	1.20
(3)	Clear spacing <= 6*Db	1.50
(4)	Clear spacing > 6*Db	1.20
(5)	Lateral c-c spacing >= 6" & cover along spacing >= 3"	0.80
(6)	Enclosed by spiral with diam > 0.25" & pitch <= 4"	0.75

EPOXY BARS - OTHER

	3	4	5	6	7	8	9	10	11	14	18
3*Db	1.125	1.500	1.875	2.250	2.625	3.000	3.384	3.810	4.230	5.079	6.771
6*Db	2.250	3.000	3.750	4.500	5.250	6.000	6.768	7.620	8.460	10.158	13.542
	Development Length - inches										
(1)	14	18	23	27	34	45	57	72	88	122	158
(2)	12	15	18	22	27	36	45	58	71	98	126
(3)	14	18	23	27	34	45	57	72	88	122	158
(4)	12	15	18	22	27	36	45	58	71	98	126
(1) & (5)	12	15	18	22	27	36	45	58	71	98	126
(2) & (5)	12	12	15	18	22	29	36	46	57	78	101
(1) & (6)	12	14	17	21	26	34	43	54	66	92	119
(2) & (6)	12	12	14	17	21	27	34	43	53	73	95
(3) & (6)	12	14	17	21	26	34	43	54	66	92	119
(4) & (6)	12	12	14	17	21	27	34	43	53	73	95
(1) & (5) & (6)	12	12	14	17	21	27	34	43	53	73	95
(2) & (5) & (6)	12	12	12	13	17	22	27	35	43	59	76

MODIFICATION FACTORS - TOP EPOXY BARS

(1)	Cover <= 3*Db	1.50
(2)	Cover > 3*Db	1.20
(3)	Clear spacing <= 6*Db	1.50
(4)	Clear spacing > 6*Db	1.20
(5)	Lateral c-c spacing >= 6" & cover along spacing >= 3"	0.80
(6)	Enclosed by spiral with diam > 0.25" & pitch <= 4"	0.75
	Top Bars	1.40

EPOXY BARS - TOP

	3	4	5	6	7	8	9	10	11	14	18
3*Db	1.125	1.500	1.875	2.250	2.625	3.000	3.384	3.810	4.230	5.079	6.771
6*Db	2.250	3.000	3.750	4.500	5.250	6.000	6.768	7.620	8.460	10.158	13.542
	Development Length - inches										
(1)	16	21	26	31	39	51	64	81	100	138	179
(2)	16	21	26	31	38	50	63	81	99	137	177
(3)	16	21	26	31	39	51	64	81	100	138	179
(4)	16	21	26	31	38	50	63	81	99	137	177
(1) & (5)	16	21	26	31	38	50	63	81	99	137	177
(2) & (5)	13	17	21	25	31	40	51	65	79	109	142
(1) & (6)	15	19	24	29	36	47	60	76	93	128	166
(2) & (6)	12	16	19	23	29	38	48	61	74	103	133
(3) & (6)	15	19	24	29	36	47	60	76	93	128	166
(4) & (6)	12	16	19	23	29	38	48	61	74	103	133
(1) & (5) & (6)	12	16	19	23	29	38	48	61	74	103	133
(2) & (5) & (6)	12	13	16	19	23	30	38	49	59	82	106

COMPRESSION DEVELOPMENT LENGTH
Grade 60 reinforcement; 4000 psi concrete

MODIFICATION FACTORS

(1)	Basic development length	1.00
(2)	Enclosed by spiral with diam > 0.25" & pitch <= 4"	0.75

	3	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**14**	**18**
	Development Length - inches										
(1)	8	10	12	15	17	19	22	25	27	32	43
(2)	8	8	9	11	13	15	16	19	20	24	32

HOOKED TENSION DEVELOPMENT LENGTH
Grade 60 reinforcement; 4000 psi concrete

MODIFICATION FACTORS FOR PLAIN BARS

(1)	Basic hooked development length	1.00
(2)	Side cover for <#14 bar normal to plane of hook >=2.5"	0.70
(3)	90° hook: Side cover for <#14 bar normal to plane of hook >=2.5" & cover on extension beyond hook >=2"	0.70
(4)	Hooks for <#14 enclosed in stirrups/ties along full development length at a spacing <=3*Db	0.80

PLAIN BARS

	3	4	5	6	7	8	9	10	11	14	18
3*Db	1.125	1.5	1.875	2.25	2.625	3	3.384	3.81	4.23	5.079	6.771
	Development Length - inches										
(1)	8	10	12	15	17	19	22	25	27	33	43
(2)	6	7	9	10	12	14	16	17	19		
(3)	6	7	9	10	12	14	16	17	19		
(4)	6	8	10	12	14	16	18	20	22		
(2) & (4)	6	6	7	8	10	11	13	14	16		
(3) & (4)	6	6	7	8	10	11	13	14	16		

MODIFICATION FACTORS FOR EPOXY BARS

(1)	Basic hooked development length	1.00
(2)	Side cover for <#14 bar normal to plane of hook >=2.5"	0.70
(3)	90° hook: Side cover for <#14 bar normal to plane of hook >=2.5" & cover on extension beyond hook >=2"	0.70
(4)	Hooks for <#14 enclosed in stirrups/ties along full development length at a spacing <=3*Db	0.80
	Epoxy Bar	1.20

EPOXY BARS

	3	4	5	6	7	8	9	10	11	14	18
3*Db	1.125	1.500	1.875	2.250	2.625	3.000	3.384	3.810	4.230	5.079	6.771
	Development Length - inches										
(1)	9	12	15	18	20	23	26	29	33	39	52
(2)	6	8	10	12	14	16	19	21	23		
(3)	6	8	10	12	14	16	19	21	23		
(4)	7	10	12	14	16	19	21	24	26		
(2) & (4)	6	7	8	10	12	13	15	17	19		
(3) & (4)	6	7	8	10	12	13	15	17	19		

5.11.5.3.1 LAP SPLICES IN TENSION

Grade 60 reinforcement; 4000 psi concrete

(1)	Lateral c-c spacing >= 6" & cover along spacing >= 3"
(2)	Enclosed by spiral with diam > 0.25" & pitch <= 4"

PLAIN BARS - OTHER

	3			4			5			6			7			8			9			10			11			14			18		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
	Lap Splice Length - inches																																
Basic Ldb	12	12	16	12	16	20	15	20	26	18	23	31	23	30	39	30	39	51	38	49	65	48	62	82	59	77	100	81	105	138	105	137	179
(1)	12	12	13	12	16	20	12	16	20	15	20	26	18	23	31	24	31	41	30	39	51	39	51	66	47	61	80	65	85	111	84	109	143
(2)	12	12	12	12	16	20	12	16	20	14	18	24	17	22	29	23	30	39	29	38	49	36	47	61	44	57	75	61	79	104	79	103	134
(1) & (2)	12	12	12	12	16	20	12	16	20	12	16	20	14	18	24	18	23	31	23	30	39	29	38	49	36	47	61	49	64	83	63	82	107

PLAIN BARS - TOP

	3			4			5			6			7			8			9			10			11			14			18		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
	Lap Splice Length - inches																																
Basic Ldb	13	17	22	17	22	29	21	27	36	26	34	44	32	42	54	42	55	71	53	69	90	67	87	114	82	107	139	114	148	194	147	191	250
(1)	12	14	18	14	18	24	17	22	29	21	27	36	26	34	44	34	44	58	42	55	71	54	70	92	66	86	112	91	118	155	118	153	201
(2)	12	12	12	13	17	22	16	21	27	19	25	32	24	31	41	32	42	54	40	52	68	51	66	87	62	81	105	86	112	146	111	144	189
(1) & (2)	12	12	12	12	16	20	13	17	22	16	21	27	19	25	32	25	33	43	32	42	54	41	53	70	50	65	85	69	90	117	89	116	151

5.11.5.3.1 LAP SPLICES IN TENSION
Grade 60 reinforcement; 4000 psi concrete

(1)	Cover <= 3*Db
(2)	Cover > 3*Db
(3)	Clear spacing <= 6*Db
(4)	Clear spacing > 6*Db
(5)	Lateral c-c spacing >= 6" & cover along spacing >= 3"
(6)	Enclosed by spiral with diam > 0.25" & pitch <= 4"

EPOXY BARS - OTHER

	3			**4**			**5**			**6**			**7**			**8**			**9**			**10**			**11**			**14**			**18**		
3*Db	1.125			1.500			1.875			2.250			2.625			3.000			3.384			3.810			4.230			5.079			6.771		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
	Lap Splice Length - inches																																
(1)	14	18	23	18	23	31	23	30	39	27	35	46	34	44	58	45	59	77	57	74	97	72	94	122	88	114	150	122	159	207	158	205	269
(2)	12	15	19	15	20	26	18	23	31	22	29	37	27	35	46	36	47	61	45	59	77	58	75	99	71	92	121	98	127	167	126	164	214
(3)	14	18	23	18	23	31	23	30	39	27	35	46	34	44	58	45	59	77	57	74	97	72	94	122	88	114	150	122	159	207	158	205	269
(4)	12	15	19	15	20	26	18	23	31	22	29	37	27	35	46	36	47	61	45	59	77	58	75	99	71	92	121	98	127	167	126	164	214
(1) & (5)	12	15	19	15	20	26	18	23	31	22	29	37	27	35	46	36	47	61	45	59	77	58	75	99	71	92	121	98	127	167	126	164	214
(2) & (5)	12	12	15	12	16	20	15	20	26	18	23	31	22	29	37	29	38	49	36	47	61	46	60	78	57	74	97	78	101	133	101	131	172
(1) & (6)	12	14	18	14	18	24	17	22	29	21	27	36	26	34	44	34	44	58	43	56	73	54	70	92	66	86	112	92	120	156	119	155	202
(2) & (6)	12	12	14	12	16	20	14	18	24	17	22	29	21	27	36	27	35	46	34	44	58	43	56	73	53	69	90	73	95	124	95	124	162
(3) & (6)	12	14	18	14	18	24	17	22	29	21	27	36	26	34	44	34	44	58	43	56	73	54	70	92	66	86	112	92	120	156	119	155	202
(4) & (6)	12	12	14	12	16	20	14	18	24	17	22	29	21	27	36	27	35	46	34	44	58	43	56	73	53	69	90	73	95	124	95	124	162
(1) & (5) & (6)	12	12	14	12	16	20	14	18	24	17	22	29	21	27	36	27	35	46	34	44	58	43	56	73	53	69	90	73	95	124	95	124	162
(2) & (5) & (6)	12	12	12	12	16	20	12	16	20	13	17	22	17	22	29	22	29	37	27	35	46	35	46	60	43	56	73	59	77	100	76	99	129

EPOXY BARS - TOP

	3			**4**			**5**			**6**			**7**			**8**			**9**			**10**			**11**			**14**			**18**		
3*Db	1.125			1.500			1.875			2.250			2.625			3.000			3.384			3.810			4.230			5.079			6.771		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
	Lap Splice Length - inches																																
(1)	16	20	27	21	27	36	26	34	44	31	40	53	39	51	66	51	66	87	64	83	109	81	105	138	100	130	170	138	179	235	179	233	304
(2)	16	20	26	21	27	36	26	34	44	31	40	53	38	49	65	50	65	85	63	82	107	81	105	138	99	129	168	137	178	233	177	230	301
(3)	16	20	27	21	27	36	26	34	44	31	40	53	39	51	66	51	66	87	64	83	109	81	105	138	100	130	170	138	179	235	179	233	304
(4)	16	20	26	21	27	36	26	34	44	31	40	53	38	49	65	50	65	85	63	82	107	81	105	138	99	129	168	137	178	233	177	230	301
(1) & (5)	16	20	26	21	27	36	26	34	44	31	40	53	38	49	65	50	65	85	63	82	107	81	105	138	99	129	168	137	178	233	177	230	301
(2) & (5)	13	16	21	17	22	29	21	27	36	25	33	43	31	40	53	40	52	68	51	66	87	65	85	111	79	103	134	109	142	185	142	185	241
(1) & (6)	15	19	25	19	25	32	24	31	41	29	38	49	36	47	61	47	61	80	60	78	102	76	99	129	93	121	158	128	166	218	166	216	282
(2) & (6)	12	15	20	16	21	27	19	25	32	23	30	39	29	38	49	38	49	65	48	62	82	61	79	104	74	96	126	103	134	175	133	173	226
(3) & (6)	15	19	25	19	25	32	24	31	41	29	38	49	36	47	61	47	61	80	60	78	102	76	99	129	93	121	158	128	166	218	166	216	282
(4) & (6)	12	15	20	16	21	27	19	25	32	23	30	39	29	38	49	38	49	65	48	62	82	61	79	104	74	96	126	103	134	175	133	173	226
(1) & (5) & (6)	12	15	20	16	21	27	19	25	32	23	30	39	29	38	49	38	49	65	48	62	82	61	79	104	74	96	126	103	134	175	133	173	226

COMPRESSION DEVELOPMENT LENGTH
Grade 60 reinforcement; 4000 psi concrete

MODIFICATION FACTORS

(1)	Basic development length	1.00
(2)	Ties along splice with effective area >= 15% of (compression component thickness)*(tie spacing)	0.83
(3)	With spirals	0.75

	3	4	5	6	7	8	9	10	11	14	18
	Lap Splice Length - inches										
(1)	12	15	19	23	27	30	34	39	43	51	68
(2)	12	13	16	19	22	25	29	32	36	43	57
(3)	12	12	15	17	20	23	26	29	32	39	51

PRESTRESSED GIRDER PROPERTIES

GIRDER W/CAST-IN-PLACE DECK

Section Properties and maximum span length curves for the AASHTO, Bulb Tee, and Wide Flange girders are included in this Article.
The maximum span length curves should only be used as an aid in preliminary design. The curves on based on the following design parameters:

- AASHTO LRFD Design Specifications
- Simple Span lengths are centerline-centerline bearing
- 42'-0" out-out bridge width
- Girder spacing is for 4,5,6,& 7 girders (6'-0"; 7'-3"; 9'-3"; 12'-0")
- Concrete parapet
- Slab f'c = 4.0 ksi
- Future wearing surface = 28 psf
- HL93 live load
- Harp points at 0.4 & 0.6 points
- Deck thickness determined by (S+10)/30 where S is computed in accordance with Article 9.7.2.3
- Minimum 8" nominal deck slab thickness. Structural deck thickness is 0.5" less than the nominal thickness
- Maximum number of 0.6" dia. 270k strand = 62
- The maximum number of straight strand for each girder is:

AASHTO Type 2	16 straight
AASHTO Type 3	34 straight
AASHTO Type 4	48 straight
Bulb Tee Girder	24 straight
WF Girders	46 straight

DECK BULB TEE GIRDER W/ASPHALT OVERLAY

Section Properties and maximum span length curves for the Bulb Tee girders with an 8" thick top flange are included in this Article.
The maximum span length curves should only be used as an aid in preliminary design. The curves on based on the following design parameters:

- AASHTO LRFD Design Specifications
- Simple Span lengths are centerline-centerline bearing
- 42'-0" out-out bridge width
- Girder spacing is for 6, 7, & 8 girders (84", 72" & 63" top flange width)
- Concrete parapet
- Slab f'c = 4.0 ksi
- Future wearing surface = 28 psf
- HL93 live load
- Harp points at 0.4 & 0.6 points

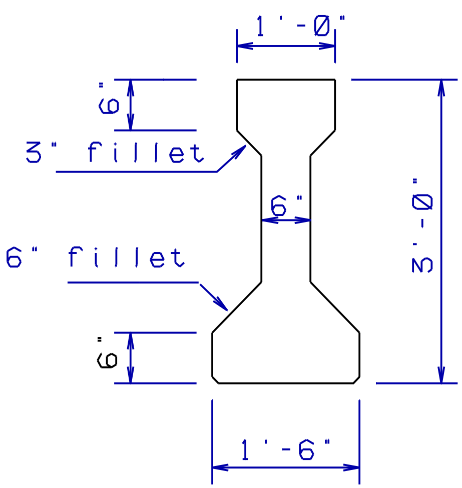
Revisions:

July 2009	Added design parameters on page 1 Added WSDOT WF girder data Added Deck Bulb Tee Girder maximum span curves

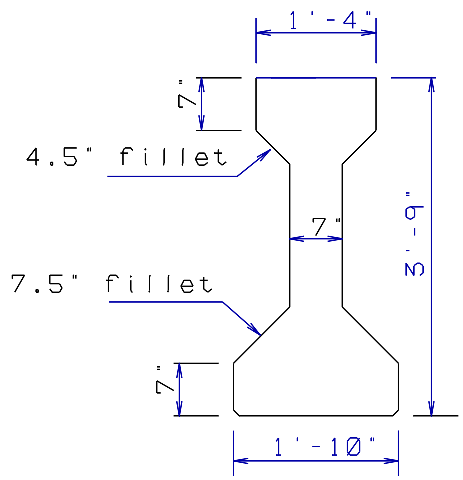
AASHTO GIRDER SECTION PROPERTIES

		CENTER OF GRAVITY			SECTION MODULUS		WEIGHT
DEPTH	AREA	TOP	BOTTOM	MOMENT OF INERTIA	TOP	BOTTOM	LB/FT
36"	368.44	20.147	15.853	50,842	2524	3207	384
45"	558.94	24.706	20.294	125,165	5066	6168	582
54"	788.44	29.249	24.751	260,403	8903	10,521	821

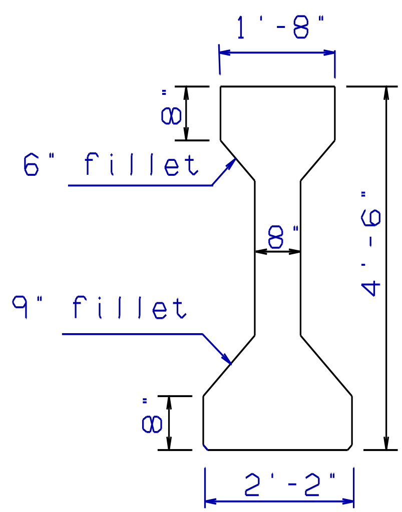
ALL UNITS IN INCHES EXCEPT WEIGHT



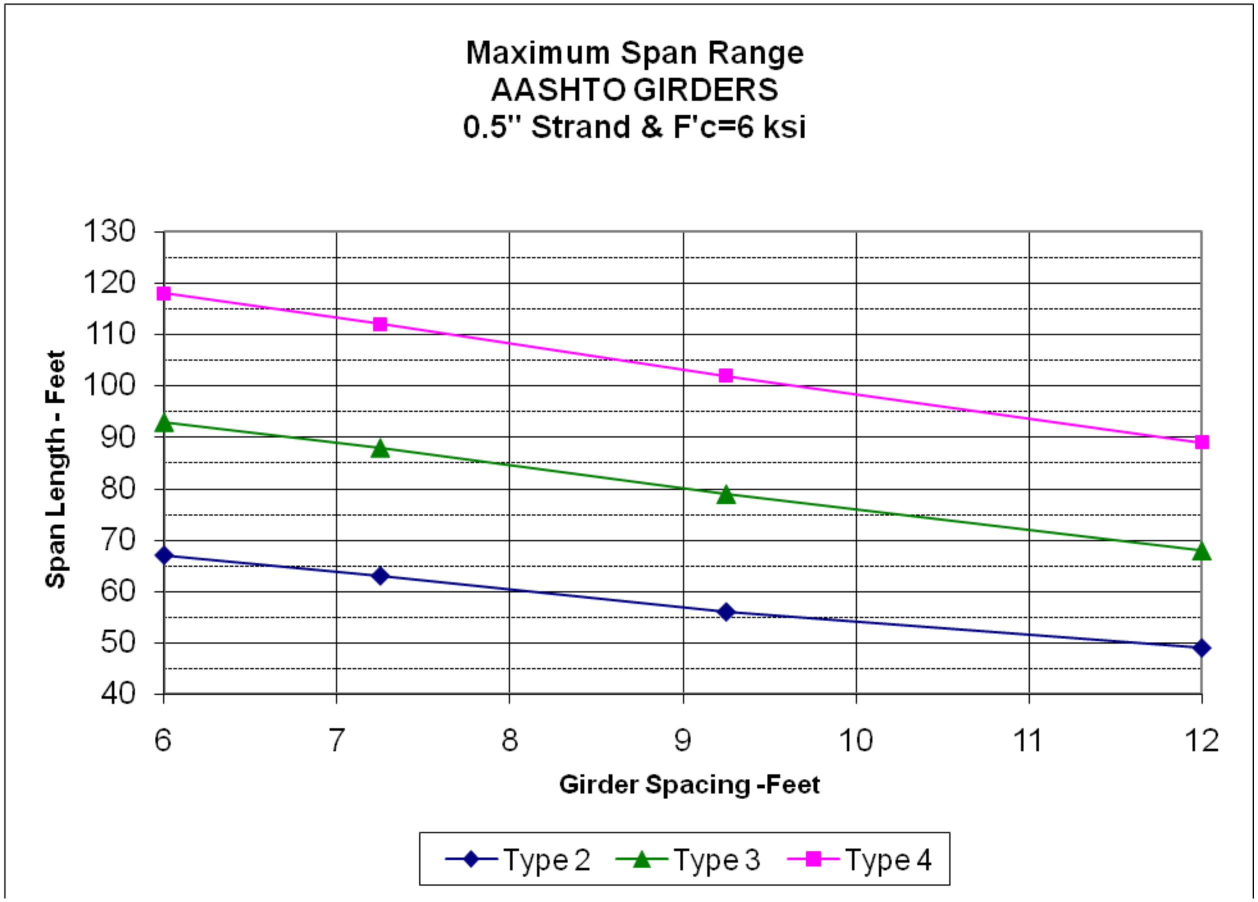
TYPE 2 GIRDER

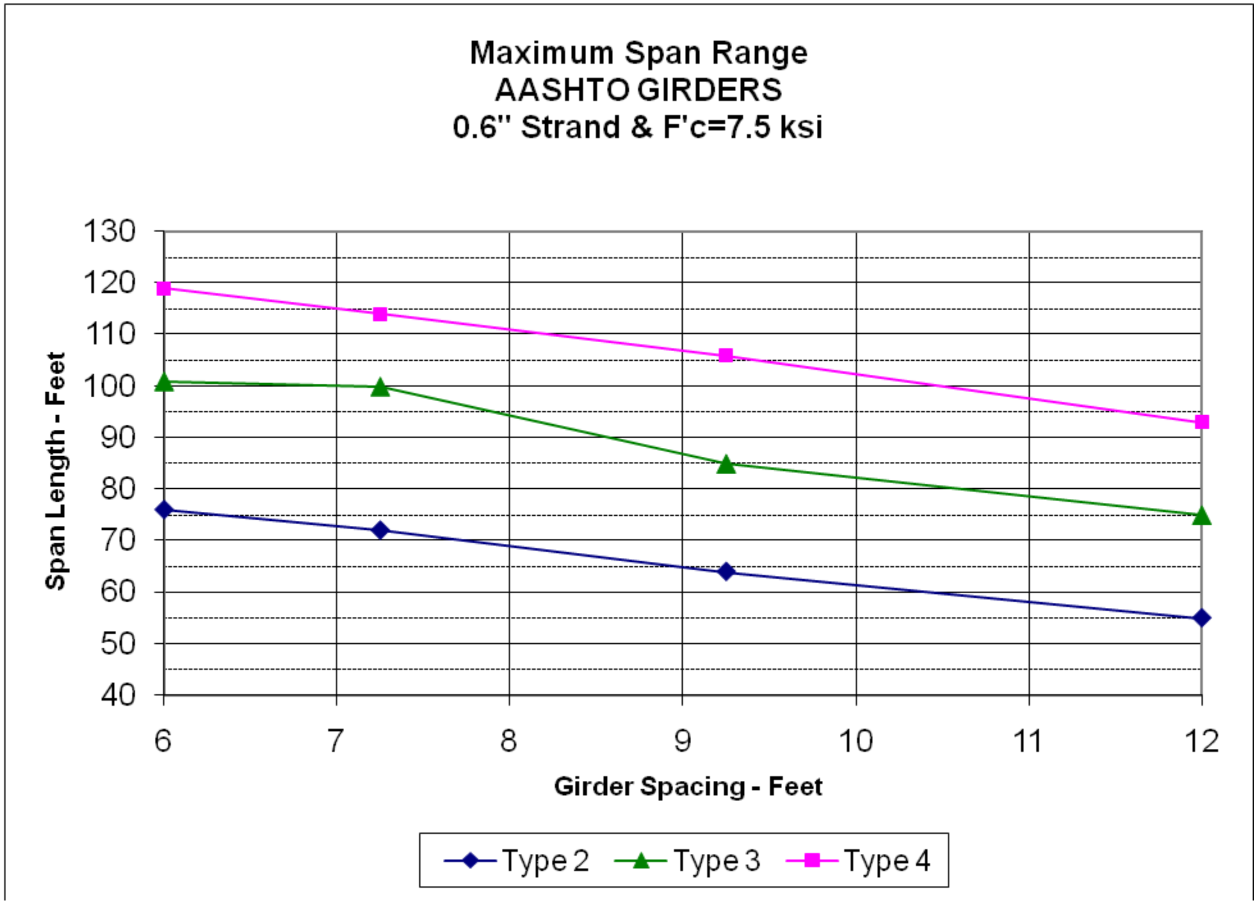


TYPE 3 GIRDER



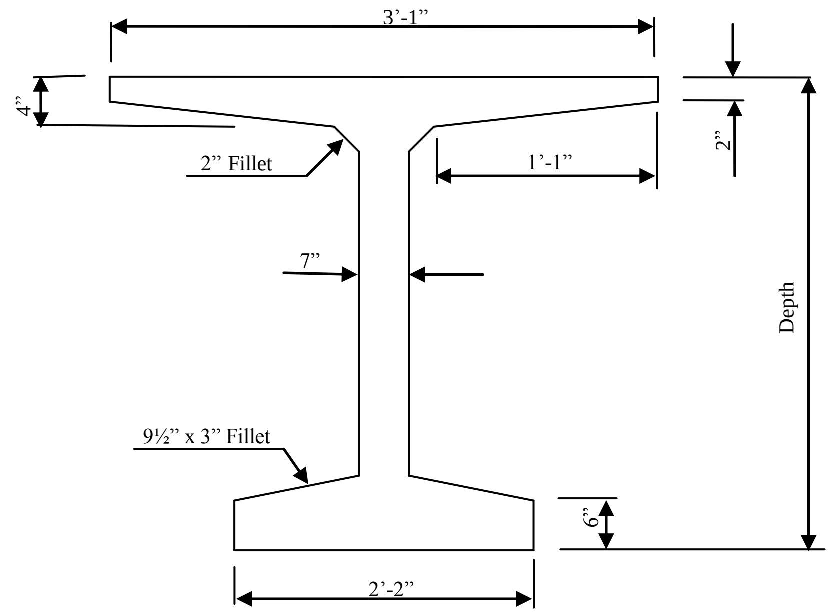
TYPE 4 GIRDER

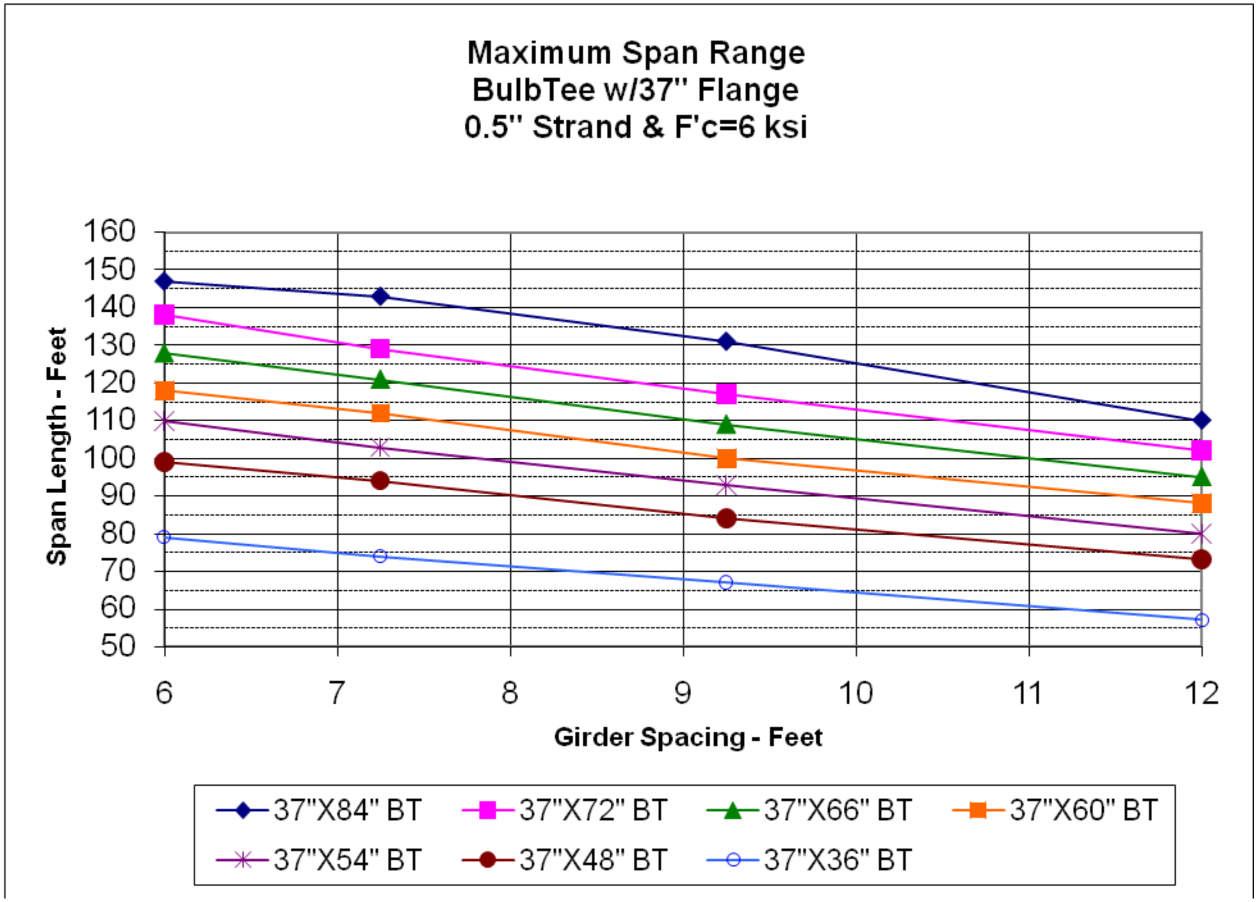

Maximum Span Range
AASHTO GIRDERS
0.5'' Strand & F'c=6 ksi
Span Length - Feet
130
120
110
100
90
80
70
60
50
40
6
7
8
9
10
11
12
Girder Spacing -Feet
Type 2
Type 3
Type 4

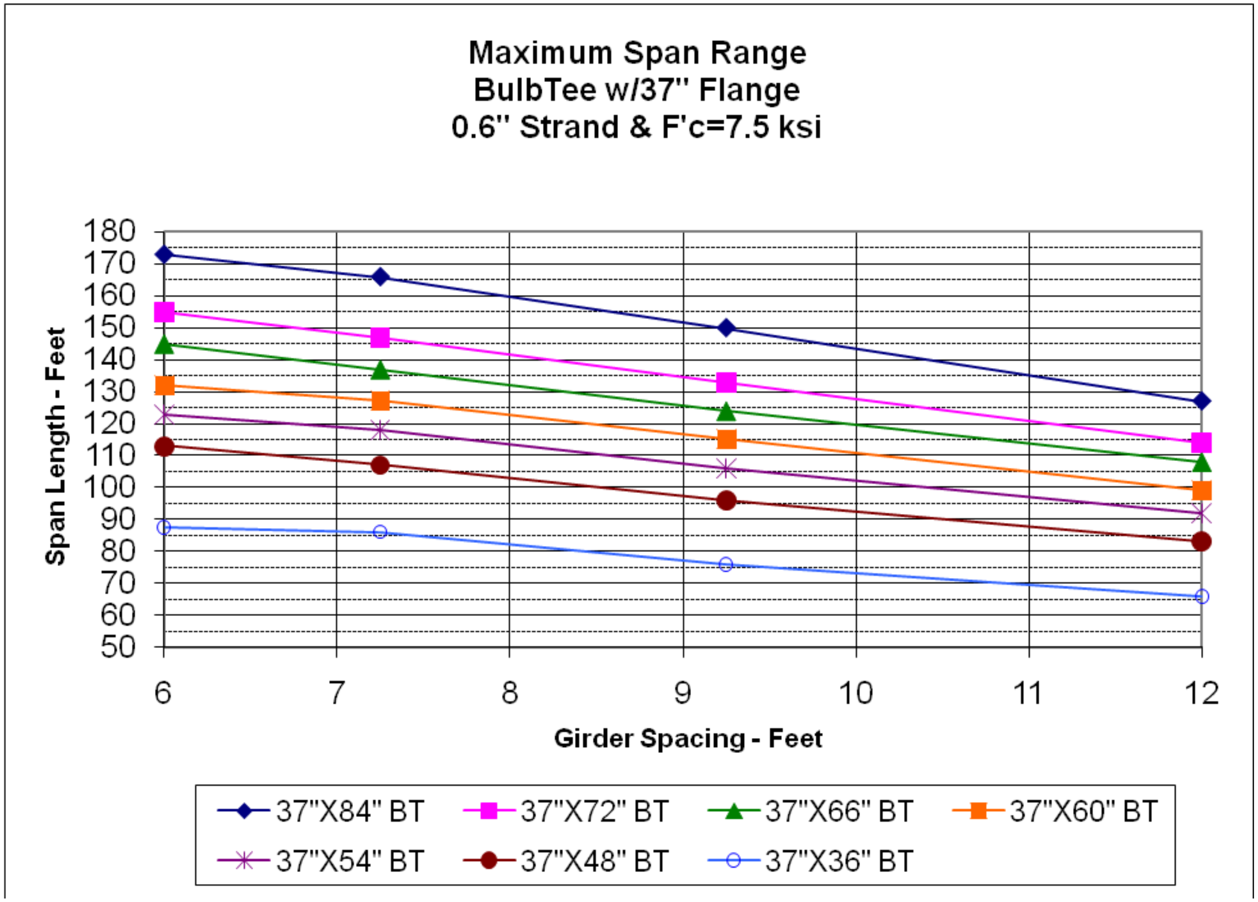

Maximum Span Range
AASHTO GIRDERS
0.6" Strand & F'c=7.5 ksi
Span Length - Feet
130
120
110
100
90
80
70
60
50
40
6
7
8
9
10
11
12
Girder Spacing - Feet
Type 2
Type 3
Type 4

37” TOP FLANGE BULB TEE GIRDER SECTION PROPERTIES

		CENTER OF GRAVITY			SECTION MODULUS		WEIGHT LB/FT
DEPTH	AREA	TOP	BOTTOM	MOMENT OF INERTIA	TOP	BOTTOM	
30"	449.9375	15.644	14.356	51,361	3,283	3,578	469
36"	491.9375	18.857	17.143	82,126	4,355	4,791	512
42"	533.9375	22.036	19.964	121,708	5,523	6,096	556
48"	575.9375	25.190	22.810	170,872	6,783	7,491	600
54"	617.9375	28.322	25.678	230,379	8,134	8,972	644
60"	659.9375	31.438	28.562	300,990	9,574	10,538	687
66"	701.9375	34.539	31.461	383,465	11,102	12,189	731
72"	743.9375	37.630	34.370	478,562	12,718	13,924	775
84"	827.9375	43.783	40.217	709,652	16,208	17,645	862

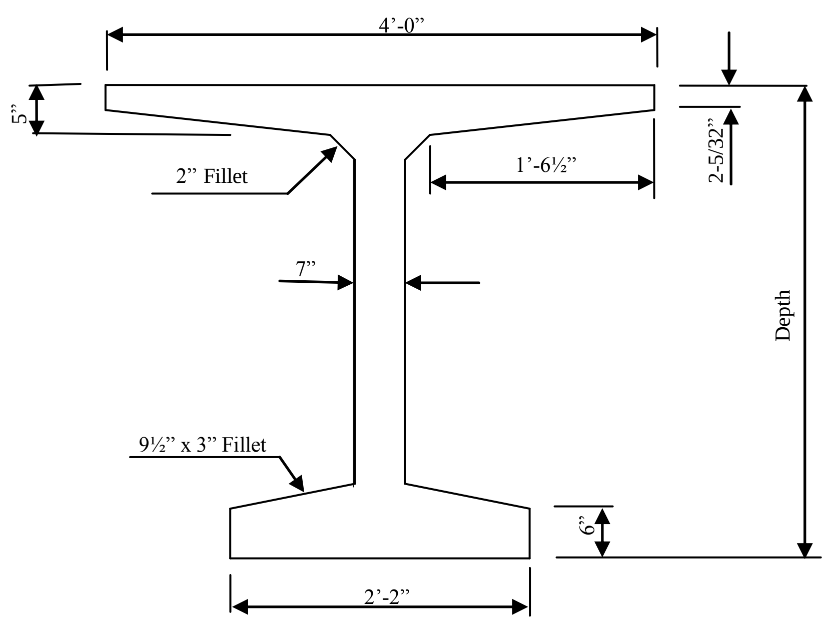


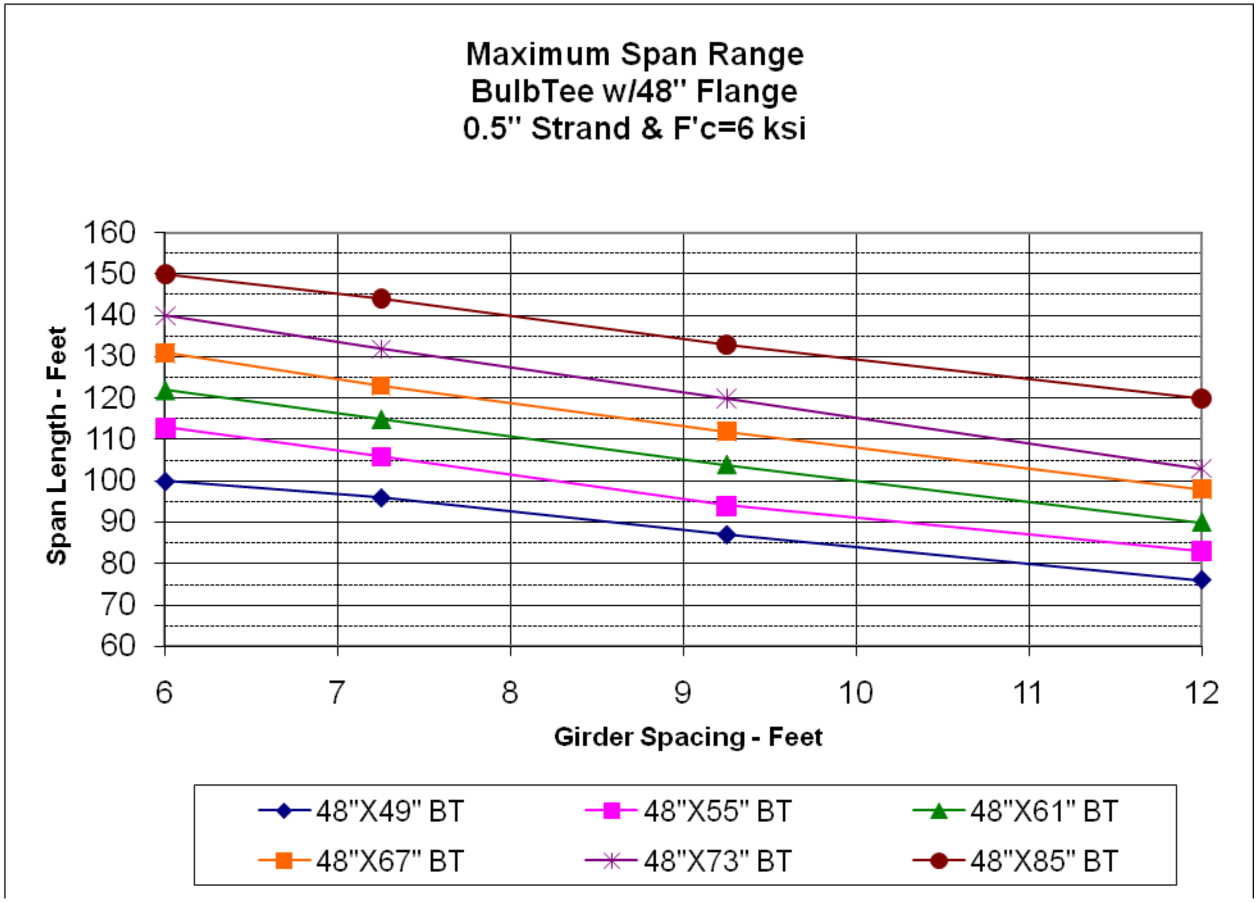

Maximum Span Range
BulbTee w/37" Flange
0.5" Strand & F'c=6 ksi
Span Length - Feet
160
150
140
130
120
110
100
90
80
70
60
50
6
7
8
9
10
11
12
Girder Spacing - Feet
37"X84" BT
37"X72" BT
37"X66" BT
37"X60" BT
37"X54" BT
37"X48" BT
37"X36" BT

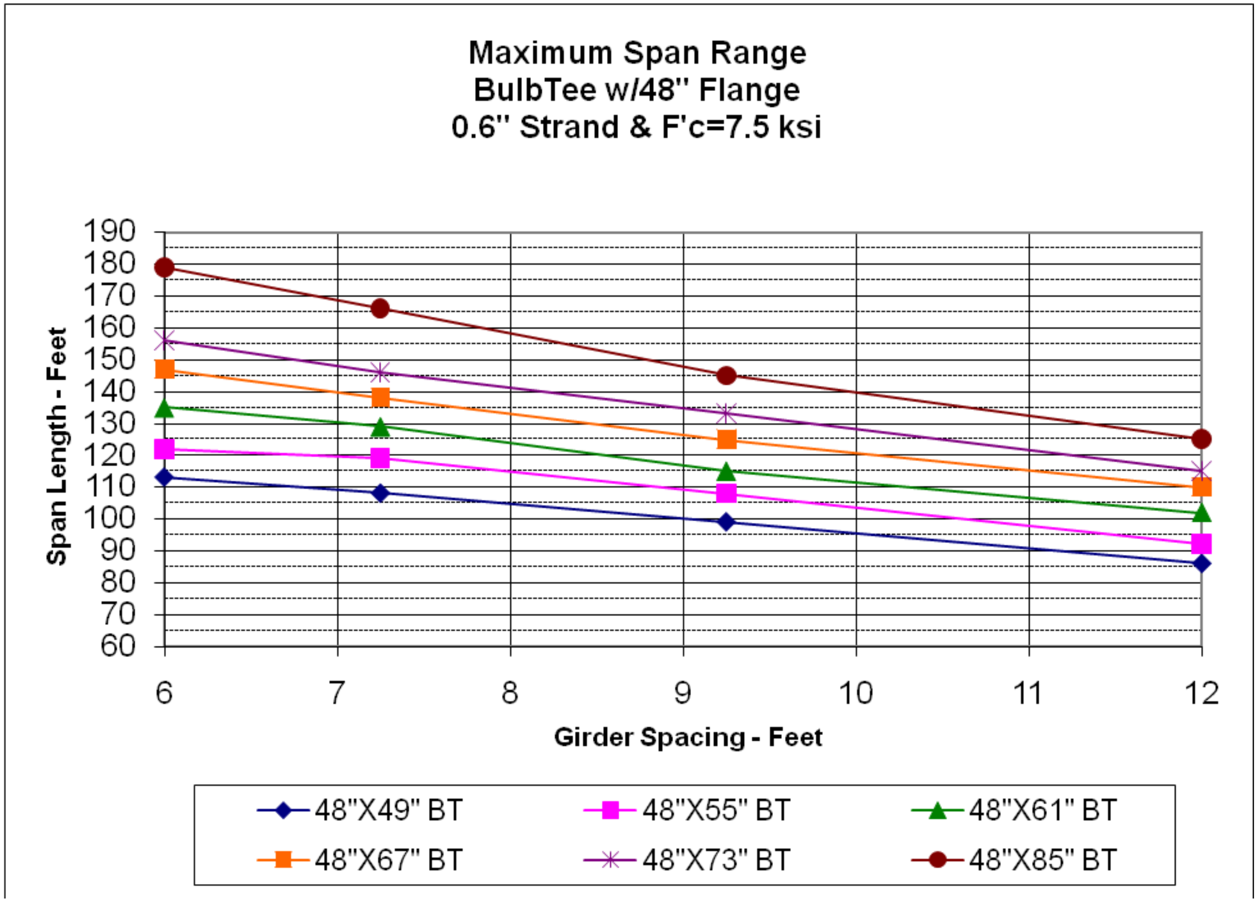

Maximum Span Range
BulbTee w/37" Flange
0.6" Strand & F'c=7.5 ksi
Span Length - Feet
180
170
160
150
140
130
120
110
100
90
80
70
60
50
6
7
8
9
10
11
12
Girder Spacing - Feet
37"X84" BT
37"X72" BT
37"X66" BT
37"X60" BT
37"X54" BT
37"X48" BT
37"X36" BT

48” TOP FLANGE BULB TEE GIRDER SECTION PROPERTIES

		CENTER OF GRAVITY			SECTION MODULUS		WEIGHT LB/FT
DEPTH	AREA	TOP	BOTTOM	MOMENT OF INERTIA	TOP	BOTTOM	
49"	641.284	23.607	25.393	208,593	8,836	8,215	668
55"	683.284	26.600	28.400	278,326	10,464	9,800	712
61"	725.284	29.591	31.409	360,358	12,178	11,473	756
67"	767.284	32.585	34.415	455,445	13,977	13,234	799
73"	809.284	35.579	37.421	564,342	15,862	15,081	843
85"	893.284	41.569	43.431	826,595	19,885	19,032	931

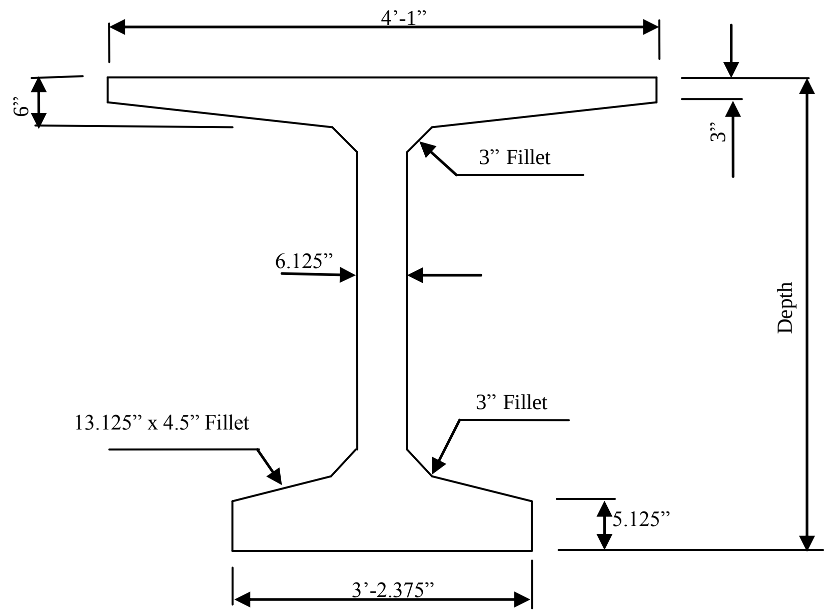


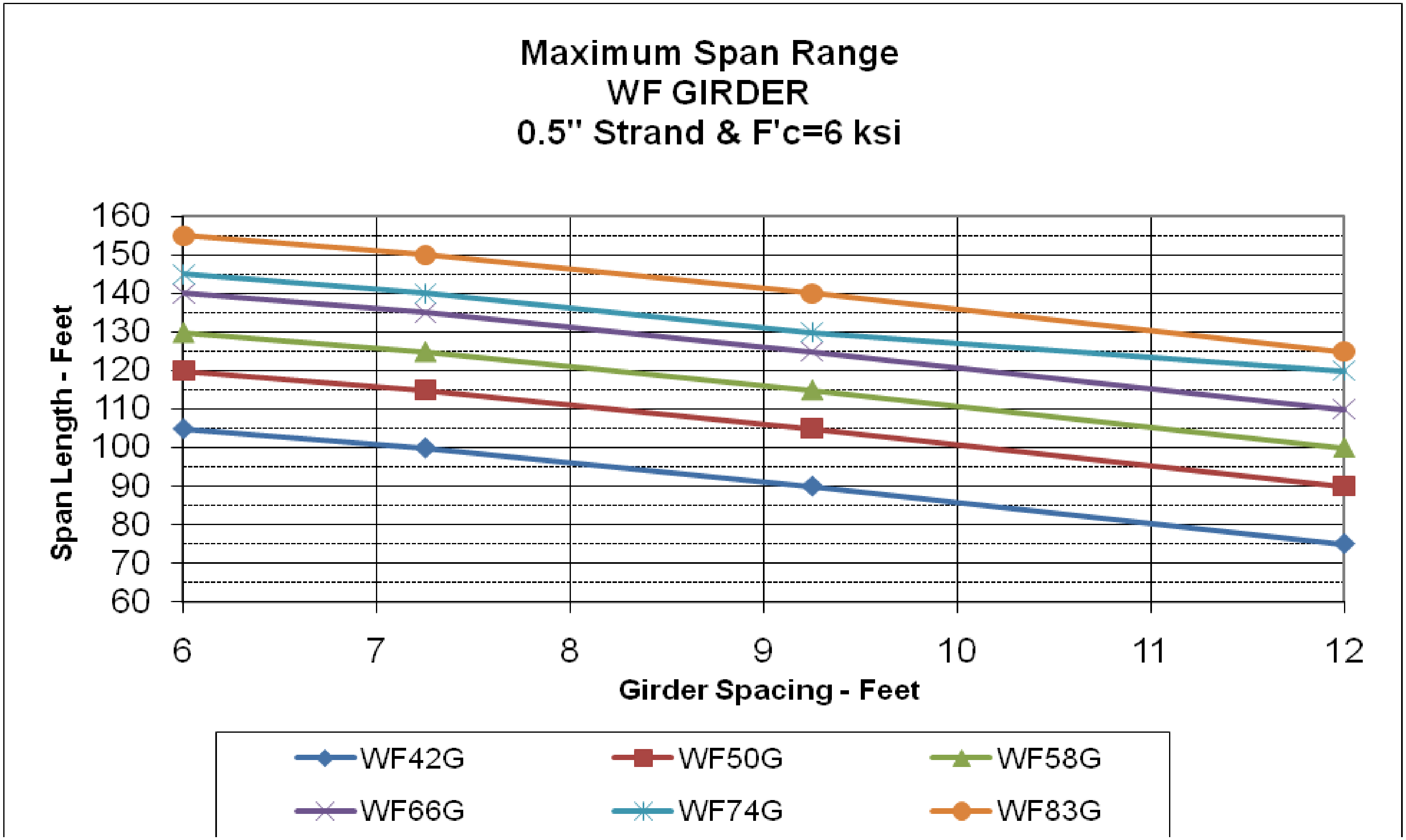

Maximum Span Range
BulbTee w/48" Flange
0.5" Strand & F'c=6 ksi
Span Length - Feet
160
150
140
130
120
110
100
90
80
70
60
6
7
8
9
10
11
12
Girder Spacing - Feet
48"X49" BT
48"X55" BT
48"X61" BT
48"X67" BT
48"X73" BT
48"X85" BT

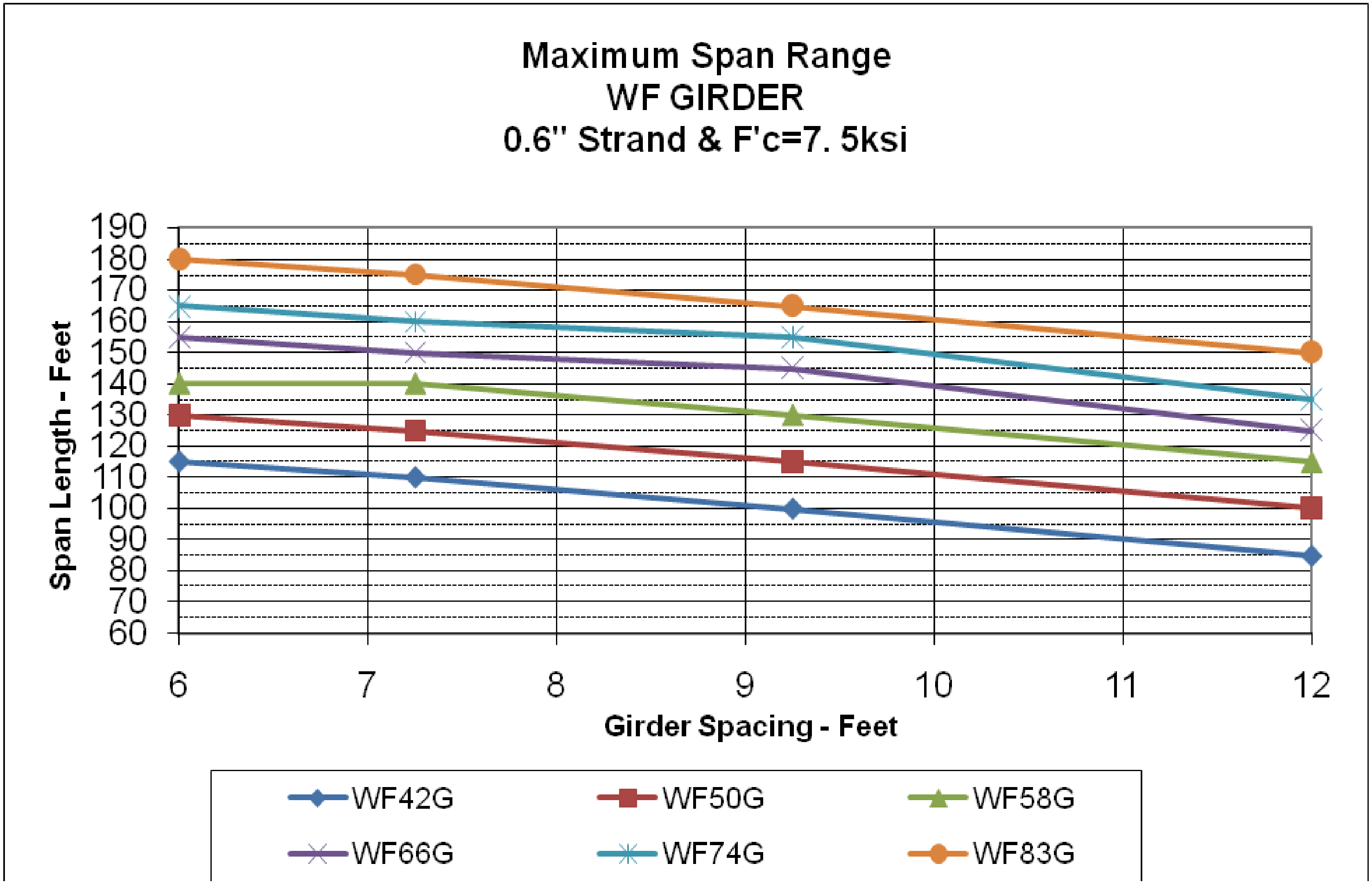

Maximum Span Range
BulbTee w/48" Flange
0.6" Strand & F'c=7.5 ksi
Span Length - Feet
190
180
170
160
150
140
130
120
110
100
90
80
70
60
6
7
8
9
10
11
12
Girder Spacing - Feet
48"X49" BT
48"X55" BT
48"X61" BT
48"X67" BT
48"X73" BT
48"X85" BT

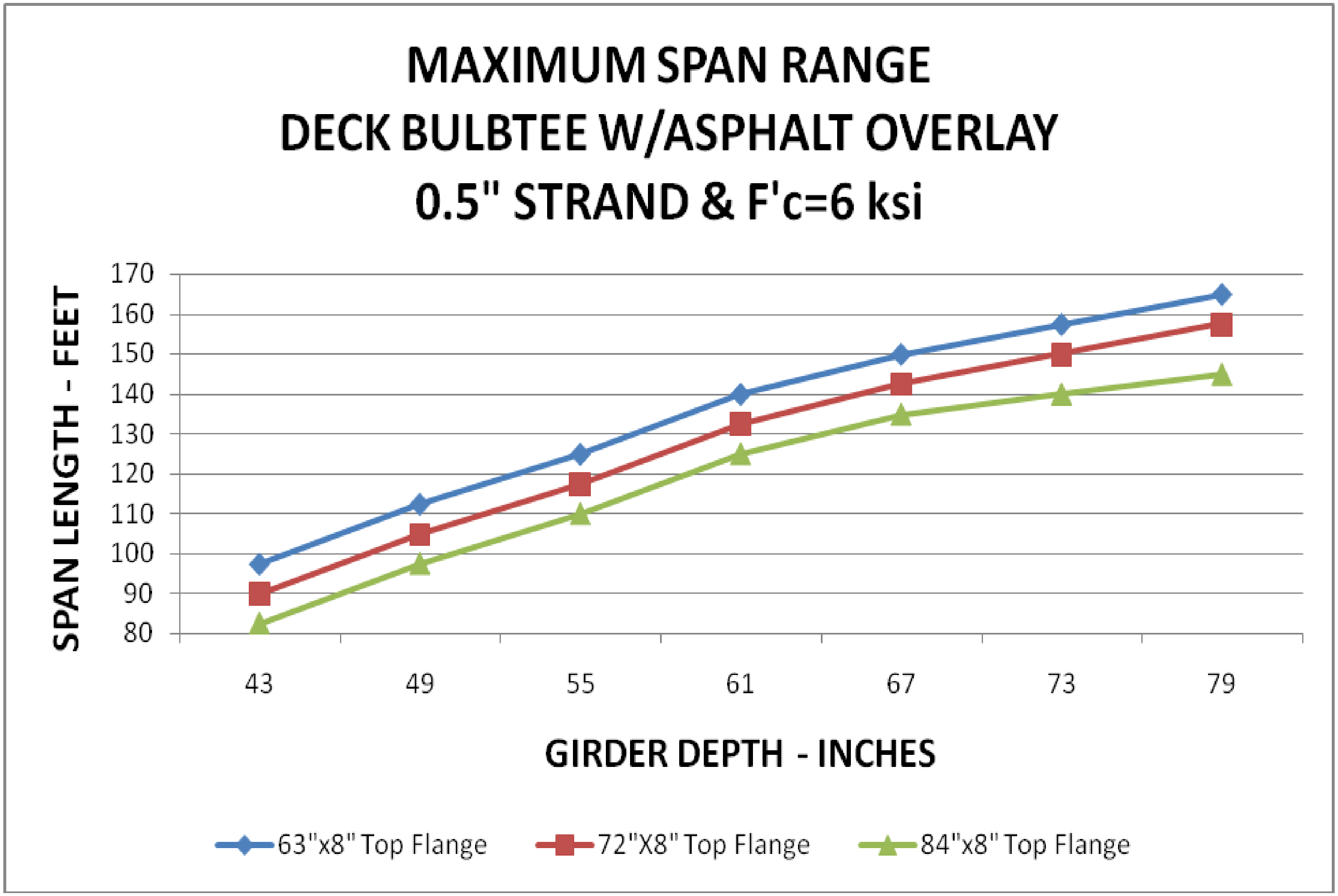
WF GIRDER SECTION PROPERTIES

DEPTH	AREA	CENTER OF GRAVITY TOP	CENTER OF GRAVITY BOTTOM	MOMENT OF INERTIA	SECTION MODULUS TOP	SECTION MODULUS BOTTOM	WEIGHT LB/FT
42"	727.531	21.640	20.360	183,642	8,486	9,020	758
50"	776.531	25.849	21.151	282,559	10,931	11,700	809
58"	825.531	30.033	27.967	406,266	13,527	14,527	860
66"	874.531	34.196	31.804	556,339	16,269	17,493	911
74"	923.531	38.343	35.657	734,356	19,152	20,595	962
82.625"	976.359	42.796	39.829	959,393	22,418	24,088	1017




Maximum Span Range
WF GIRDER
0.5" Strand & F'c=6 ksi
Span Length - Feet
160
150
140
130
120
110
100
90
80
70
60
6
7
8
9
10
11
12
Girder Spacing - Feet
WF42G
WF50G
WF58G
WF66G
WF74G
WF83G

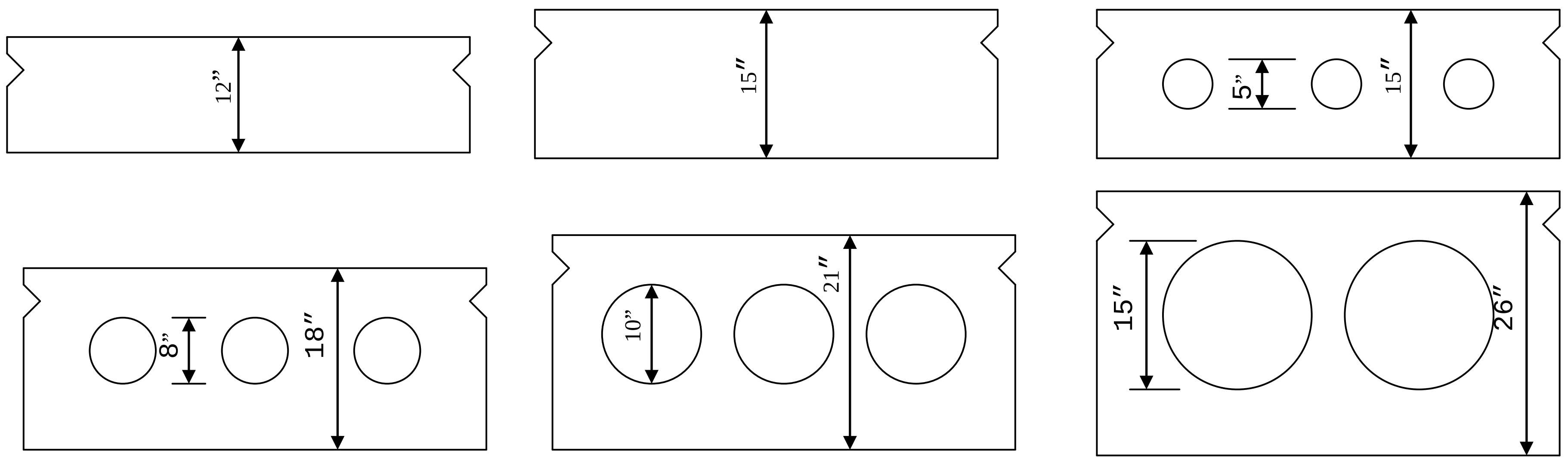

Maximum Span Range
WF GIRDER
0.6" Strand & F'c=7. 5ksi
Span Length - Feet
190
180
170
160
150
140
130
120
110
100
90
80
70
60
6
7
8
9
10
11
12
Girder Spacing - Feet
WF42G
WF50G
WF58G
WF66G
WF74G
WF83G


MAXIMUM SPAN RANGE
DECK BULBTEE W/ASPHALT OVERLAY
0.5" STRAND & F'c=6 ksi
SPAN LENGTH - FEET
170
160
150
140
130
120
110
100
90
80
43
49
55
61
67
73
79
GIRDER DEPTH - INCHES
63"x8" Top Flange
72"X8" Top Flange
84"x8" Top Flange

PRECAST PRESTRESSED SLAB SECTION PROPERTIES

		CENTER OF GRAVITY			SECTION MODULUS		WEIGHT
DEPTH	AREA	TOP	BOTTOM	MOMENT OF INERTIA	TOP	BOTTOM	LB/FT
12" SOLID	561.188	6.055	5.945	6782	1120	1141	585
15" SOLID	705.188	7.573	7.427	13233	1746	1780	735
15" VOID	646.283	7.534	7.466	13121	1742	1757	673
18" VOID	698.391	8.995	9.005	22208	2469	2466	728
21" VOID	757.568	10.622	10.378	34798	3276	3353	789
26" VOID	879.758	13.144	12.856	63943	4865	4974	917

ALL UNITS IN INCHES EXCEPT WEIGHT



All slabs are 48" wide

MAXIMUM SPAN LENGTH
PRECAST PRESTRESSED SLABS

	Span	**# of strand**	**Center of gravity**
12” Solid Slab	20’	16	3.75”
15” Solid Slab	30’	22	4.18”
15” Voided Slab	30’	22	4.18”
18” Voided Slab	40’	26	4.31”
21” Voided Slab	50’	30	4.80”
26” Voided Slab	55’	32	6.38”

DESIGN CRITERIA
AASHTO LRFD
Beams are parallel
Minimum 4 girders
28 psf asphalt pavement
44'-0" out - out bridge width with a concrete parapet
HL93 Live Load

½" ϕ 270^k low relaxation strand
Girder f’c = 6000 psi maximum

NOTES TO DESIGNER FOR PRECAST PRESTRESSED SLAB

The Standard Drawings for precast prestressed slabs are shown in Appendix B for Section 5.

The following criteria shall be used in developing details for projects utilizing precast prestressed slabs:

PRESTRESSING STEEL
Prestressing steel shall be designed as straight strand.

TIE RODS
Tie rods spacing shall be as follows:

SPAN ≤ 40'	at centerline span
SPAN > 40'	at 1/3 points

Tie rods shall be oriented as follows:

skew angle < 20°	parallel to centerline bearing
skew angle > 20°	perpendicular to slab centerline

When tie rod lengths greater than 20' are required, specify heavy-duty sleeve nuts to obtain the required length.

BEARING PADS
Bearing pads should be designed in accordance with Article 14.7.5 of the Bridge Design Manual.

The beam seat shall be parallel to the bottom of the beams.

MEMBRANE SEAL
A waterproof membrane seal shall be applied to the top surface of the slabs when an asphalt wearing surface is used. The membrane seal shall conform to Section 511, Concrete Waterproofing System, Type A or Type D.

Revisions:
July 2009 Corrected references on page 3 to Section 511 and System Type D.

DEFLECTION

Beam self weight

$$\Delta_g = \frac{5 * w_{beam} * L^4 * 1728}{384 * E_{ci} * I_g}$$

$$E_{ci} = 57.619\sqrt{f'_{ci}} \quad \text{(ksi)}$$

f'_{ci} = girder concrete strength at release (psi)
I_g = Moment of Inertia of girder (in^4)
L = span length (ft)
W_{beam} = beam dead load (kips/ft)

Non-composite Dead Loads

$$\Delta_{SDL} = \frac{5 * w_{SDL} * L^4 * 1728}{384 * E_c * I_g} \qquad E_c = \frac{E_{ci}}{0.85}$$

Composite Dead Loads

$$\Delta_{CDL} = \frac{5 * w_{CDL} * L^4 * 1728}{384 * E_c * I_c}$$

I_c = Composite Moment of Inertia (in^4)

Prestressing Force at Transfer

For 2 point harping; $\Delta_p = \dfrac{P_i}{E_{ci} * I_g}\left(\dfrac{e_{mid} * L^2 * 144}{8} - \dfrac{e_{mid} * a^2 * 144}{6} + \dfrac{e_{end} * a^2 * 144}{6}\right)$

For straight strand; $\Delta_p = \dfrac{P_i * e * L^2 * 144}{8 * E * I_g}$

$P_i = P_0 - f_{ES} * A_{ps}$
f_u = strand ultimate strength (ksi)
A_{ps} = total area of strand (in^2)
$P_0 = 0.75 * f_u * A_{ps}$

a = distance from end of girder to harp point (ft)

$e_{end} = Y_b - cg_{end}$ — Y_b = centroid of girder from bottom (in)

$e_{mid} = Y_b - cg_{mid}$ — cg = strand center of gravity (in)

$$f_{ES} = \frac{\dfrac{P_0}{A_g} + \dfrac{P_0 * e_{mid}^2}{I_g} - \dfrac{M_{girderDL} * e_{mid} * 12}{I_g}}{\dfrac{E_{ci}}{E_{ps}} + \dfrac{A_{ps}}{A_g} + \dfrac{A_{ps} * e_{mid}^2}{I_g}}$$

At Release:

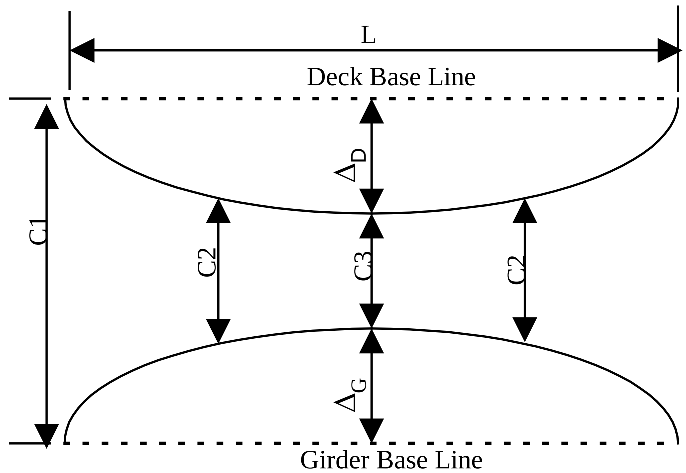
$$\Delta_{release} = \Delta_g + \Delta_p$$

At Slab Placement:

$$\Delta_{erection} = 1.65 * \Delta_g + 1.55 * \Delta_p$$

CAMBER STRIP

Both the deck and girder profiles can be assumed to be parabolic curves, either above or below a base line. The base line for the deck is a straight line between the point at the beginning of the span to the point at the end of the span on the underside of the deck. The base line for the girder is the straight line between the point at the beginning of the span to the point at the end of the span on the top of the girder.



C1 = Camber strip thickness at ends of girder
C2 = Camber strip thickness at ¼ point of girder
C3 = Camber strip thickness at mid-span of girder

Δ_D and Δ_G must be first calculated to determine C1, C2, and C3. A positive value for Δ_D or Δ_G indicates the profile is above its respective base line.

$$\Delta_D = \Delta_{VC} - \Delta_{HC}$$

Δ_{VC} = Vertical curve effect (inches)
Δ_{HC} = Horizontal curve effect (inches)

$$\Delta_{VC} = \frac{1.5 * G * L^2}{VC}$$

G = algebraic difference in profile tangent grades (ft/ft)
+ for crest vertical curve
- for sag vertical curve
VC = vertical curve length (ft)
L = girder span length (ft)

$$\Delta_{HC} = \frac{1.5 * s * L^2}{R}$$

S = super-elevation rate (ft/ft)
R = radius of horizontal curve (ft)
L = girder span length (ft)

$$\Delta_G = 1.55\,\Delta_p - 1.65\,\Delta_g - \Delta_{SDL} - \Delta_{CDL}$$ (All values are absolute values)

If $\Delta_D > \Delta_G$; camber strip thickest at mid-span (Δ_D and Δ_G are algebraic values)

$$C_1 = F + s * \frac{W_f}{2}$$

F = minimum fabrication/construction tolerance value; ½” for spans up 80’ & 1” for spans over 80’.
S = super-elevation rate (ft/ft)
W_f = girder top flange width (inches)

$$C_2 = C_3 - \frac{C_3 - C_1}{4}$$

$$C_3 = C_1 + \Delta_D - \Delta_G$$

If $\Delta_G > \Delta_D$; camber strip thickest at ends (Δ_D and Δ_G are algebraic values)

$$C_3 = F + s * \frac{W_f}{2}$$

$$C_1 = C_3 + \Delta_G - \Delta_D$$

$$C_2 = C_3 + \frac{C_1 - C_3}{4}$$

If $\Delta_G = \Delta_D$;

$$C_1 = C_2 = C_3 = F + \frac{s * W_f}{2}$$

Girder Stirrups
The 4” standard projection of the girder stirrup into the deck slab should be checked. The variable depth of the camber strip may require the stirrup projection to be increased so the hook is above the bottom mat of slab reinforcement or decreased so the hook is below the top mat of slab reinforcement.

Revisions:

April 2008	Revised f'_{ci} to psi
July 2009	Added ksi units to definition of E_{ci} Added 144 units conversion to straight strand Δ_p equation. Added paragraph on girder stirrups.

CHAPTER 5
STANDARD DRAWING REVISION LOG

B5.2A AASHTO Type 2 Prestressed Girder

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.2B AASHTO Type 3 Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.2C AASHTO Type 4 Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.2D Prestressed AASHTO Girder Details

June 2006 Extended strand for the positive moment connection in the Type C Diaphragm to comply with Article 5.14.1.4 of the LRFD Manual.
Changed "Abutment Diaphragm Dowel Details" to "Girder End Details" and labeled Type C "Positive Moment Connection".

July 2009 Added Note 9 for size of strand used in the design.
Revised Note 5 for lateral restraint until deck is cured.

B5.2E Pier Diaphragm for AASHTO Girders

June 2006 Extended strand for the positive moment connection and widened the gap between girder ends to 10".

B5.3A 36" Bulb Tee Prestressed Girder

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3B 42" Bulb Tee Prestressed Girder

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3C 48" Bulb Tee Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3D 54" Bulb Tee Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3E 60" Bulb Tee Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3F 66" Bulb Tee Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3G 72" Bulb Tee Prestressed Girder

June 2006 Extended the 6" stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3H 84" Bulb Tee Prestressed Girder

June 2006 Added new standard drawing.

July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

CHAPTER 5
STANDARD DRAWING REVISION LOG

B5.3I 49" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3J 55" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3K 61" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3L 67" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3M 73" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3N 85" Bulb Tee Prestressed Girder
June 2006 Added new standard drawing for bulb tee with 48" top flange.
July 2009 Modified the end block reinforcement to comply with AASHTO Article 5.10.10.

B5.3O WF42G Prestressed Girder
July 2009 Added new standard drawing.

B5.3P WF50G Prestressed Girder
July 2009 Added new standard drawing.

B5.3Q WF58G Prestressed Girder
July 2009 Added new standard drawing.

B5.3R WF66G Prestressed Girder
July 2009 Added new standard drawing.

B5.3S WF74G Prestressed Girder
July 2009 Added new standard drawing.

B5.3T WF83G Prestressed Girder
July 2009 Added new standard drawing.

B5.3U Prestressed Bulb Tee Girder Details
June 2006 Renumbered the sheet and extended strand for the positive moment connection in the Type C Diaphragm to comply with Article 5.14.1.4 of the LRFD Manual.
Changed "Abutment Diaphragm Dowel Details" to "Girder End Details" and labeled Type C "Positive Moment Connection".
July 2008 Added Note 12 for size of strand used in the design.
July 2009 Renumbered the sheet from B5.3O to B5.3U.
Added Note 9 for size of strand used in the design.
Revised Note 5 for lateral restraint until deck is cured.

CHAPTER 5
STANDARD DRAWING REVISION LOG

B5.3V Temporary Diaphragm Details
June 2006 Renumbered the sheet.
July 2009 Renumbered the sheet from B5.3P to B5.3V.

B5.3W Pier Diaphragm for Bulb Tee Girders
June 2006 Renumbered the sheet, extended the strand for the positive moment connection, and widened the gap between girder ends to 10”.
July 2009 Renumbered the sheet from B5.3Q to B5.3W.

B5.4A Typical Deck Bulb Tee Section & Details
June 2006 Added new standard drawing.
July 2009 Corrected spelling of Bulb Tee in title block
Corrected wording in Note 6(“one” to “when”)
Added Note 14 placing girders perpendicular to the cross-slope.
Changed intermediate diaphragm reinforcement G9, G9A, D1, &D2 to #4
Specified G9 & G9A bars @ maximum 12” spacing

B5.4B Prestressed Deck Bulb Tee Girder
June 2006 Added new standard drawing.
April 2008 Defined “W”
Changed G14 to #4
Computed max. spacing for G11 & G12 flange reinforcement
Modified Reinforcement Diagram Sketch for G1, G5, G5A, G11, G13, & G14.
July 2009 Modified Reinforcement Diagram Sketch for G6 & G8
Added G9 & G9A bars to Reinforcement Diagram
Added spacing of intermediate diaphragm bars in the Elevation view
Added confinement length “Y” and Bursting Reinforcement length “Z” to End Reinforcement Detail

B5.4C Prestressed Deck Bulb Tee Girder Details
June 2006 Added new standard drawing.
July 2009 Added Note 11 for size of strand used in the design.
Added Notes 12 & 13 for deflection data
Modified Deflection Data Table for ΔD

B5.5F 2’-2” Prestressed Voided Slab
June 2006 Extended the 6” stirrup spacing in the end block to comply with AASHTO Article 5.10.10.2.

B5.5G Typical Prestressed Slab Details
June 2006 Corrected the reference in Note 10 to paragraph H to conform to the 2004 Standard Specifications.
July 2009 Added Note 19 for size of strand used in the design.

B5.7A Box Girder Details
June 2006 Renumbered the sheet.

B5.7B Post-tensioning Standard Details
June 2006 Renumbered the sheet.

B5.8 Electrical Connection to Top Reinforcing Steel
June 2006 Added new standard drawing.

6.7.2 DEAD LOAD CAMBER

Steel girders require cambering in order to compensate for the effects of roadway vertical curvature, dead load deflections and concrete deck shrinkage deflections. In order to maintain a constant dimension from the top of the bridge deck to the top of the girder web throughout the length of the girder, the web must be fabricated to a profile that matches the finished grade profile of the bridge, along the girder line, after all permanent design loads are applied to the girder. The camber information shall be shown on the plans at the tenth points of each span and at field splice points in both graphic (a camber diagram) and tabular form.

The first aspect of developing the camber diagram is to establish a baseline from which all camber dimensions can be measured. While any straight line, whether horizontal or sloped, is adequate for this purpose the preferred baseline is a line that connects a point at the top of the web at one end of a continuous girder to a point at the top of the web at the other end of the same girder. This line is represented on the camber diagram as a horizontal line. This baseline definition is preferred over other definitions because if the conditions stated below are met the resulting camber diagram will have zero offset from the baseline at both the beginning and end of the girder and the same camber values will be good for all girders in the cross-section, regardless of bridge skew angle.

- All girders in the cross-section are straight and the same length.
- The beginning or end of a vertical curve does not fall within the limits of the bridge (this is only critical if the bridge is skewed).
- No abrupt changes occur in the deck cross-slope on the bridge (such as the start or end of a super transition).

In those cases where the bridge geometry does not meet the above conditions the baseline may be defined, at the discretion of the engineer, as something other than what is described above. In such cases a separate camber table may be required for each girder. Curved girders will always require a separate camber table for each girder. In all cases the definition of the baseline shall be clearly indicated on the bridge plans.

It should be noted here that interior and exterior girders on straight bridges should be cambered the same even if the design dead loads are different. For camber purposes all dead loads should be distributed equally to all girders in a cross-section. If the exterior girders are cambered different from the interior it will not only be difficult to attach the cross-frames but it will be impossible to control the finished grade across the width of the deck with a screed machine supported on the exterior girders. If the screed rails are set for finishing to the exterior girders they will not be correct for finishing over the interior girders. The exterior and interior girders essentially deflect the same during deck placement due to the load distribution effects of the diaphragms.

The second part of developing the camber diagram is determining the amount of offset from the baseline to the finished position of the girder after all permanent loads have been applied. For bridges with no vertical curvature or abrupt changes in cross-section this offset will be zero the full length of the girder. Bridges fully or partially on vertical curves will have camber values varying from zero at the ends to a maximum value near the center of the girder (with positive values for crest and negative values for sag vertical curves). These values should be labeled in the camber table as vertical curve.

The third part of developing the camber diagram is determining the amount of deflection dead loads and deck shrinkage cause in the girder. There are four types of deflections to consider: the self-weight of the steel (non-composite load), the weight of the deck concrete (partial-composite load), the weight of all other dead loads such as parapet & future wearing surface (fully-composite load) and the effect of deck shrinkage over time (fully-composite load). Each one of these deflection types are calculated separately and shown in the camber table, a downward deflection is shown as positive camber and vice versa. The camber diagram shall only show the total of all camber values added together for a given point.

The calculation of the deflections for girder self-weight and the parapet & FWS are relatively straight forward, however the calculation of the deflections due to the deck concrete and shrinkage are more complex. It is ITD policy to determine deflections due to the deck concrete based on the pour sequence. The deflections of the first pour are based on the whole girder acting non-composite, then the deflections of the second pour are based on the area of the first pour acting compositely with the rest of the girder non-composite and so on until the last pour. The deflections due to each pour sequence are added together and only the total is shown in the camber table. For deck shrinkage deflections a method based on classical structure analysis is presented below. Other methods that utilize structural analysis software (such as LARSA) may also be used provided they are based on the same concrete shrinkage strain.

STEEL GIRDER CAMBER DUE TO DECK SHRINKAGE

In addition to dead load and vertical curvature, steel structures should also be cambered for the effect of concrete deck shrinkage. The deflection due to shrinkage should be calculated based on the shrinkage strain of the deck acting on the composite girder sections of the structure.

From Article 5.4.2.3.3 the shrinkage strain can be calculated as follows: $\varepsilon_{sh} = -k_{vs}\, k_{hs}\, k_f\, k_{td}\,(0.00048)$

where:
- ε_{sh} = shrinkage strain of plain concrete
- k_{vs} = factor for volume to surface ratio (assume 1.0 for decks)
- k_{hs} = humidity factor (1.16 for Idaho based on an average humidity of 60%)
- k_f = factor for concrete strength (1.0 for 4000 psi concrete)
- k_{td} = time development factor (0.99 after ten years)

Using these assumptions; $\varepsilon_{sh} = -0.00055$

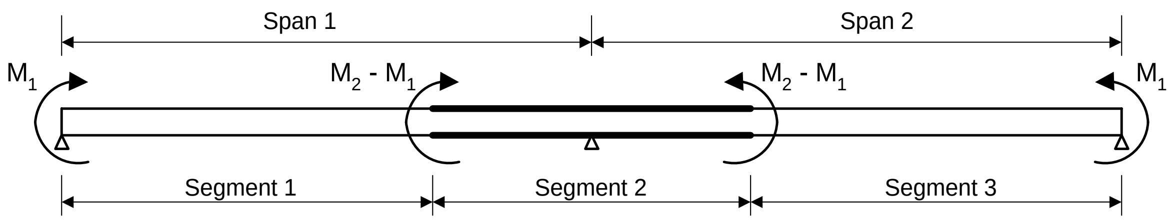
The shrinkage deflection of composite beams can be determined by substituting an equivalent compression force for the shrinkage strain, this force is applied to the ends of each girder segment at mid depth of the deck. For deflection purposes this eccentric compression load has the same effect as applying a positive moment to the ends of each girder segment (a segment being a length of girder with the same section properties). The magnitude of the applied moments is equal to the compression force times the distance from the mid depth of the deck to the c.g. of the composite section for that segment. Where two segments join the applied moment is the difference between the calculated moments for each segment. The equivalent force and moments are determined as follows:

$$F_{sh} = (1 - \rho)\varepsilon_{sh}\, AE/3 \qquad \text{(this expression is divided by 3 to account for creep)}$$

$$M_i = F_{sh}(cg_i - T/2)$$

where:
- F_{sh} = compression force required to produce a strain equal to ε_{sh}
- M_i = moment applied to each end of a particular segment to model the shrinkage effect
- A = tributary area of the deck per girder (use total area of deck divided by number of girders)
- E = modulus of elasticity of the concrete
- cg_i = center of gravity as measured from the top of a particular composite section
- T = thickness of the deck
- ρ = average ratio of longitudinal deck steel to deck area for a given girder segment

Example of a two span bridge with two different girder sections:

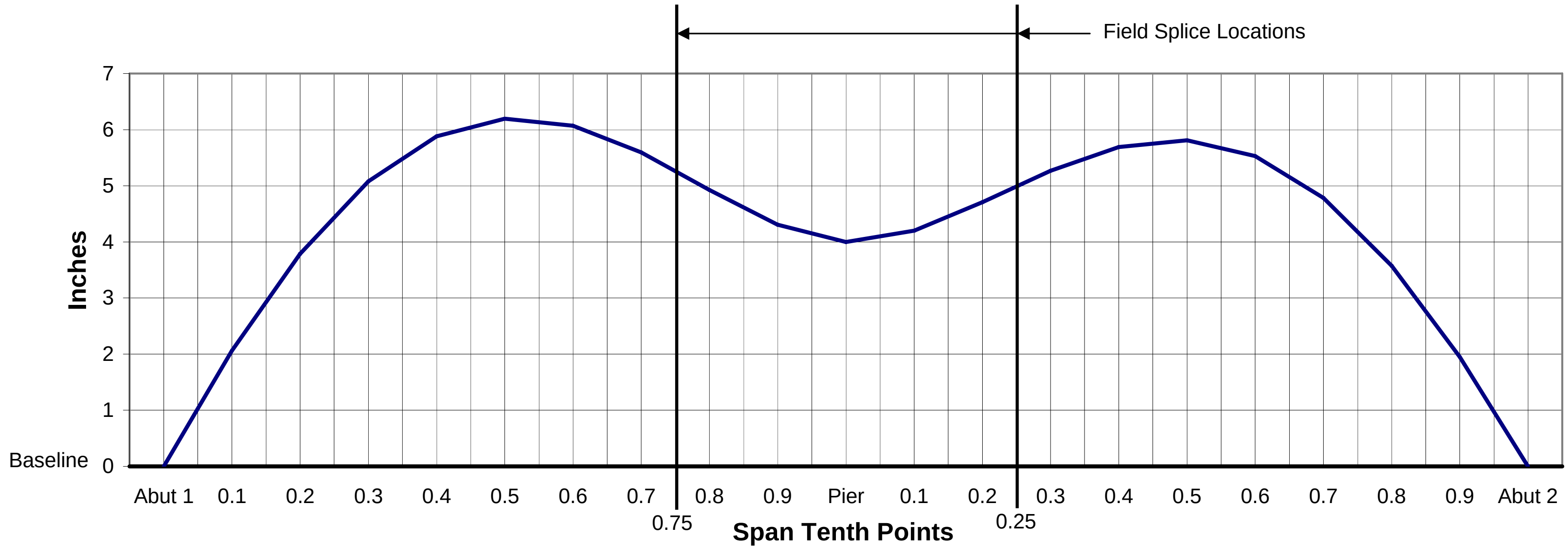


The composite “I” for the girder sections in this analysis should be based on “3n” to account for the long-term creep effect and, at the engineer’s option, the transformed deck section to take advantage of the deck reinforcement. The resulting deflections for this load case are equivalent to the shrinkage deflections. These deflections can then be added to the dead load deflections and combined with the vertical curvature corrections to determine the total camber. This method is appropriate for both single and multiple spans.

Revisions:

June 2006 Revised the shrinkage strain calculations to comply with the 2005 Interims.

Example of a Steel Girder Camber Diagram



	Span 1												Span 2										
Span Point	Abut 1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	Field Splice 0.75	0.8	0.9	Pier	0.1	0.2	Field Splice 0.25	0.3	0.4	0.5	0.6	0.7	0.8	0.9	Abut 2
Girder D.L.	0	0.18	0.34	0.44	0.49	0.47	0.40	0.29	0.22	0.16	0.05	0	0.05	0.16	0.22	0.29	0.40	0.47	0.49	0.44	0.34	0.18	0
* Deck D.L.	0	0.75	1.39	1.83	2.04	1.99	1.71	1.26	0.99	0.71	0.24	0	0.13	0.49	0.71	0.93	1.33	1.60	1.68	1.53	1.17	0.64	0
Shrinkage	0	0.20	0.32	0.37	0.36	0.31	0.24	0.16	0.12	0.08	0.02	0	0.02	0.08	0.12	0.16	0.24	0.31	0.36	0.37	0.32	0.20	0
All Other D.L.	0	0.16	0.30	0.40	0.44	0.42	0.36	0.26	0.20	0.14	0.04	0	0.04	0.14	0.20	0.26	0.36	0.42	0.44	0.40	0.30	0.16	0
Total Deflection Camber	0	1.30	2.35	3.04	3.32	3.20	2.71	1.96	1.52	1.09	0.35	0	0.24	0.87	1.24	1.63	2.33	2.81	2.97	2.74	2.14	1.19	0
Vertical Curve	0	0.76	1.44	2.04	2.56	3.00	3.36	3.64	3.75	3.84	3.96	4.00	3.96	3.84	3.75	3.64	3.36	3.00	2.56	2.04	1.44	0.76	0
Total Camber	0	2.06	3.79	5.08	5.88	6.20	6.07	5.60	5.27	4.93	4.31	4.00	4.20	4.71	4.99	5.27	5.69	5.81	5.53	4.78	3.58	1.95	0

* The deck dead load deflections are not symmetrical because they are based on a three part deck placement sequence.

6.10.3 Constructibility Considerations for Steel Plate Girder Bridges

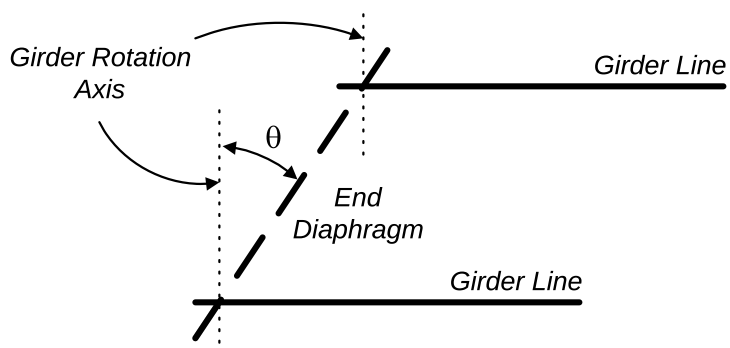
There are several constructibility issues that are unique to steel plate girder bridges that designers should account for in the design, plan preparation, and shop drawing review stages of a project.

Diaphragm Installation for Skewed Bridges:

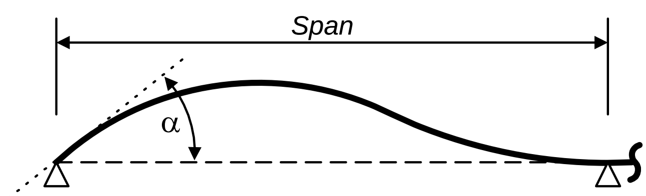
On straight non-skewed bridges all girders across the width of the bridge rotate at their bearings about a single axis line which is perpendicular to the girders, therefore as each girder is loaded all the girders rotate about a common line and the end diaphragms that connect the girders are unaffected. On skewed bridges each girder rotates about its own axis offset from adjacent girders, consequently as the girders are loaded and the girder ends rotate a distorting force is induced in the end diaphragms that connect the girders, since the diaphragms are very stiff diagonally and the girders are relatively flexible torsionally the diaphragms will twist the girders, pulling the web out of plumb.

The angle that the girder web is twisted out of plumb can be calculated as follows:

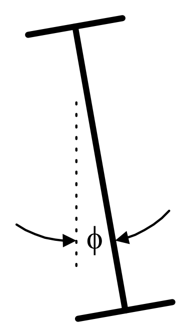
$\phi = \alpha \tan(\theta)$ where: ϕ = the out-of-plumb angle
θ = the bridge skew angle
α = the angle of the end of the girder due to the dead load camber (other than girder self weight)



Plan View



Elevation View of Camber



Girder End View

While there is no way to prevent girder twisting without the complete removal of diaphragms, when and how the girders twist can be controlled by the way the girders are detailed and fabricated. If the girders and diaphragms are detailed and fabricated for the diaphragms to fit the initial position of the girders, before the bridge deck is placed, then the girders will be plumb when the erection is complete. However, after the deck is placed, the girders will be twisted permanently in their final position, the girders will not sit level on the bearings and high distortional stresses will be locked into the diaphragms and girders. The only advantage to this method is that the girders and diaphragms fit initially, making it easier for the contractor to assemble.

On the other hand, if the girders and diaphragms are detailed and fabricated for the final position then the girders will need to be twisted out of plumb initially in order to get the diaphragms installed. However, after the deck is placed, the girders will be plumb for their final permanent position with a minimum amount of permanent distortional stresses in the diaphragms and girders. Standard practice for ITD is to detail diaphragms for the final position. Since some fabricators detail for the initial fit all shop drawings should be carefully checked to ensure they conform to the plans and the design. It is also good practice to specify the direction of the deck placement on the plans. The girders will initially be out of plumb to the greatest extent at the ends of the girders so the deck placement should progress from the dead load inflection point of the span toward the end of the girders so that the girders are near plumb by the time the placement reaches the girder ends.

There is a similar effect from intermediate diaphragms on skewed bridges. Because intermediate diaphragms are typically detailed to attach to the girders such that the diaphragms are perpendicular to the girder lines, they connect adjacent girders at slightly different span points. Since the amount of camber is different at each of these points the girders will need to twist initially out of plumb if the diaphragms are detailed to fit in the final deflected position. But, like the end diaphragms, once the deck has been placed the girders at the intermediate diaphragms will be in the correct upright position.

Camber Differences of Interior and Exterior Girders:

Interior and exterior girders for a given span are often designed for different amounts of dead load due to different tributary areas. This can result in a difference in the dead load camber shown in the plans for interior and exterior girders. However, if the girders are fabricated with different cambers it may become difficult to install the diaphragms during erection, but more importantly the deck may not be finished to the desired thickness across the full width of the bridge. Typically the screed rails for deck finishing machines are supported by the exterior girders, therefore if the exterior girders have more or less camber than the interior girders the deck will either be screeded too thick or too thin over the interior girders since the girders will not yet be fully loaded and therefore not in their final deflected position at the time the screed is finishing the deck. Even though it is appropriate to design the interior and exterior girders for different dead loads for the strength limit state, the dead load camber should be based on the dead load distributed equally to all girders so that all girders are cambered the same. This is really more representative of how the girders will actually deflect during construction as they are all connected to each other by diaphragms.

Bearing Alignment and Changes in Girder Length:

Bearings are often designed to accommodate the movement of girders for such effects as temperature change and live load rotations, but bearings should also accommodate construction tolerances and the length change that occurs in the bottom flange of girders between the initial unloaded condition and the final fully loaded condition.

While there are two construction tolerances to consider: 1) girder length, and 2) bearing placement on abutments and piers, there are also two parts to bottom flange lengthening: 1) the elastic lengthening due to the strain in the bottom flange resulting from the applied loads, and 2) the geometric change in the effective horizontal length of the girder due to the flattening of the fabricated camber. Typically most of the lengthening is the result of elastic strain while the effect due to the flattening of the camber is usually minor and can be ignored in most cases, however camber flattening may become significant when there is a large amount of total camber constructed in the girder.

Accommodating the various factors that contribute to nonalignment of sliding bearings is relatively straight forward since the sliding surface can just be increased. However for elastomeric bearings without a sliding surface it becomes more difficult because the bearings need to be designed to accommodate the extra lateral displacement by increasing the elastomer thickness. This in turn can affect other bearing properties such as load capacity, stability and vertical deflection. To avoid potential problems due to these issues, rather than increasing the elastomer thickness, the contractor can be required by specification to jack the girders and reposition the bearings after all dead loads have been applied and the temperature is near the midpoint of the design range.

Lateral Deflections of Exterior Girders due to Deck Overhang Brackets:

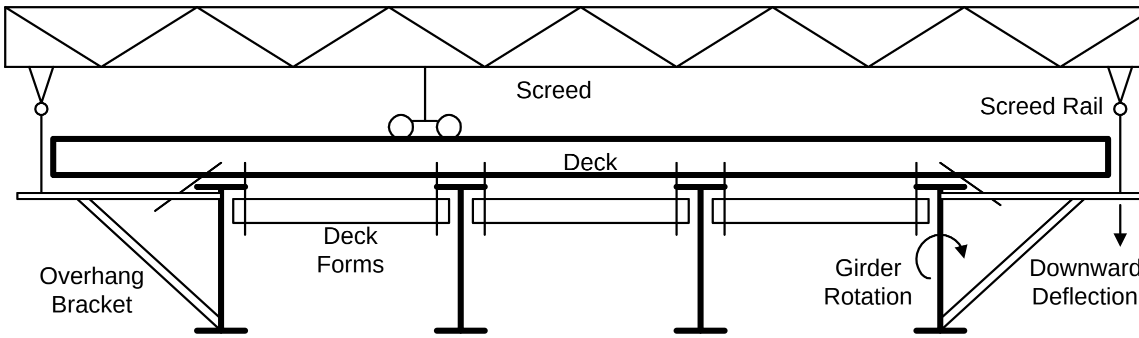
The construction of deck overhangs on steel girders is usually done with the use of standard deck overhang brackets. The use of these brackets introduces a torsional load into the exterior girders during placement of the concrete deck and the potential exists for the girder to rotate laterally, resulting in excessive overhang deflections. This can adversely affect finished grades when the screed rail is placed at the end of the overhang, the resulting deflection causes the screed to finish the deck lower than anticipated across the full width of the bridge. In order to minimize this problem all steel girder bridges should be checked for exterior girder rotational deflections due to overhang brackets. Anytime the calculated deflection at the edge of the overhang due to torsional effects exceeds 0.20" a revision should be made in the plans to reduce the deflection, such as closer diaphragm spacings, a larger top girder flange, or smaller overhangs. When the structural details cannot be altered temporary girder bracing should be detailed in the plans to resist the rotational effect. For more information on analyzing torsional loading refer to Article 6.10.3.4 and Appendix 6.1 of the Bridge Design LRFD Manual or to the Kansas Torsional Analysis of Exterior Girders method found on the Bridge Design X-Drive or available from KDOT.

Revisions:
April 2008 Added new article

6.10.3 CONSTRUCTIBILITY

LATERAL GIRDER ROTATION

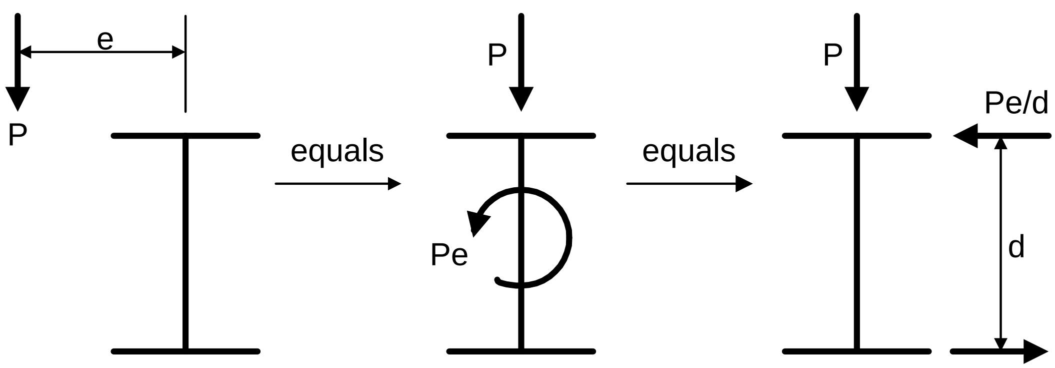
The construction of deck overhangs on steel girders is most often accomplished with the use of standard overhang brackets. The use of these brackets introduces a torsional load into the exterior girders during placement of the concrete deck and the potential exists for the girder to rotate, resulting in excessive overhang deflections. This can adversely affect finished grades when the screed rail is placed at the end of the overhang, the resulting deflection causes the screed to finish the deck lower than anticipated. In order to minimize this problem all steel girder bridges should be checked for rotational deflection.



The magnitude of the deflection at the edge of the deck in relation to the exterior girder is dependent on several factors; rotational stiffness of the girder, diaphragm spacing, amount of deck overhang and screed weight and location.

The calculation of actual rotational stiffness of a welded I-girder is complex, and is made more difficult by the use of diaphragms, the change in flange sizes and the use of unsymmetrical sections. However the problem can be simplified by assuming that any applied torsion acts as a couple with one component of the couple applied to the top flange and the other opposing component applied to the bottom flange. The flanges can then be assumed to carry their respective loads as independent beams in the lateral direction (neglecting the effect of the web). The sum of the two resulting deflections divided by the depth of the girder is the amount of rotation at any given location. This method is conservative and results in rotations approximately 10% greater than would be calculated from a more precise torsional stiffness analysis.

The flanges can be assumed to be continuous beams along the length of the structure with the diaphragms acting as supports. As a result the diaphragm spacing becomes the effective span when calculating the lateral deflections in each flange. The diaphragms should also be checked to be sure they are structurally adequate to carry the resulting reactions.



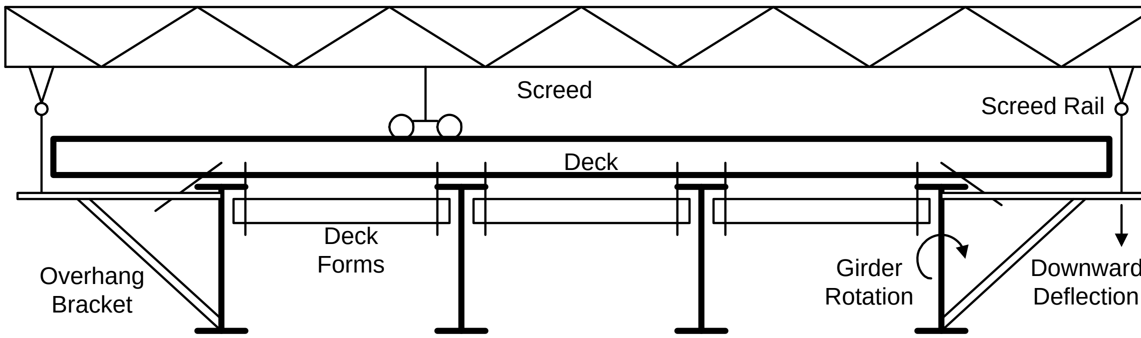
The couple that is applied to the flanges can be calculated from the net eccentric vertical loading applied to the exterior girders. A typical construction detail involves an overhang bracket on the exterior side of the girder and a beam hanger on the interior side. The loads from this arrangement are multiplied by their respective moment arms as measured from the centerline of the girder and algebraically added together resulting in the applied torsional moment. This moment is then divided by the girder depth and the resulting load applied to both the top and bottom flange to form a couple.

The flanges are then checked for the greatest deflection case and the resulting deflections for the top and bottom flange are added together and divided by the girder depth, which will give the rotation in radians. This value is then multiplied by the overhang dimension giving the amount of deflection that will occur at the end of the overhang. The deflection at the overhang should not be greater than 0.20 inches.

6.10.3.4 DECK PLACEMENT

LATERAL GIRDER ROTATION

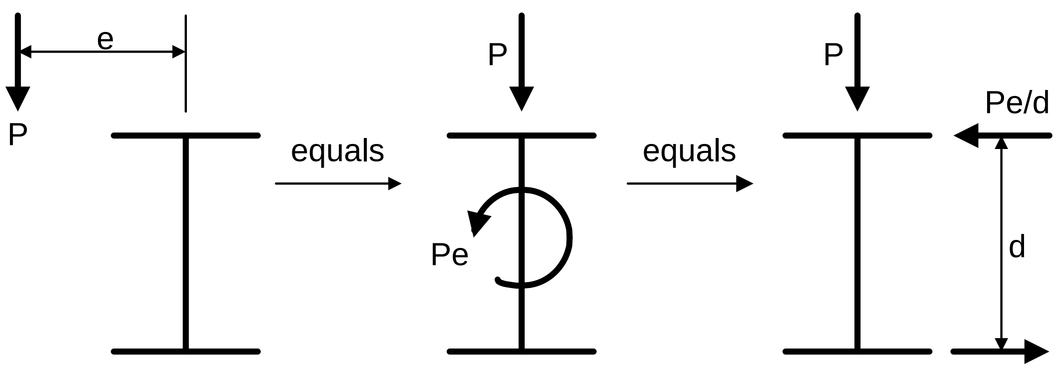
The construction of deck overhangs on steel girders is most often accomplished with the use of standard overhang brackets. The use of these brackets introduces a torsional load into the exterior girders during placement of the concrete deck and the potential exists for the girder to rotate, resulting in excessive overhang deflections. This can adversely affect finished grades when the screed rail is placed at the end of the overhang, the resulting deflection causes the screed to finish the deck lower than anticipated. In order to minimize this problem all steel girder bridges should be checked for rotational deflection.



The magnitude of the deflection at the edge of the deck in relation to the exterior girder is dependent on several factors; rotational stiffness of the girder, diaphragm spacing, amount of deck overhang and screed weight and location.

The calculation of actual rotational stiffness of a welded I-girder is complex, and is made more difficult by the use of diaphragms, the change in flange sizes and the use of unsymmetrical sections. However the problem can be simplified by assuming that any applied torsion acts as a couple with one component of the couple applied to the top flange and the other opposing component applied to the bottom flange. The flanges can then be assumed to carry their respective loads as independent beams in the lateral direction (neglecting the effect of the web). The sum of the two resulting deflections divided by the depth of the girder is the amount of rotation at any given location. This method is conservative and results in rotations approximately 10% greater than would be calculated from a more precise torsional stiffness analysis.

The flanges can be assumed to be continuous beams along the length of the structure with the diaphragms acting as supports. As a result the diaphragm spacing becomes the effective span when calculating the lateral deflections in each flange. The diaphragms should also be checked to be sure they are structurally adequate to carry the resulting reactions.



The couple that is applied to the flanges can be calculated from the net eccentric vertical loading applied to the exterior girders. A typical construction detail involves an overhang bracket on the exterior side of the girder and a beam hanger on the interior side. The loads from this arrangement are multiplied by their respective moment arms as measured from the centerline of the girder and algebraically added together resulting in the applied torsional moment. This moment is then divided by the girder depth and the resulting load applied to both the top and bottom flange to form a couple.

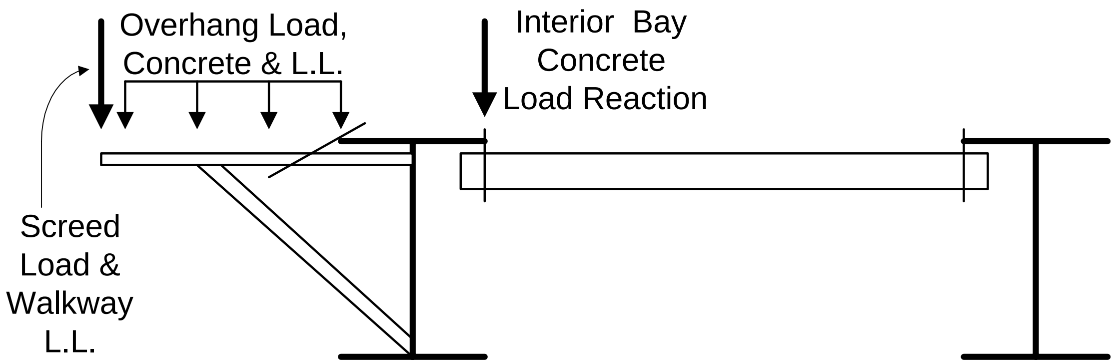
The flanges are then checked for the greatest deflection case and the resulting deflections for the top and bottom flange are added together and divided by the girder depth, which will give the rotation in radians. This value is then multiplied by the overhang dimension giving the amount of deflection that will occur at the end of the overhang. The deflection at the overhang should not be greater than 0.20 inches.

Revision:

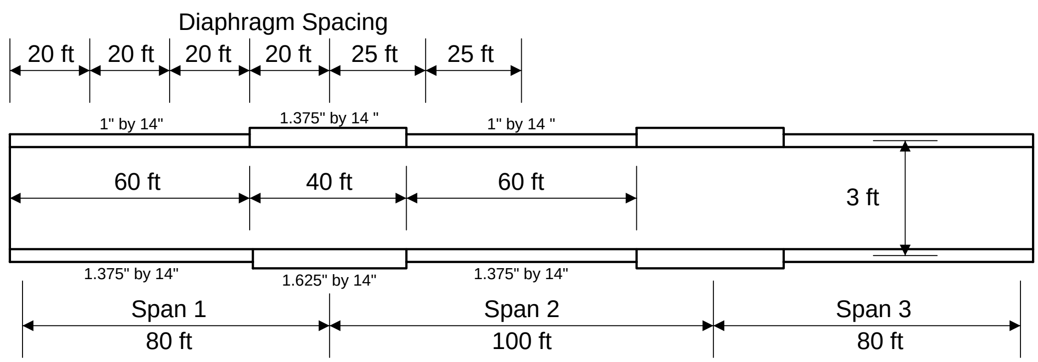
April 2008	Renumbered article from 6.10.3 to 6.10.3.4 and changed title to Deck Placement.

LATERAL GIRDER ROTATION

Example:



Assume a three span welded plate girder bridge with overhangs of 3.25 ft and a deck thickness of 8 inches. The spans are 80 ft –100 ft – 80 ft. The distance between the centers of the top and bottom flange is approximately 3.0 ft. The diaphragms are bent plates spaced at 20 ft in spans one and three and 25 ft in span two. The Girder spacings are 9.5 ft. The flanges vary as follows:



The top and bottom flanges actually act as twelve continuous sections for lateral load, however the remote sections have very little effect on deflections. Only two sections behind and one section in front of the section being checked need to be considered in the structural model, this limits the analysis to no more than four sections (see case 1 and 2 in this example).

There are four vertical loads that should to be considered for the rotational effects:

1. The deck overhang concrete load.
2. The deck interior bay concrete load reaction on the exterior girder.
3. The screed load on the overhang.
4. The construction live load and walkway load in the work zone.
   (The work zone is considered to be a 20 ft. long zone centered on the screed)

It is not necessary to consider the weight of the falsework because it is already in place when the form grades and screed rail settings are made.

In this example the deck weighs 100 lbs/ft^2, however an assumption must be made for the screed load. In this example the screed is assumed to weigh 4500 lbs per side spread out over 10 ft. The construction live load is 30 lbs/ft^2 and the walkway load is 75 lbs/ft.

The load per foot of length with associated moment arms, the resulting torsion and the equivalent lateral load applied to the flanges are as follows:

Deck Overhang – Load = 100(3.25 – (1.16/2)) = 267 lbs/ft (overhang minus half the flange width)
MomArm = (3.25 – (1.16/2))/2 + 1.16/2 = 1.92 ft
Torsion = 267(1.92) = 513 lbs*ft/ft
Lateral Load = 513/3 = 171 lbs/ft

Interior Deck – Load = 100{(9.5 – 1.16)/2} = 417 lbs/ft (girder spacing minus flange width divided by 2)
MomArm = 1.16/2 = 0.58 ft (one half the flange width)
Torsion = 417(0.58) = 242 lbs*ft/ft
Lateral Load = 242/3 = 81 lbs/ft

Total Lateral Concrete Loads = 171 – 81 = <u>90 lbs/ft</u> (applied to all sections behind and including the one being checked)

Screed Load – Load = 4500/10 = 450 lbs/ft
MomArm = 3.25 ft
Torsion = 450(3.25) = 1463 lbs*ft/ft

Screed Lateral Load = 1463/3 = <u>488 lbs/ft</u> (only applied to 10 ft at center of section checked)

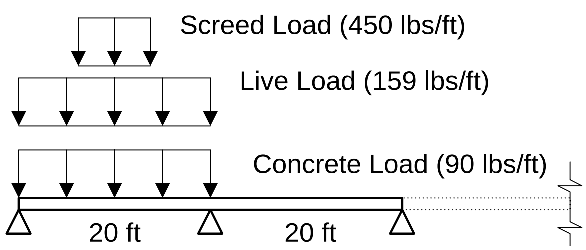
Uniform L.L. – Load = 30(3.25) = 98 lbs/ft (only applied to overhang area, a conservative assumption)
MomArm = (3.25)/2 = 1.63 ft
Torsion = 98(1.63) = 160 lbs*ft/ft
Lateral Load = 160/3 = 53 lbs/ft

Walkway L.L. – Load = 75 lbs/ft
MomArm = 3.25+1 = 4.25 (applied one foot beyond edge of deck)
Torsion = 75(4.25) = 319 lbs/ft
Lateral Load = 319/3 = 106 lbs/ft

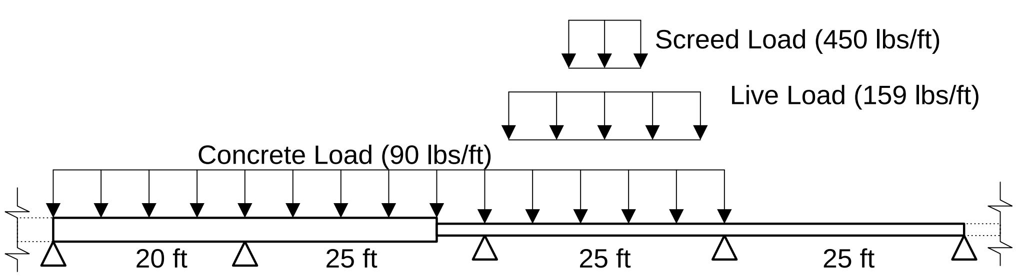
Total Lateral L.L. = 53 + 106 = <u>159 lbs/ft</u> (only applied to 20 ft at center of section checked)

The top flange has a moment of inertia of 229 in^4 or 314 in^4, depending on flange thickness. The bottom flange has a moment of inertia of 314 in^4 or 372 in^4. The deflection in this example is checked at two places, halfway between the abutment and first diaphragm (Case 1) and halfway between the fifth and sixth diaphragm, which is near the center of the second span (Case 2). The girder is assumed to be progressively loaded as the deck is placed with the screed located at the center of the last section to be loaded. For analysis purposes only one unloaded section and at most two loaded sections, other than the section being checked, need to be included when calculating the deflections. This is a conservative simplification; actual deflections will be slightly less.

Case 1



Case 2



Case 1: Deflection at midspan of the first section in the case 1 model:

Concrete Load Deflection Top Flange = 0.035 in
Screed Load Deflection Top Flange = 0.124 in
Live Load Deflection Top Flange = 0.061 in

Concrete Load Deflection Bottom Flange = 0.025 in
Screed Load Deflection Bottom Flange = 0.090 in
Live Load Deflection Bottom Flange = 0.044 in

Rotation = (0.035 + 0.124 + 0.061 + 0.025 +0.090 + 0.044)/36 = 0.0105 radians

Deflection at end of overhang = 0.0105(3.25)(12) = 0.41 in > 0.20 *Deflection is too great.*

Case 2: Deflection at midspan of the third section of the case 2 model:

Concrete Load Deflection Top Flange = 0.043 in
Screed Load Deflection Top Flange = 0.182 in
Live Load Deflection Top Flange = 0.100 in

Concrete Load Deflection Bottom Flange = 0.031 in
Screed Load Deflection Bottom Flange = 0.134 in
Live Load Deflection Bottom Flange = 0.074 in

Rotation = (0.043 + 0.182 + 0.100 + 0.031 + 0.134 + 0.074)/36 = 0.0157 radians

Deflection at end of overhang = 0.0157(3.25)(12) = 0.61 in > 0.20 *Deflection is also too great, either the diaphragm spacing will need to be reduced or a smaller overhang used.*

Once the deflections are satisfactory the diaphragm and its connection to the girder should be checked:

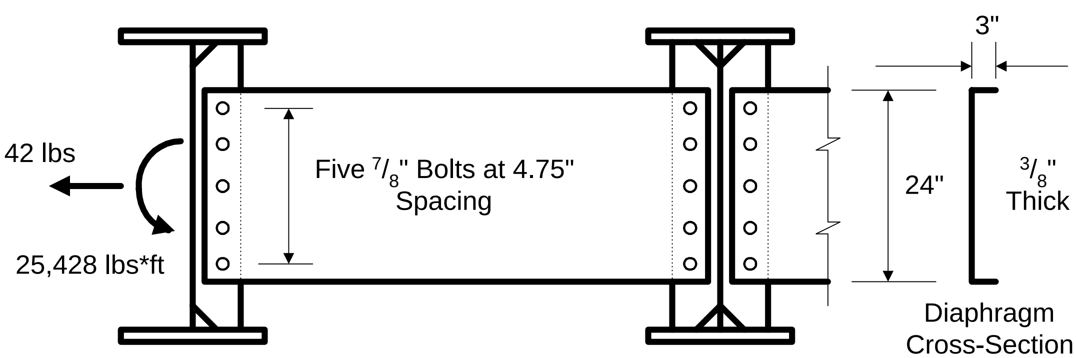
The greatest reaction at a diaphragm is in Case 2 just to the left of the section that has the screed load applied to it. The reaction loads at this diaphragm are as follows:

Top Flange Reaction– Concrete Load = 2400 lbs
Screed Load = 2790 lbs
Live Load = 1900 lbs

Bottom Flange Reaction– Concrete Load = 2410 lbs
Screed Load = 2760 lbs
Live Load = 1890 lbs

Assuming the diaphragm is centered on the girder web the net “LRFD Service II” slip critical moment and force on the bolt pattern is,

Net factored moment = (3/2){1.0(2400 + 2410) + 1.3(2790 + 1900 + 2760 + 1890)} = 25,428 lbs*ft
Net factored force = 1.0(2400 – 2410) + 1.3(2790 + 1900 – 2760 – 1890) = 42 lbs



The bolt pattern has a moment of inertia of 226 in^2 therefore the maximum bolt shear is:

25,428(12)(9.5)/226 + 42/5 = 12835 lbs

The nominal resistance of a $^7/_8$", A325 bolt for a standard hole with a Class A surface (Art. 6.13.2.8):

$K_h K_s N_s P_t$ = 1.0(0.33)(1.0)(39) = 12.87 kips 12.835 kips < 12.87 kips *OK*

The diaphragm has a section modulus of 63 in^3 and therefore the factored bending moment in the diaphragm and the associated stress for "Strength I":

Factored moment = (3/2){1.25(2400 + 2410) + 1.75(2790 + 1900 + 2760 + 1890)} = 33,536 lb*ft
Factored stress = 33,536(12)/63 = 6388 lbs/in^2

The factored resistance is based on a stress equal to the yield stress of 50 ksi.

6.388 ksi < 50 ksi *OK*

*** Revision of 11/2003 added uniform live load and walkway live load to overhang.**

CHAPTER 6
STANDARD DRAWING REVISION LOG

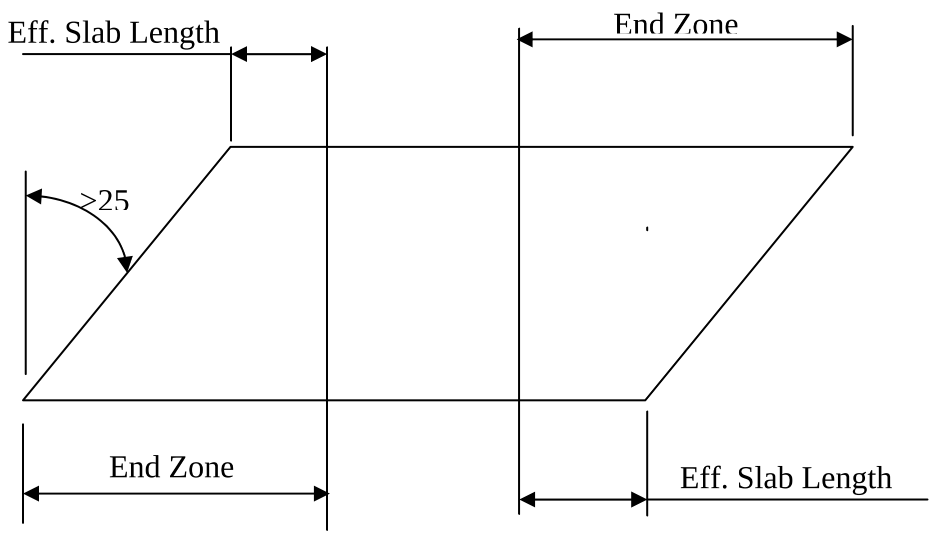
B6.1 Standard Steel Details

June 2006	Corrected weld symbol for bearing stiffener bottom flange/web weld. Added Jacking Stiffeners to title for Bearing Stiffeners. Made drip control bar gap around bottom flange equal to 1/16”. Added reference for locating shop splices to Assembly and Fabrication Details. Deleted the faying surfaces from Note 1 to conform to the requirements of 627.03 paragraph C of the Supplemental Specifications. Changed “field splices” to “bolted connections” in Note 3. Added Note 8 requiring girder web to be vertical under full dead load and deck shrinkage. Added Note 9 limiting shop splices.
April 2008	Note 2 revised to match SSP for painting structural steel. Note 3 added AASHTO M164 Type 3 and defined design assumption for threads in the shear plane.
July 2009	Revised Note 3 for threads in the shear plane.

9.7.2.5 REINFORCEMENT REQUIREMENTS FOR SKEWED END ZONES

For skews exceeding 25º, the specified reinforcement in both directions shall be doubled in the end zones.

The following diagram shall define the end zone.



9.7.2 EMPIRICAL DESIGN
Recommended Deck Thickness

AASHTO TYPE 2

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.0	8.00	2.00	3.33
5.5	8.00	2.00	3.33
6.0	8.00	2.00	3.33
6.5	8.00	2.00	3.33
7.0	8.00	2.00	3.33
7.5	8.00	2.00	3.33
8.0	8.00	2.00	3.33
8.5	8.00	2.00	3.33
9.0	8.00	2.00	3.33
9.5	8.50	2.13	3.54
10.0	8.50	2.13	3.54
10.5	8.50	2.13	3.54
11.0	9.00	2.25	3.75
11.5	9.00	2.25	3.75
12.0	9.50	2.38	3.96
12.5	9.50	2.38	3.96
13.0	9.50	2.38	3.96
13.5	10.00	2.50	4.17

AASHTO TYPE 3

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.0	8.00	2.00	3.33
5.5	8.00	2.00	3.33
6.0	8.00	2.00	3.33
6.5	8.00	2.00	3.33
7.0	8.00	2.00	3.33
7.5	8.00	2.00	3.33
8.0	8.00	2.00	3.33
8.5	8.00	2.00	3.33
9.0	8.00	2.00	3.33
9.5	8.00	2.00	3.33
10.0	8.50	2.13	3.54
10.5	8.50	2.13	3.54
11.0	9.00	2.25	3.75
11.5	9.00	2.25	3.75
12.0	9.00	2.25	3.75
12.5	9.50	2.38	3.96
13.0	9.50	2.38	3.96
13.5	10.00	2.50	4.17

9.7.2 EMPIRICAL DESIGN

Recommended Deck Thickness

AASHTO TYPE 4

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.0	8.00	2.00	3.33
5.5	8.00	2.00	3.33
6.0	8.00	2.00	3.33
6.5	8.00	2.00	3.33
7.0	8.00	2.00	3.33
7.5	8.00	2.00	3.33
8.0	8.00	2.00	3.33
8.5	8.00	2.00	3.33
9.0	8.00	2.00	3.33
9.5	8.00	2.00	3.33
10.0	8.50	2.13	3.54
10.5	8.50	2.13	3.54
11.0	8.50	2.13	3.54
11.5	9.00	2.25	3.75
12.0	9.00	2.25	3.75
12.5	9.50	2.38	3.96
13.0	9.50	2.38	3.96
13.5	9.50	2.38	3.96

MODIFIED BULB TEE

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.0	Deck can not be designed by the Empirical Design method		
5.5	Deck can not be designed by the Empirical Design method		
6.0	8.00	2.00	1.74
6.5	8.00	2.00	1.94
7.0	8.00	2.00	2.15
7.5	8.00	2.00	2.36
8.0	8.00	2.00	2.57
8.5	8.00	2.00	2.78
9.0	8.00	2.00	2.99
9.5	8.00	2.00	3.19
10.0	8.00	2.00	3.40
10.5	8.00	2.00	3.61
11.0	8.50	2.13	3.82
11.5	8.50	2.13	4.03
12.0	9.00	2.25	4.24
12.5	9.00	2.25	4.44
13.0	9.00	2.25	4.65
13.5	10.00	2.50	4.86

9.7.2 EMPIRICAL DESIGN
Recommended Deck Thickness

STEEL GIRDER W/12" TOP FLANGE

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.00	8.00	2.00	3.33
5.50	8.00	2.00	3.33
6.00	8.00	2.00	3.33
6.50	8.00	2.00	3.33
7.00	8.00	2.00	3.33
7.50	8.00	2.00	3.33
8.00	8.00	2.00	3.33
8.50	8.00	2.00	3.33
9.00	8.00	2.00	3.33
9.50	8.50	2.13	3.54
10.00	8.50	2.13	3.54
10.50	9.00	2.25	3.75
11.00	9.00	2.25	3.75
11.50	9.00	2.25	3.75
12.00	9.50	2.38	3.96
12.50	9.50	2.38	3.96
13.00	10.00	2.50	4.17
13.50	10.00	2.50	4.17

STEEL GIRDER W/15" TOP FLANGE

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.00	8.00	2.00	3.33
5.50	8.00	2.00	3.33
6.00	8.00	2.00	3.33
6.50	8.00	2.00	3.33
7.00	8.00	2.00	3.33
7.50	8.00	2.00	3.33
8.00	8.00	2.00	3.33
8.50	8.00	2.00	3.33
9.00	8.00	2.00	3.33
9.50	8.50	2.13	3.54
10.00	8.50	2.13	3.54
10.50	8.50	2.13	3.54
11.00	9.00	2.25	3.75
11.50	9.00	2.25	3.75
12.00	9.50	2.38	3.96
12.50	9.50	2.38	3.96
13.00	9.50	2.38	3.96
13.50	10.00	2.50	4.17

9.7.2 EMPIRICAL DESIGN

Recommended Deck Thickness

STEEL GIRDER W/18" TOP FLANGE

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.00	8.00	2.00	3.33
5.50	8.00	2.00	3.33
6.00	8.00	2.00	3.33
6.50	8.00	2.00	3.33
7.00	8.00	2.00	3.33
7.50	8.00	2.00	3.33
8.00	8.00	2.00	3.33
8.50	8.00	2.00	3.33
9.00	8.00	2.00	3.33
9.50	8.50	2.13	3.54
10.00	8.50	2.13	3.54
10.50	8.50	2.13	3.54
11.00	9.00	2.25	3.75
11.50	9.00	2.25	3.75
12.00	9.50	2.38	3.96
12.50	9.50	2.38	3.96
13.00	9.50	2.38	3.96
13.50	10.00	2.50	4.17

STEEL GIRDER W/24" TOP FLANGE

Girder Spacing Feet	ITD Slab Depth Inches	Min Overhang to center of girder w/continuous barrier Feet	Min Overhang to center of girder w/o continuous barrier Feet
5.00	8.00	2.00	3.33
5.50	8.00	2.00	3.33
6.00	8.00	2.00	3.33
6.50	8.00	2.00	3.33
7.00	8.00	2.00	3.33
7.50	8.00	2.00	3.33
8.00	8.00	2.00	3.33
8.50	8.00	2.00	3.33
9.00	8.00	2.00	3.33
9.50	8.00	2.00	3.33
10.00	8.50	2.13	3.54
10.50	8.50	2.13	3.54
11.00	9.00	2.25	3.75
11.50	9.00	2.25	3.75
12.00	9.00	2.25	3.75
12.50	9.50	2.38	3.96
13.00	9.50	2.38	3.96
13.50	10.00	2.50	4.17

<u>9.7.2 EMPIRICAL DESIGN</u>
Recommended Deck Thickness

[1] Section 9.7.2.4 Design Condition
The ratio of effective length to design depth shall not exceed 18

[2] Section 9.7.2.4 Design Condition
The ratio of effective length to design depth shall not be less than 6

[3] Section 4.6.2.2.2b Interior Beams with Concrete Decks
Table 4.6.2.2.2b-1 limits the maximum deck thickness to 12 inches

[4] Deck can not be designed by the Emperical Design method

CHAPTER 9
STANDARD DRAWING REVISION LOG

B9.1A Permanent Metal Deck Forms – Steel Girders

June 2006 Revised Note 1 to conform to the 2004 Standard Specifications..

Added new Note 4 requiring submittal of shop drawings & design calculations for the deck overhang brackets with the metal forms submittal..

Added new Note 5 requiring mylars of approved shop drawings.

Renumbered Notes.

Added “Styrofoam shall not be placed in the flutes” to new Note 6.

ASTM A-525 has been deleted and the reference to ASTM A-525 in Note 2 was removed.

Deleted Note 10 for not adding chlorides to the concrete because it is covered in the Standard Specifications.

B9.1B Permanent Metal Deck Forms – Concrete Girders

June 2006 Revised Note 1 to conform to the 2004 Standard Specifications..

Added new Note 4 requiring submittal of shop drawings & design calculations for the deck overhang brackets with the metal forms submittal..

Added new Note 5 requiring mylars of approved shop drawings.

Renumbered Notes.

Added “Styrofoam shall not be placed in the flutes” to new Note 6.

ASTM A-525 has been deleted and the reference to ASTM A-525 in Note 2 was removed.

Deleted Note 10 for not adding chlorides to the concrete because it is covered in the Standard Specifications.

DESIGN GUIDELINES FOR SPREAD FOOTINGS ON ROCK

The design of spread footing on rock shall be in accordance with the AASHTO LRFD Bridge Design Specifications as summarized in the following procedure.

General

On rock, a linear pressure distribution is used for all limit states. This is because the rock will not be able to yield to form a more rectangular pressure distribution. The deformation of the rock will basically stay linear. The flexibility of the footing will affect this, but the linear distribution is the closest distribution to use.

Service Limit State

- In the design of the footing reinforcement, the service limit state must be checked for crack control.
- Settlement does not need to be checked for footings on competent rock.
- As per Article 10.6.2.2.3d, elastic settlement on rock may generally be assumed to be less than ½”. If settlements of this magnitude are unacceptable and a settlement analysis is required, there are no eccentricity requirements.
- There are no sliding requirements.
- The resistance factor is 1.0.

Strength Limit State

- The eccentricity requirements should be in accordance with Article 10.6.3.2.5.
- The resistance factors should be provided by the Geotechnical Engineer based upon his method of estimating the capacity of the rock.
- Sliding should be checked in accordance with Article 10.6.3.3. For equation 10.6.3.3-2, the friction angle shall be in accordance with Table 3.11.5.3-1.

Extreme Limit State

- The eccentricity requirements should be in accordance with Article 10.6.3.2.5 which limits the length of the pressure distribution under the footing is at least ⅜ the footing length or width. This is to keep the footing from rocking too much.
- Sliding should be checked in accordance with Article 10.6.3.3. For equation 10.6.3.3-2, the friction angle shall be in accordance with Table 3.11.5.3-1.

Seismic Design of Abutments

Article 11.6.5 allows the location of resultant of the reaction forces to within the middle ⅔ to middle 0.8 of the footing base.

Commentary

Spread Footings on Rock Design Guidelines established based on LRFD specifications and correspondence from Monte Smith of Sargent Engineers.

Revisions:

June 2006 Added new article.

10.7.1.2 PILE SPACING, CLEARANCES, AND EMBEDMENT

Spacing, clearances, and embedment of piles shall be in accordance with AASHTO Article 10.7.1.2, except as follows:

1. If there is calculated uplift on a pile, positive means of anchorage and 12" of embedment shall be provided.

2. If there is a possibility of scour under footings, use 12" of embedment.

3. Use 24" embedment in the case of stubby abutments where the superstructure is integral with the pile cap and rotation is accomplished by bending in the piles.

4. For encased pile bents, the cut-off elevation should be a minimum of 1' above normal water elevation to avoid cutting off piles under water.

5. The concrete cover at the sides of piles in encased pile bents shall be a minimum of 9".

Pile spacing greater than 10' should be avoided. Consult your Group Leader on designs with proposed pile spacing exceeding 10'.

Revisions:

November 2006	Corrected the article number to agree with the 2006 Interims.
April 2008	Revised the concrete cover in note 5 from 8” to 9”.

10.7.1.5 PILE DESIGN REQUIREMENTS

The ultimate load capacity for driven steel H-pile or Pipe-pile shall not exceed:

1) The geotechnical capacity determined by an analysis of the foundation soils as recommended in the Foundation Investigation Report.

2) The structural capacity of the pile as determined by the Nominal Compressive Resistance per Article 6.9.4.1 of the AASHTO LRFD Design Specifications based on the pile acting as a column with the appropriate unsupported length.

3) 75% of the yield strength of the specified pile steel multiplied by the cross-sectional area of the pile.

Commentary

For the purpose of determining the structural capacity of the pile per Article 6.9.4.1the appropriate unsupported pile length for exposed pile bents should be based on the initial unsupported length plus the anticipated scour depth. For pile caps that are initially buried the appropriate unsupported pile length should be based on the anticipated scour depth below the bottom of the pile cap or seal concrete.

A limit of 75% of the yield strength of the pile is imposed to avoid potential pile damage during driving. For economy 36 ksi steel should be specified in most cases because the large ultimate loads that are possible with 50 ksi steel are very difficult to achieve and verify with standard pile driving equipment.

Revisions:
April 2008 Added new article.

10.7.3.11 GROUP LATERAL LOAD RESISTANCE

Pile group factored resistance for lateral loads shall be taken as:

$$Q_R = \eta \phi_L \Sigma Q_L$$

Where:

Q_L = nominal lateral resistance

For seat type abutments service limit state shall be limited to ¼ inch, and for Strength and Extreme Event Limit States shall be limited to 2”

ϕ_L = resistance factor[1]

For Service Limit State $\phi_L = 1.0$

For Strength Limit State $\phi_L = 0.9$

For Extreme Event Limit State $\phi_L = 1.0$

η = Group Efficiency Factor

$\eta = 0.75$ cohesionless soil

$\eta = 0.85$ cohesive soil

Note: for integral abutments see ..\11 Abutments Piers and Walls\11.6.1.2 Integral Abutments.doc

[1] These resistance factors have been approved by the Geotechnical Engineer

FOOTING LAYOUT AND PILE NOTES

The details and notes shown on B10.1 are intended as a design and detailing aid. Only those details and notes required should be shown on the plans. Notes should be modified or added to match the requirements of each structure.

Steel shell piles in a bent that are exposed shall have a minimum wall thickness of 5/16” with a useable wall thickness of ¼”.

Unless otherwise noted, exposed pile bents shall be painted in accordance with specifications.

For estimating purposes, test piles shall be approximately 50 percent longer than normal piles.

Revisions:

June 2006 Split Footing Layout & Pile Notes and Approved Points, Shoes, Boots, & Splicers into separate articles.

APPROVED POINTS, SHOES, BOOTS, AND SPLICERS

Approved points, shoes, boots, and splicers for pipe and H piles prepared by the Materials Section are listed in the Tables on the following pages. Manufacturers' names, addresses, and telephone numbers used in the tables are listed below.

MANUFACTURERS' NAMES, ADDRESSES, AND TELEPHONE NUMBERS

APF
Associated Pile & Fitting Corp.
Tel # 800-526-9047

DFP
Dougherty Foundation Products, Inc.
Tel # 201-337-5748

ICE
International Construction Equipment, Inc.
Products Manufactured by:
Mid-America Foundation Supply Inc.
Tel # 888-893-7453

PAI
Piling Accessories Inc
Part of APF

VERSABITE
Versabite Foundation Accessories
Part of APF

VERSA STEEL
Versa Steel, Inc. Pile Accessories Division
Tel # 800-678-0814

CONSTRUCTION SUPPLY COMPANY
Tel # 503-620-2971

APPROVED H-PILE ACCESSORIES

POINTS

POINT TYPE	APF	DFP	ICE	VERSA STEEL	CONSTRUCTION SUPPLY
Square tip end bearing in gravels & level rock	75500 VB-300 Series				
Common point for Gravels	77750-B PAR Series	H-777	HPH Series		
Common point with teeth for rock or gravels with boulders	77600-B VB-300P Series PAR Series	H-776	HPH-RB Series	VS-300 Series	HT-3300
Rock Point - slim section w/teeth	77750-B PAR Series	H-777	HPR-RB Series		

SPLICERS

APF	DFP	ICE	VERSA STEEL	CONSTR SUPPLY
HP-30000	HP-300	HSA Series	VS 400	HS-1000

APPROVED PIPE-PILE ACCESSORIES

60º CONICAL POINT

	APF	DFP	ICE	VERSA STEEL	CONSTRUCTION SUPPLY
Inside Fit	P-13006 VB-900 Series		60 HD Series	VS-900 Series	CP-9900
Inside Fit (Bullet Nose)	P-14006	P-77R			

OPEN-END CUTTING SHOE

	APF	DFP	ICE	VERSA STEEL	CONSTRUCTION SUPPLY
Inside Fit	O-14001 VB-700 Series	0140	ICE Inside Cutting Shoe	VS-700 Series	CS-7700

CLOSURE BOOT

	APF	DFP	ICE
	PB-20000	PB-170	ICE Round Tite Boot

SPLICERS

APF	DFP	ICE	CONSTRUCTION SUPPLY	VERSA STEEL
S-18000 S-20000 VB 800 Series	S-1800	Round Bite Coupler	PS 8800	VS800

Revisions:

June 2006 Split Footing Layout & Pile Notes and Approved Points, Shoes, Boots, & Splicers into separate articles

July 2009 Revised data based on Materials list of February 2009.

CHAPTER 10
STANDARD DRAWING REVISION LOG

B10.1 Footing Layout and Pile Notes

June 2006	Modified Note 4 to require prefabricated splicers to be welded in accordance with ANSI/AWS D1.1.
	Modified Note 5 to include Inspection and limited AWS D1.5 to piles with an OD greater than 18".
	Added Note 5a requiring all HP piles and shell piles ≤ 18" OD to conform to AWS D1.1.
April 2008	Corrected reference in Notes 3 & 4 to A10.2
July 2009	Deleted pile pay length detail
	Added Note 14 for estimated pile length shown in the Pile Schedule.
	Added Estimated Cut-off and Estimated Pile Tip Elevations in the Pile Schedule.
	Revised Note 5 to AWS D1.1 with 25% of all welds UT.

11.6.1.3 DESIGN GUIDELINES FOR INTEGRAL ABUTMENTS

Integral abutments, where appropriate, should be considered for most bridge projects in order to eliminate joints and bearings thereby simplifying construction and reducing maintenance problems. Integral abutments are defined as those abutments that are rigidly attached to both the superstructure and the supporting piles so that all thermal movements and girder end rotations are transferred from the superstructure through the abutment to the piles.

Design Limitations for the Use of Integral Abutments

Length: The total bridge length shall not exceed the following limits unless a more in-depth analysis is made.
Concrete Structures – 650 feet
Steel Structures – 350 feet

Skew: For bridge skews of 25 degrees or less, no skew effects need to be considered. For skews greater than 25 degrees the forces tending to rotate the structure shall be accounted for in the design.

Foundations: Abutments shall be supported on a single row of steel H-piles, steel smooth hollow pipe pile or steel-encased concrete piles utilizing smooth steel tubes. Piles should be embedded into the abutment concrete at least 2 feet. The preferred orientation of the piles is for bending about the strong axis. On skewed bridges the pile flanges should remain parallel with the abutment.

Abutment Thickness: The minimum thickness of the abutment wall should be 3 feet in order to provide enough width to encase the piles and girders.

Wing Walls: The wing walls should be cantilevered off of the abutments and shall be constructed parallel with the girders in order to minimize the soil pressure against the wings.

Pile Design Procedures

The number of piles in the abutment shall be based on the vertical load requirements. The total dead load reaction of the structure at the abutment shall be distributed to all piles equally. The live load reactions at each pile shall be determined by assuming the piles in the abutment act as a group:

$$R = \frac{P}{N} + \frac{Pex}{I}$$

where: R = single pile live load reaction (KIPS)
P = total live load reaction at the abutment, without impact (KIPS)
N = number of piles in the abutment
e = the eccentricity of the total live load relative to the center of the pile group (FT)
x = the distance from a given pile to the center of the pile group (FT)
I = the moment of inertia of the pile group (FT^2)

For vertical loads piles shall be designed in accordance with Article 10.7 of the AASHTO LRFD Bridge Design Specifications for Strength I.

Lateral loads on the piles may need to be considered on bridges skewed more than 25° when required to resist the soil pressure forces tending to rotate the structure.

Piles must be ductile enough to accommodate both thermal movements and dead load and live load rotations of the superstructure. The ductility of the piles may be checked using the following equations:

For Steel H-pile:

$$2\left[\frac{\Delta}{L} - \frac{M_pL}{6EI}\right] + \theta_w \le \frac{3C_iM_pL}{4EI} \qquad C_i = \frac{19}{6} - 5.68\sqrt{\frac{f_y}{E}}\frac{b_f}{2t_f}, \qquad 0 < C_i < 1.0$$

For hollow and concrete-filled pipe piles:

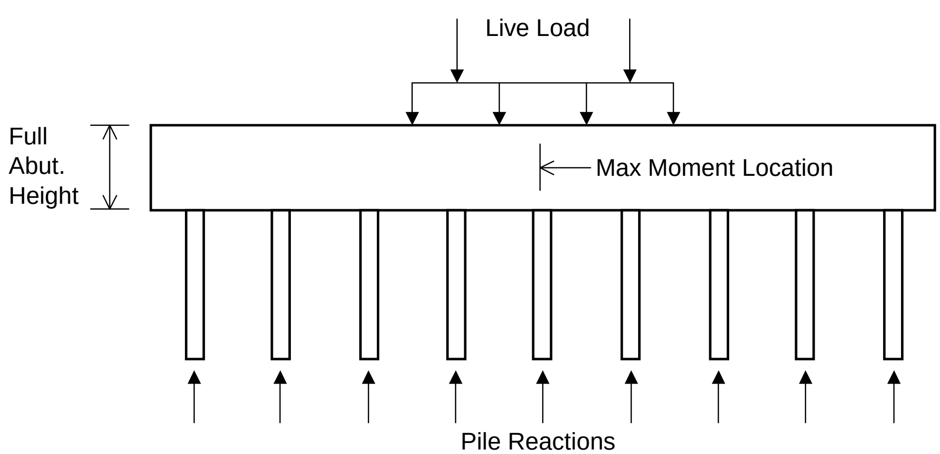
$$2\left[\frac{\Delta}{L}-\frac{M_pL}{6EI}\right]+\theta_w \le \frac{C_iM_pL}{2.08EI} \qquad C_i = 3.5-1.25\sqrt{\frac{f_y}{E}\frac{D}{t}}, \qquad 0 < C_i < 1.0$$

where:
- Δ = one half the factored thermal movement range at the abutment (IN)
- L = twice the length from the bottom of the abutment to the first point of zero moment in the pile determined taking into account the effect of the soil on pile behavior and assuming a lateral deflection of Δ (IN)
- M_p = plastic moment of the H-pile about the axis of bending or the plastic moment of the steel pipe pile without considering the concrete filling (KIP-IN)
- E = modulus of elasticity of the steel (KSI)
- I = H-pile moment of inertia about axis of bending, the moment of inertia of the hollow pipe, or moment of inertia of the concrete-filled pipe considering both the concrete and steel (IN^4)
- θ_w = maximum range of the factored angle of rotation of the superstructure at the abutment calculated assuming the structure is simply supported on the abutment (continuity of the superstructure over piers may be considered on multi-span bridges). This rotation is the sum of the rotations due to live loads plus all dead loads applied after making the rigid connection between the superstructure and the abutment assuming the loads are equally distributed to all girders (RAD)
- C_i = a ductility reduction factor for piles
- b_f = width of H-pile flange (IN)
- t_f = thickness of H-pile flange (IN)
- D = outer diameter of pipe pile (IN)
- t = thickness of pipe pile (IN)

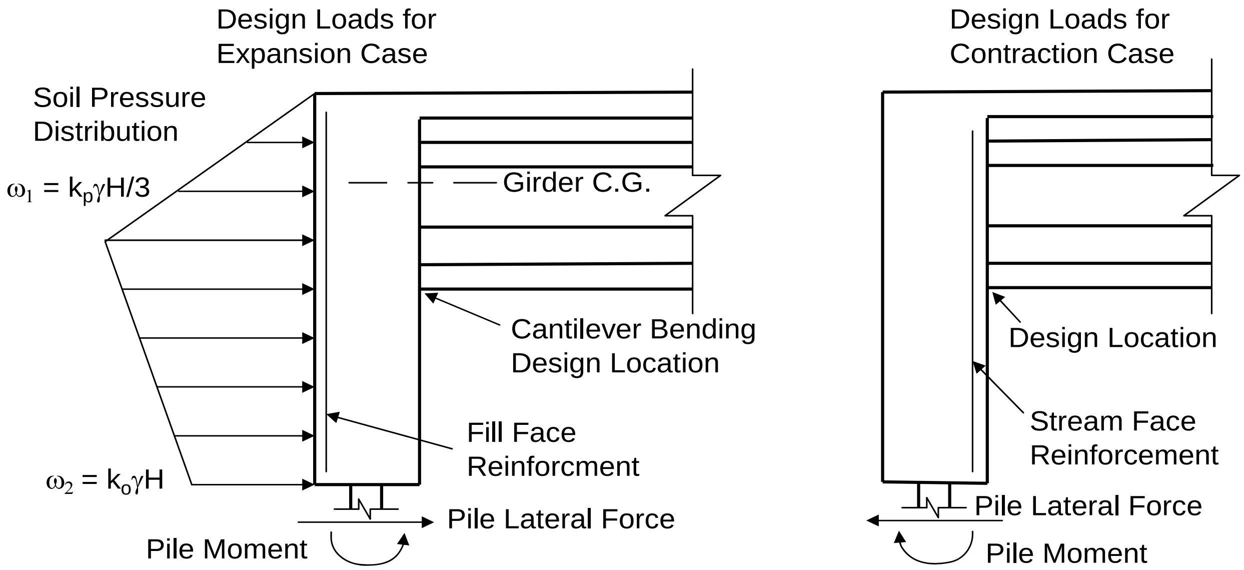
Abutment Backwall Design Procedures

The total height of the abutment shall be as short as practical with a minimum of 3 feet below the fill slope and 2 feet clear between the bottom of the girders and the fill slope. The abutment shall be designed for both Strength I and Service I using the following cases:

- Bending of the abutment acting as a horizontal pile cap for vertical dead loads and live loads assuming that all piles are loaded equally. The maximum positive moment (bottom steel) will occur approximately in the center of the abutment when the live load reactions are concentrated near the center; one, two, or possibly more lanes should be checked for positive moment. The maximum negative moment will occur approximately in the center of the abutment when two lanes are placed at the outside edges of the structure, two lanes will govern on abutments up to 80’ wide. For dead loads only the pile cap section of the abutment shall be considered effective and for live loads the full abutment (both pile cap and end diaphragm) may be considered effective.



- Bending of the abutment acting as a vertical cantilever below the beam seat elevation to resist soil pressures and lateral pile forces and moments for thermal expansion, and the lateral pile forces and moments only for thermal contraction.



- Bending of the abutment acting as a continuous beam spanning between the girders under a soil pressure load. The full section of abutment (both pile cap and end diaphragm) may be used.

For all design elements the soil pressure distribution may be assumed as the passive pressure for the top third of the abutment with the pressure varying linearly down to the at-rest pressure at the base of the abutment (strength I load factor for soil pressure of 1.00, 3rd Edition Article C10.5.5). This distribution is appropriate for concrete bridges up to 320 feet in length and steel bridges up to 120 feet in length. A more in-depth analysis of soil pressure distribution should be made for longer structures.

The abutment need not be designed by the strut and tie model if the longitudinal reinforcement is determined by conventional beam analysis for each of the above cases and a minimum orthogonal grid of reinforcing bars is provided in accordance with Article 5.6.3.6.

Girder End Capacity

The capacity of the end of the girders shall be checked for negative moment using Strength I for steel and concrete girders with an additional check of Service III for prestressed concrete girders. The design load shall be determined by assuming the maximum soil pressure (strength I load factor of 1.00), as determined above, acting on the back of the abutment combined with the lateral pile reaction and moment resulting from thermal expansion (strength I load factor of 0.5). The resulting compression force and bending moment taken about the c.g. of the composite girder section shall be applied to the end of the girders. The loads shall be distributed equally to all girders.

Also, for concrete girders, the tension force resulting from the lateral resistance of the piles due to contraction shall be used in the design of the longitudinal reinforcement of the girders in accordance with Article 5.8.3.5 (strength I load factor of 0.5).

Skew Affects

On structures with skews greater than 25 degrees, the tendency for the structure as a whole to rotate due to the non-concentric soil pressure forces acting against the abutments must be checked. A rotational moment about the center of the structure is induced by the soil pressure (strength I load factor of 1.00) acting on the abutment backwall at an angle normal to the backwall minus the soil-concrete interface friction angle;

$$\delta = \tan^{-1}(0.8 \tan \phi_f) \qquad \text{(Article 10.6.3.4)}$$

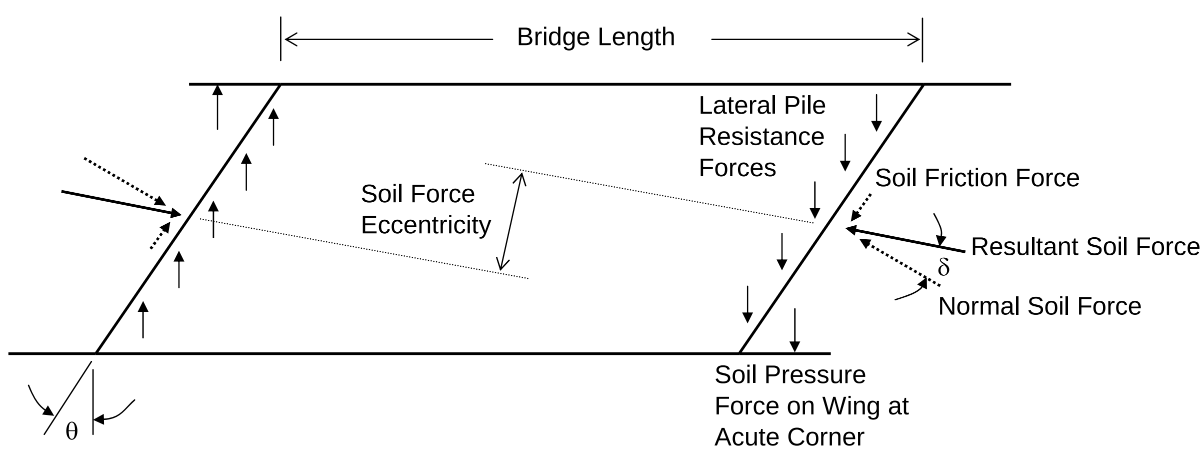
Where: δ = soil-concrete friction angle
ϕ_f = internal soil friction angle

If δ is greater than or equal to the bridge skew angle there will be no tendency for the structure to rotate and no further check is required. If δ is less than the bridge skew angle the magnitude of the induced rotational moment will be equal to the resultant soil force times the eccentricity of the opposing forces. The resultant soil force is based on the normal soil force (from assumed soil pressure distribution used for the abutment backwall design procedures) increased by the vector addition of the friction component. The eccentricity of these two opposing soil forces is equal to the bridge length times the sine of the difference between the bridge skew angle and the friction angle;

$$M_{RS} = F_{RS}(e_{RS}) \qquad F_{RS} = \frac{F_{NS}}{\cos\delta} \qquad e_{RS} = L\sin(\theta - \delta)$$

where: M_{RS} = the soil pressure induced rotational moment
F_{RS} = resultant soil force
e_{RS} = the eccentricity of the opposing soil forces
F_{NS} = normal soil force based on the assumed soil pressure distribution
δ = the soil-concrete friction angle
L = bridge length
θ = the bridge skew angle

The resulting rotational moment must be resisted by a combination of the lateral pile resistance (strength I resistance factor of 0.55, ITD policy), and the soil force acting against the inside face of the wings at the acute corners (strength I resistance factor of 0.50, Table 10.5.5.2.2-1), as indicated in the sketch below. It can be assumed for simplicity that the resisting forces in the piles as well as the soil force against the wing walls is applied at right angles to the bridge centerline and their respective eccentricities calculated accordingly.



Wing Wall Design

The wing walls shall be designed for at-rest pressure unless they are utilized to resist the rotational tendency of skewed bridges. In which case the wings at the acute corners shall be designed for the actual pressure required to stabilize the structure but need not exceed the passive pressure (strength I factor of 1.35).

Approach Slabs

Approach slabs shall be used on all integral abutment bridges with a total thermal movement of more than ¾ inch. This will result in approach slabs on all concrete girder bridges over 130 feet long and on all steel girder bridges over 65 feet long. The expansion joints at the ends of the approach slabs need only be sized to accommodate thermal expansion assuming the joints are constructed at 60º F with a maximum temperature of 80º F for concrete and 100º F for steel.

Commentary

The allowable bridge length for integral abutments was determined by limiting the total thermal movement range to 4”, which results in 2” of movement per abutment. Studies at Iowa State University in 1987 and at the University of Tennessee in 1996 have shown that for lateral displacements of H-piles up to 2” there is no reduction in vertical capacity of the pile. Therefore the only design requirements are vertical capacity and ductility, provided the piles remain embedded in the soil. (It is unlikely that a bridge would remain in service with an abutment that has scoured to the extent the piles become exposed for any significant length.) When the bridge is skewed more than 25 degrees there is also the design consideration for lateral capacity if it is assumed that the piles are used to resist skew induced rotation.

It is preferred to orient H-piles so that thermal expansion and contraction causes bending about the strong axis. While the resulting forces induced in the abutment will be higher, the pile itself will handle larger deflections without flange buckling. The forces and moments induced in the pile due to thermal deflections should be determined based on the soil-pile interaction assuming the pile head is fixed against rotation. This can be done using the COM624 program or a similar program that takes into account the soil properties. In order to simplify details and provide sufficient room for reinforcement the piles on skewed bridges should also be skewed so that the flanges remain parallel with the abutments.

Studies by the University of Tennessee in 1999 showed that piles with an embedment depth of 2 feet led to significantly lower stresses in the concrete and, in turn, to significantly less cracking at large values of lateral displacement when compared to piles embedded only 1 foot.

The equations used to determine pile ductility are from, “Rational Design Approach for Integral Abutment Piles,” Transportation Research Record 1233 (year 1983).

The assumed soil pressure distribution at the abutments, which is based on passive pressure for the top third of the abutment varying to at-rest pressure at the base, was developed by Maine as the result of studies done in 1992 where the pressure was monitored behind an integral abutment for one year. Because the appropriate maximum span length for this distribution is not known it is limited here to a deflection into the soil of a ¼ inch, assuming the abutments are constructed at 60° F.

The strut and tie model can become complicated for an abutment with a large number of piles and several models may be necessary for the different live load cases. Because the minimum reinforcement requirements of Article 5.6.3.6 almost always governs when checking the strut and tie model it is sufficient to design the main steel for bending using conventional analysis and then provide the orthogonal grid of minimum reinforcement required for strut and tie.

Skew affects do not need to be considered for skews less than 25 degrees. The soil-to-concrete friction angle at the abutment will always be at least 25 degrees; therefore soil friction alone will stabilize the structure. The ultimate lateral capacity of the piles, for the purpose of resisting rotational effects, shall be defined as the lateral force developed in the pile when subjected to a lateral displacement of 2”. As stated above this is the limiting displacement for which there is no reduction in vertical capacity of the pile.

Approach slabs are needed to prevent excessive compaction behind the abutments as well as provide an expansion joint that will minimize, if not prevent, the approach pavement from shoving, settling and cracking. It also isolates the joint away from the main structure where damage due to a leaking joint will be minimized. The joint width at the end of the approach slab where it rests on the sleeper beam should only be large enough to prevent the joint from completely closing during hot weather, with an allowance for the minimum compressed width of the joint seal material. It is not necessary to design the joint for the full movement range that would be required in a typical joint design.

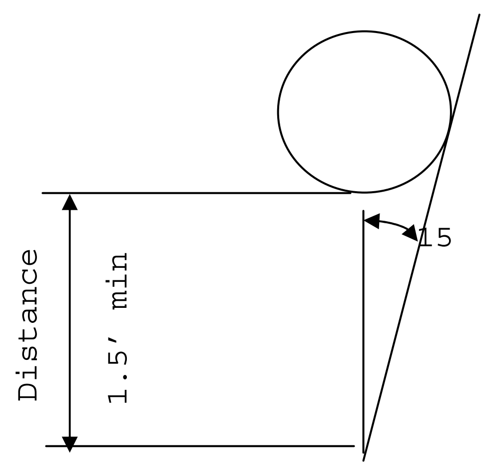
Revisions:
April 2008 Corrected references to agree with 2008 Interims.

11.10.11 MSE ABUTMENTS

Where interference between the piles and soil reinforcement occurs, the reinforcements must be designed around the piles, and the piles treated as backfill obstructions. A clear distance of no less than 1.5 feet from the back of the wall facing to the edge of the nearest pile or pile casing shall be provided.

ITD's Special Provision further stipulates that 15º is the maximum angle that reinforcement can be skewed from a line perpendicular to the wall.

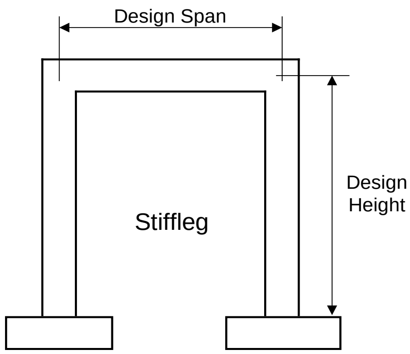
The layout of the abutment in relation to the MSE wall should take the above into account.

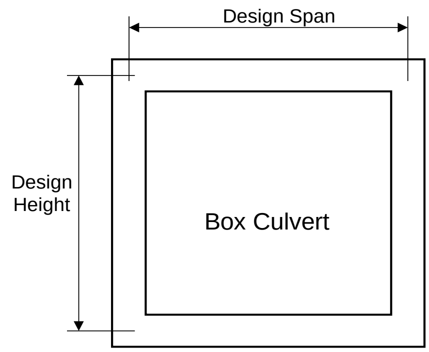


Casing Diameter -ft	Distance from casing to fill face of MSE panel - ft
1.0	1.50
1.5	2.15
2.0	2.86
2.5	3.58
3.0	4.30
3.5	5.01
4.0	5.77

DESIGN GUIDELINES FOR CAST-IN-PLACE STIFFLEGS AND BOX CULVERTS

The design of concrete cast-in-place culverts shall be in accordance with the AASHTO LRFD Bridge Design Specifications as summarized in the following procedure.





Maximum Span for Stiffleg = 25 feet

Maximum Span for Box Culvert = 15 feet

When the maximum span for the structure type is exceeded, a different structure type or multiple cells should be evaluated.

Class 40 concrete and Grade 60 reinforcement should be used on all designs. Class 40B should be used for the footings or floors, barrel walls and wing walls. Class 40A should be used for deck slabs with less than 2’ of fill and Class 40B when the fill is 2’ or greater.

When the distance between the finished grade and the top of deck is less than 4” between the paved shoulders the concrete cover over the top layer of reinforcement should be 2½” and all reinforcement within 4” of the surface should be epoxy coated. When the fill is greater than 4” but less than 2’ the cover over the top layer of reinforcement should be 2½” however no epoxy steel is required. When the fill is 2’ or greater the cover should be 2” with no epoxy steel.

Main reinforcement for skews greater than 25° (Article 9.7.1.3) should be placed perpendicular to the centerline of the culvert. For skews 25° or less, the main reinforcement should be placed on the skew and the design span measured along the skew.

Fillets should have a minimum leg of 6”.

Walls should be double reinforced and have a minimum thickness of 10”.

Slab thickness should be a minimum of 8” or (S +10)/30 whichever is greater (Table 2.5.2.6.3-1). The span length to determine the minimum thickness is the clear span between walls.

Construction joints with keyways should be placed transverse to the barrel and should be located in the walls, top slab, and bottom slab. Joint spacing should not exceed 40’. In lieu of a construction joint in the walls, the contractor may substitute an approved contraction joint.

The minimum footing width for stifflegs should be 2’ on rock and 3’ on other materials.

The design of culverts should meet the criteria for **Strength - 1** and **Service - 1** limit states.

Design Loads and Factors

(Table 3.4.1-1 & 2)	Strength – 1 Max. Factor	Strength – 1 Min. Factor	Service – 1 Factor
Concrete Member D.L.	1.25	0.9	1.0
Wearing Surface	1.50	0.0	1.0
Earth Fill D.L.	1.30	0.9	1.0
Earth Pressure (at rest, for barrel)	1.35	0.5 (Art. 3.11.7)	1.0
Earth Pressure (active, for wings)	1.50	0.9	1.0
Earth Surcharge (at rest, for barrel)	1.50	0.5 (Art. 3.11.7)	1.0
Earth Surcharge (active, for wings)	1.50	0.75	1.0
Live Load Surcharge	1.75	0.0	1.0
Live Loads	1.75	0.0	1.0
Water Pressure	1.00	0.0	1.0

Concrete D.L. Assume 150 lbs/ft^3 for the concrete and reinforcement (Table 3.5.1-1 & C3.5.1).

Wearing Surface Assume a future overlay of 6" with a weight of 140 lbs/ft^3 (Table 3.5.1-1).

Earth Fill D.L. Use the density given in the Phase IV report. When not given assume 140 lbs/ft^3 (rolled gravel from table 3.5.1-1). The earth loads should be modified to account for soil-structure interaction. The soil-structure interaction factor is as follows (Art. 12.11.2.2):

For embankment installations (the typical case),

$F_e = 1 + 0.2(H/B_c)$ F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section, or 1.40 otherwise.

Where, H = fill height above the deck surface
B_c = out-to-out dimension of the culvert span

For trench installations,

$F_t = C_b B_d^2/HB_c \le F_e$ Where, C_b = coefficient from figure 12.11.2.2.1-3
B_d = horizontal width of trench

Earth Pressure Shall be assumed to be linearly proportional to the depth of the soil based on the at rest pressure coefficient taken as $k_o = 1 - \sin \varphi_f$ where φ_f is the internal friction angle of the soil (Art. 3.11.5.2).

Earth Surcharge When the structure is buried the fill above the deck is considered an earth surcharge and a constant uniform horizontal earth pressure shall be applied in addition to the basic earth pressure. The uniform horizontal pressure due to earth surcharge should be based on the at rest coefficient k_o (Art. 3.11.6.1).

Live Load Surcharge At the barrel wall live load surcharge shall be determined as follows (based on table 3.11.6.4-1):

Up to 5' H:	h_{eq} = 4.0'
From 5' to 10' H:	h_{eq} = 4'- 0.2(H-5')
From 10' to 20' H:	h_{eq} = 3'- 0.1(H-10')
Higher than 20' H:	h_{eq} = 2.0'

Where h_{eq} is the equivalent height of soil in feet and H is the distance between the surface and the bottom of the footing in feet.

Live Loads Where the span exceeds 15 feet (Art. 3.6.1.3.3) the live loads shall be either a design truck in combination with a lane load or the design tandem in combination with a lane load (Art. 3.6.1.2). Where the span does not exceed 15 feet, only the axle loads of the design truck or design tandem shall be applied (Art. 3.6.1.3.3). The live load applied shall be a single loaded lane with the single lane multiple presence factor applied to the load (Art. 12.11.2.1).

Impact shall be $0.33(1.0 – 0.125D_E)$ but shall not be less than 0.0 where D_E is the minimum depth of earth cover over the structure (Art. 3.6.2.2). Impact shall only be applied to the truck or tandem (Art. 3.6.2.1).

Fill less than 2' (Article 4.6.2.10):

Case 1. Traffic Travels Parallel to the Span (normal case)

Analysis shall be based on a single loaded lane with the single lane multiple presence factor.

The axle load shall be distributed to the top slab for moment, thrust, and shear as follows:

$E = 96 + 1.44(S)$

$Espan = Lt + LLDF(H)$

Where E = lateral distribution width perpendicular to the span (inches)
Espan = longitudnal distribution length parallel to the span (inches)
S = the clear span length (ft)
Lt = length of the tire contact area parallel to the span, 10 inches (Art. 3.6.1.2.5)
H = depth of fill from top of culvert to top of pavement (inches)
LLDF = fill distribution factor of 1.15 or 1.00 (Art. 3.6.1.2.6)

The equivalent strip width used for the top slab should also be used for the walls and floor (Art. 12.11.2.3).

Shear strength may be considered adequate when the slab is designed by the above procedure (see commentary).

Case 2. Traffic Travels Perpendicular to the Span

Live load shall be distributed according to Article 4.6.2.1 for concrete decks with primary strips perpendicular to the direction of traffic.

Fill greater than 2':

Wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area increased by 1.15 times the depth of fill in granular backfill, or the depth of fill in all other cases. The lane load may be distributed over 12 feet in all cases. The tire contact area is 20 inches wide and 10 inches long. Where such areas overlap the total load shall be uniformly distributed over the area within the resultant perimeter. Multiple presence factors shall be used where applicable. Live load may be neglected where the depth of fill is over 8 feet and exceeds the total length of the structure. Where the live load moment based on distribution through fill depths more than 2' exceeds the moment calculated by Art. 4.6.2.10, the moment calculated by Art. 4.6.2.10 should be used. (Art. 3.6.1.2.6)

Shear design when fill is greater than 2' (Art. 5.14.5.3): $\phi V_c > V_u$ $\phi = 0.85$

$V_c = [0.0676(f_c`)^{0.5} + 4.6(A_s/bd_e)(V_u d_e/M_u)]bd_e$ (in kips)

But V_c shall not exceed $0.126(f_c`)^{0.5}(bd_e)$

where: A_s = area of tension steel (in^2)
d_e = effective depth from extreme compression to tension centroid (in)
V_u = shear from factored loads (kips)
M_u = moment from factored loads occurring simultaneously with V_u (kip-in)
b = design width (in)

For single span stifflegs and single cell boxes only, V_c need not be less than $0.0948(f_c`)^{0.5}(bd_e)$

V_u may be calculated at either the distance d_v (Art. 5.8.3.2) from the face of the wall or at the end of the fillet. d_v = center of compression force to center of tension force but not less than $0.9d_e$ or $0.72h$.

Water Pressure Culverts should be designed assuming static water pressure on the inside of the walls for the full design height.

Analysis The analysis should be based on the equivalent strip method assuming a rigid frame fixed against lateral movement at the base and free to side-sway at the top (classical force and displacement method). The design span length and wall height should be based on the centerline-of-member to centerline-of-member dimensions.

For simplification in determining the shears and moments in the structure the foundation soil pressure on box culverts may be considered to be uniformly distributed across the floor for all load cases (ITD Bridge Section policy).

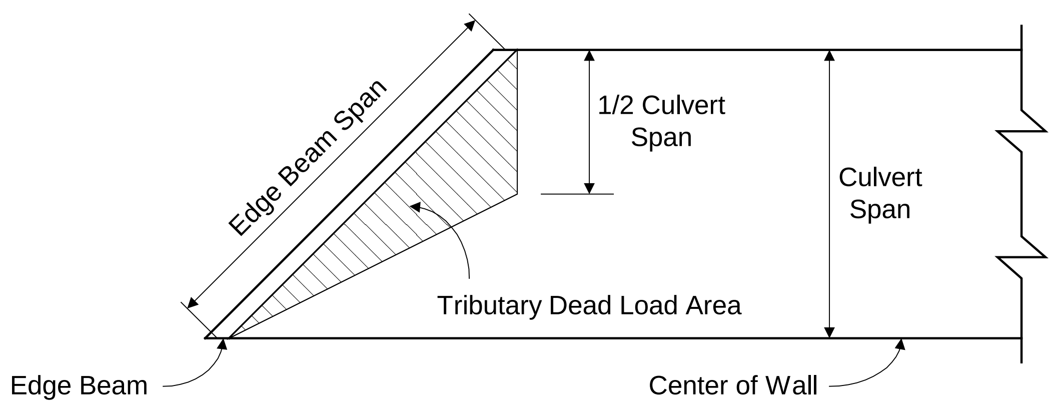
Where monolithic haunches inclined at 45° are used the negative reinforcement in the walls and slabs may be proportioned based on the flexural moment at the intersection of the haunch and uniform depth member (Art. 12.11.4.2).

Reinforcement

Minimum Reinforcement (Art. 5.7.3.3.2);	Min A_s shall be sufficient to resist $1.2M_{cr}$ or $1.33M_u$ whichever is less	
Transverse Distribution Reinforcement; (Art. 5.14.4.1)	$A_{dist} = A_s/(L)^{0.5}$	but no more than $0.5A_s$
	where A_s is the required positive moment reinforcement and L is the span length in feet	
For crack control (Art. 5.7.3.4);	$\gamma_e = 0.75$	to be used in equation 5.7.3.4-1

Note: When calculating the Service-1 limit state stresses in the reinforcement for the purpose of satisfying crack control requirements the compression thrust forces in the culvert members may be considered in order to take advantage of the resulting reduction in tensile stresses. The equations presented in the commentary for Article 12.11.3 may be used for this purpose.

Edge Beam Design The Live Load on edge beams shall be one line of wheels (either truck or tandem) plus a tributary portion of the lane load (Art. 4.6.2.1.4b). The tributary portion of the lane load shall be considered to be a uniform load of 64 lbs/ft^2 on either the effective edge beam width, or in the case where the edge beam is skewed relative to the main slab reinforcement, on the same tributary area as defined for dead load. The effective edge beam width for culverts with main slab reinforcement parallel with the edge beam shall be the distance between the edge of deck and the inside face of the barrier or curb, plus 12", plus one-quarter of the strip width, E, determined above. The effective width shall not exceed either one-half the full strip width or 72" (Art. 4.6.2.1.4b). The dead load should be the weight of all structure components and the fill on the effective width. When the end of the culvert is skewed relative to the main slab reinforcement the dead load applied to the edge beam shall also include the weight of all loads on the tributary area at the end of the culvert as shown below. These loads may be applied to the edge beam as a uniform load. The edge beam should be designed as a simple span with a span length based on center-of-wall to center-of-wall along the skew.



Footing Pressures The dead load footing pressures on the footings of stifflegs and the floor of box culverts may be uniformly distributed to the total footing area. The live load footing pressures may be assumed to act on a length of footing equal to the width of the design lane plus 1.15 times the distance from the surface to the bottom of the footing (this is the same distribution rate as the live load on the fill over a buried structure). Overlapping areas from more than one lane loaded shall be uniformly distributed over the length of the overlapping regions, multiple presence factors shall be applied where applicable. The total design loaded length shall not exceed the actual footing length. The footing pressure should be assumed to be uniform across the width of the footing or floor. Foundation design shall be in accordance with Section 10 of the LRFD Code.

Wing Wall Design

Live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall. H_{eq}, the equivalent height of soil in feet, should be based on Table 3.11.6.4-2.

The typical wing wall on a spread footing is free to deflect at the top under soil pressure and may therefore be designed using soil pressures based on the active state. The soil pressure coefficient, k_a, may be determined by the Coulomb method from the information in the Phase IV report.

Overturning:

For foundations on soil, the location of the resultant of the factored strength-1 and extreme event-2 (vehicle collision at rail, if present) forces shall be within the middle one-half of the base. On rock the resultant shall be within the middle three-fourths (Art. 11.6.3.3).

Sliding:

For footings on cohesionless soils the factored resistance against failure by sliding may be taken as (Art. 10.6.3.4):

$R_R = \varphi_T(V \tan \varphi_f)$ where:
R_R = factored resistance
φ_T = 0.8 for strength-1 (from table 10.5.5.2.2-1)
φ_T = 1.0 for extreme event-2 (Art. 10.5.5.3)
V = total factored vertical load (minimum factors)
φ_f = internal friction angle of soil

The factored (strength-1 and extreme event-2) lateral loads shall not exceed R_R .

Wing Wall Footings:

Foundation design shall be in accordance with Section 10 of the LRFD Code.

Commentary: The LRFD design requirements for shear in slabs with less than 2 feet of fill are not consistent. Article 5.14.5.2 states that the provisions of Article 5.13.3.6 shall apply which indicates shear design is required. However if the live load distribution of Article 4.6.2.3 is used, which is a less conservative distribution than Article 4.6.2.10 for spans over about 9 feet, slabs may be considered satisfactory for shear (Art. 5.14.4.1). In addition it has been ITD practice for many years, in accordance with the Standard Specifications, to consider slabs designed with less than 2 feet of fill to be adequate for shear. Because ITD has had a good history of performance with this design procedure it will remain ITD policy. For fills over 2 feet the provisions of 5.14.5.3 shall also apply to stifflegs as well as box culverts. AASHTO is silent for the most part on cast-in-place concrete three sided frames however structurally they behave and are designed more like a four sided box culvert than a simply supported slab bridge. The culvert test results for shear shown in the commentary indicate that frame structures with the walls integral with the slab behave differently than simple slabs.

Revisions:

June 2006 The crack control requirements where revised to reflect the 2005/2006 Interim changes in AASHTO Article 5.7.3.4.

The effective edge beam width was revised to reflect the 2005 Interim changes in AASHTO Article 4.6.2.1.4b.

	The live load distribution for fills less than 2' was revised to reflect the 2005 Interim changes in AASHTO Article 12.11.2.
July 2007	Skew angle changed from 20° to 25° for orientation of main reinforcement to conform to Article 9.7.1.3
	Maximum reinforcement criteria deleted to reflect the 2005 Interim changes in AASHTO Article 5.7.3.3.1.
	Corrected the reference in Edge beam Design to Article 4.6.2.1.4b.
	Corrected the reference in Wing Wall design for Sliding to Article 10.6.3.4.
Oct. 2007	Included the provision of Article 3.6.1.3.3 that the lane load is not required for spans not exceeding 15 ft.
April 2008	Revised nomenclature for terms in sliding of wing walls to agree with the 2008 Interims.
July 2009	Added statement that slab shear strength may be considered adequate for designs with less than 2' of fill. Changed ϕ for shear from 0.9 to 0.85. Added commentary.

CHAPTER 12
STANDARD DRAWING REVISION LOG

B12.1 Culvert Joint Details

June 2006 Added new standard drawing.

13.4 BRIDGE RAILINGS - GENERAL

One of the standard rail types as shown in the Bridge Design Manual Standard Drawings shall be shown on the plans, unless there is a clearly provable advantage in doing otherwise. Among those types, selection shall be made with due consideration of economy, aesthetics, Level of Service, and the preference of the District Engineer. The following table indicates the test level of ITD standard railings. Test Levels are defined in Article 13.7.2.

Bridge Railing	Test Level
Concrete Parapet	TL-4
Combination Rail	TL-3
Two Tube Curb Mount Rail	TL-4
W-Beam Railing	TL-3
Delaware Retrofit	TL-4
Iowa Retrofit	TL-4

NEW CONSTRUCTION

Concrete parapet is the preferred rail type for safety reasons. When an overlay is placed on the structure during original construction, the height of the parapet should be 2'-8" from the top of the overlay. Other rail types should be considered to meet Context Sensitive Design criteria.

Concrete parapets and median barriers shall be constructed perpendicular to the roadway cross slope for superelevation rates up to 6%. Bridges with superelevation rates greater than 6% shall be evaluated on a case-by-case basis.

For superelevation rates greater than 6%, revise the Notes on the Standard Drawing as follows:

- Concrete parapet shall be constructed so that the outside face is in a vertical plane. Height control shall be at the inside (traffic) face. End faces that fit up to precast concrete end sections shall be constructed perpendicular to the roadway grade.
- Concrete median barrier shall be constructed vertically.

Combination railings shall be used on a raised sidewalk when there is no barrier between the roadway and sidewalk.

Pedestrian/Bicycle Railing shall be used when a traffic barrier separates the roadway from the sidewalk.

W-Beam railing may be used if the site meets TL-3 criteria and the span length is less than 40’.

REHABILITATION

The following options should be considered on a deck rehab project:

DO NOTHING

When no major work is being done on the bridge and the existing rail is continuous and designed for a 10^k rail load, then the existing rail is acceptable. The railing does not require a successful crash test, but the connection to the roadway railing should be modified to meet current standards.

RETROFIT

When no major work is being done on the bridge and the existing rail was not designed for a 10^k rail load, the existing rail should be retrofit using the metal rail details shown on G-2-F in the Roadway Standard Plans, the Delaware Retrofit, or the Iowa rail retrofit. See Rail Standard drawings B13.7A, B13.7B, B13.7C, B13.7D, and B13.7E. No design calculations are needed to check the deck slab, but the exterior girder should be checked for the rail dead load and truck wheel load.

UP-GRADE

Up-grading the existing rail should be considered when the work will be cost effective; e.g., when the deck slab is replaced. One of the standard rail types shown in the Bridge Design Manual should be used. The deck slab and exterior girder shall meet the requirements of the current AASHTO code.

Revisions:

June 2006	The Combination Rail Test Level was changed to TL-3 to comply with Article 13.7.3.2.
	Corrected the w-beam railing reference from PL-1 to TL-3.
	Added the Delaware Retrofit to the Retrofit options.
	Added reference for Context Sensitive Design rail options.
	Added reference to new rail retrofit standard drawings B13.7A – B13.7E.

13.7.2 TEST LEVEL SELECTION CRITERIA

The Test Level required for a bridge site can be determined from the following table.

TEST LEVEL	HIGHWAY TYPE	DESIGN SPEED	TRAFFIC TYPE	CRASH TEST VEHICLE
1	Local Street	<30 mph	Low Volume	Small Automobile, Pickup Truck
2	Collector	<45 mph	Small Number of Heavy Vehicles	Small Automobile, Pickup Truck
3	Arterial	>50 mph	Very Low Mixture of Heavy Vehicles	Small Automobile, Pickup Truck
4	Freeway, Expressway, Interstate	>50 mph	Mixture of Trucks & Heavy Vehicles	Small Automobile, Pickup Truck, Single-Unit Van Truck
5	Freeway, Expressway, Interstate	>50 mph	Higher Ratio of Heavy Vehicles	Small Automobile, Pickup Truck, Van-Type Tractor-Trailer
6	Freeways	>50 mph	Higher Ratio of Tanker-Type Trucks	Small Automobile, Pickup Truck, Tractor-Tanker Trailer

Test Level 5 & 6 railings should be considered when a high accident rate is expected because of unfavorable site conditions and when severe consequences from rollover or penetration beyond the barrier are expected.

Unfavorable site conditions include but are not limited to the following:

- Reduced radius of curvature (Degree of curvature > 3°)
- Steep downgrades on curvature (Grade > 3%)
- Variable cross slopes
- Adverse weather conditions.

Test Levels 1-4 in the Table assume favorable site conditions. Increase one test level when site conditions are unfavorable.

CURRENT STANDARD BRIDGE RAIL WEIGHT

Rail Type	Weight – plf
Concrete Parapet	312
2-Tube Curb Mount Rail & Curb	195
Combination Pedestrian Rail & Parapet	266
Pedestrian Rail & Curb	102
Median Parapet	408
G-2-F Rail Retrofit Type 1	31
G-2-F Rail Retrofit Type 2	35
G-2-F Rail Retrofit Type 4	20
Delaware Rail Retrofit	25
Iowa Concrete Parapet Retrofit	262

OLD STANDARD BRIDGE RAIL WEIGHT

Rail Type	Weight – plf
2'-9¾" Concrete Parapet w/9"Ø Void	368
2'-9¾" Concrete Parapet w/o Void	435
5x9 Channel top/bottom Rail w/2x2x¼ angle pickets & 2'-4"x9" Curb	292
2-Tube Aluminum Rail & 6" Curb	112
3-Tube Aluminum Rail & 6" Curb	117

Revisions:

July 2009
- Deleted Combination Bicycle Rail
- Deleted Bicycle Rail
- Deleted G-2-F Type 3
- Changed weight of Pedestrian Rail from 96 plf to 102 plf
- Changed weight of G-2-F Type 4 from 13 plf to 20 plf
- Added Delaware Retrofit weight
- Added Iowa Retrofit weight
- Added Old Bridge Rail Table

A 13.2 TRAFFIC RAILING DESIGN FORCES

Rail Type	Test Level	M_c ft-kips	Within Wall Segment		End Wall Segment	
			L_c	R_w	L_c	R_w
Concrete Parapet	TL-4	12.25	5.83	90.23	4.61	55.63
Combination Rail	TL-2	12.86	7.66	84.40	4.59	65.16
City Coeur d’Alene	TL-2	12.04	8.26	85.25	4.90	59.87

A13.4.1 DECK OVERHANG DESIGN

STANDARD CONCRETE PARAPET

Bridge Section policy for the reinforcement of deck overhangs that support ITD standard concrete parapet shall be the reinforcement for the empirical deck design requirements for the top mat (#4 rebar at 12”) with the addition of #6 rebar spaced between the standard #4 bars. This shall be considered adequate for those areas at least 8 feet from any joint or discontinuity in the parapet. For areas less than 8 feet from joints or discontinuities in the parapet two #6 bars shall be evenly spaced between the #4 bars. The length of the additional #6 rebar shall be such that the bar extends at least halfway between the exterior girder and the first interior girder. This policy only applies to 8 inch thick decks with a minimum overhang of 24 inches from the centerline of the exterior girder to a maximum overhang of 72 inches.

STANDARD CONCRETE RAIL MOUNTED ON WALLS, CURBS OR EDGE BEAMS

For the design of retaining walls and curbs or edge beams supporting standard concrete parapet the following design procedure may be used (also applicable for decks supporting curbs or edge beams where parapets are constructed on the curb or edge beam, i.e. buried decks):

For walls and curbs only design Case 1 of Article A13.4.1, extreme event limit state, need be considered. The design loads are specified in Table A13.2-1. Design Case 3 need only be considered for the design of decks that support curb or edge beams with parapets. The loads for Design Case 3 shall be in accordance with Article 3.6.1, strength limit state.

Design Case 1 Loads for Traffic Railings.

$F_t = 54$ kips (from table A13.2-1 for TL-4)

For the general case, at least 8 feet from parapet joints or discontinuities, the design loads may be distributed over a length of wall, curb or deck as follows:

$E_m = 11 + 2.67x$
$E_t = 6 + 2.67x$ (for horizontal members only)

For the ends of parapet sections within 8 feet of joints or discontinuities the design loads may be distributed over a length of wall, curb or deck as follows:

$E_m = 7.5 + 0.5x$
$E_t = 10 + 4x$ (for horizontal members only)

where: E_m = the length of wall, curb or deck the design moment is distributed over in feet.
E_t = the length of deck the design tension force is distributed over in feet.
x = the distance from the base of rail to the design location in feet, either vertical or horizontal or both.

In any case the load distribution length shall not exceed the actual member length.

In lieu of the above method the provisions of Article A13.4.2 may be used where applicable.

Commentary

The above distribution equations where derived from the results of an elastic finite element analysis using the LARSA structural analysis program. These equations are based on the resulting tension and bending moment in the most severely stressed 6” width of the supporting member. If these equations are directly applied to the case of a parapet supported on a standard 8” thick deck overhang the calculated area of reinforcement at the face of rail would be 0.76 in^2/ft, while a #4 and #6 bar at 12 inch spacings equals 0.64 in^2/ft which is 84% of the calculated value. This is considered acceptable, however, because the LARSA model conservatively assumes an elastic distribution while the extreme event limit state relies on the ultimate strength of the supporting deck, which cannot be realized without significant yielding of the reinforcement (the strain at ultimate in this case is 580% of the yield strain), thereby resulting in a greater width of load distribution.

CHAPTER 13
STANDARD DRAWING REVISION LOG

B13.1A Concrete Parapet with Thrie Beam Rail
July 2009 Added ½" V Groove detail 3" from edge of deck

B13.1B Concrete Parapet with Precast End Connection
April 2008 Changed Section C-C in the Partial Elevation to D-D to avoid duplication in View A-A.
July 2009 Added ½" V Groove detail 3" from edge of deck.

B13.1C Concrete Parapet with Thrie Beam Rail
July 2009 Added ½" V Groove detail 3" from edge of deck

B13.1D Concrete Parapet with Precast End Connection
July 2009 Added ½" V Groove detail 3" from edge of deck.
Added Note 16
Corrected height of parapet in Partial parapet Elevation

B13.1E Concrete Parapet Light Pole Support
April 2008 Added new sheet

B13.2A Two-Tube Curb Mounted Rail
June 2006 Revised Note 2 to make anchor bolts conform to ASTM F-1554 Grade 105.
Revised Note 14 reference to conform to the 2004 Standard Specifications.
Added 4" dimension between slotted holes in post and specified horizontally slotted holes in post.
July 2009 Added ½" V Groove detail 3" from edge of deck.

B13.2B Two-Tube Curb Mounted Rail
July 2009 Added ½" V Groove detail 3" from edge of deck.

B13.3A Combination Pedestrian/Bicycle & Traffic Railing
June 2006 Revised Note 1 to make anchor bolts conform to ASTM F-1554 Grade 36.
Revised Note 10 reference to conform to the 2004 Standard Specifications.
Added note to Tube Splice Detail for 2'x2" tubes.
Combined the Bicycle and Pedestrian rail drawings using a 3'-6" height to conform to the approved 2006 Ballot Item.
Modified the title block to include "Bicycle".
July 2009 Reformatted the Notes
Added additional requirements for powder coating
Increased left end length on splice tube and removed "Typical at Exp. Joints & Dummy Joints".
Modified note 12 to read, "Each rail section…"

B13.3B Combination Bicycle & Traffic Railing
June 2006 Deleted the 4'-6" height rail standard drawing.

B13.3C Concrete Parapet for Railing – without approach slab
June 2006 Changed the height of the end block to 3'-7½" and revised details for the new 3-6" rail height.
April 2008 Revised bid item description in Note 4 to agree with the Standard Specifications.
July 2009 Added ½" V Groove detail 3" from edge of deck.

B13.3D Concrete Parapet for Railing – with approach slab
June 2006 Changed the height of the end block to 3'-7½" and revised details for the new 3-6" rail height.
April 2008 Revised bid item description in Note 4 to agree with the Standard Specifications.
July 2009 Added ½" V Groove detail 3" from edge of deck.

B13.3E Combination Rail Light Pole Support
April 2008 Added new sheet

CHAPTER 13
STANDARD DRAWING REVISION LOG

B13.3F Protective Pedestrian Fence for Combination Rail Concrete Parapet
July 2009 Added new sheet

B13.4A 4'-6" Standard Bicycle Rail
June 2006 Deleted the 4'-6" height rail standard drawing.

B13.4B Standard Pedestrian/Bicycle Railing
June 2006 Combined the Bicycle and Pedestrian rail drawings using a 3'-6" height to conform to the approved 2006 Ballot Item.
Deleted epoxy reinforcement from the Drilled Concrete Post Base.
Widened the curb to 10" to provide cover for reinforcement around the anchor bolts.
Modified the title block to include "Bicycle".
July 2009 Added ½" V Groove detail 3" from edge of deck.
Changed tube wall thickness to 3/16".

B13.4C Pedestrian/Bicycle Railing Details
June 2006 Revised Note 6 to require epoxy reinforcement only in the curb.
Added Note 7 specifying Concrete Class 40A.
Added Notes 16, 17, & 19 for tube ends, vent holes, and alternate splice details.
Increased the top width of bar R2.
Modified the title block to include "Bicycle".
July 2009 Renumbered the Notes
Added additional requirements for powder coating
Changed tube wall thickness to 3/16".
Modified splice tube detail.
Modified note 21 to read, "Each rail section…"

B13.6A W-Beam Railing Details
June 2006 Added a note in the title block for using roadway standard drawing G-1-L on retrofit projects.

B13.7A G-2-F Type 1 Rail Retrofit Details
June 2006 Added a new drawing.
July 2009 Added details from Roadway Standard Drawing G-2-F to allow G-2-F to be deleted.
Revised paint system in Note 3.

B13.7B G-2-F Type 2 Rail Retrofit Details
June 2006 Added a new drawing.
July 2009 Added details from Roadway Standard Drawing G-2-F to allow G-2-F to be deleted.
Revised paint system in Note 3.

B13.7C G-2-F Type 4 Rail Retrofit Details
June 2006 Added a new drawing.
April 2008 Changed size of attachment angle to 6x4x5/16 and added Detail A.
July 2009 Added details from Roadway Standard Drawing G-2-F to allow G-2-F to be deleted.
Revised paint system in Note 1.

B13.7D Delawatre Rail Retrofit Details
June 2006 Added a new drawing.
July 2009 Added details from Roadway Standard Drawing G-2-F to allow G-2-F to be deleted.
Added Note 11 for painting field drilled holes.

B13.7E Iowa Rail Retrofit Details
June 2006 Added a new drawing.

14.5 BRIDGE JOINTS

Expansion joint devices are designed to accommodate structure movement, provide smooth and quiet passage of traffic, prevent water runoff (particularly deicing chemicals) from damaging the supporting structural elements, and provide a long service life.

The following selection criteria cover the expansion joint systems most commonly used in Idaho:

SMALL MOVEMENT JOINTS

Small movement joints can be utilized where the total movement is approximately 1"-2" and the skew is less than 30°. Joint types include compression seals, silicone sealants, and asphalt plug joints. Compression Seals are the preferred joint type for new construction. Silicone Sealant joints should be limited to repair/rehabilitation projects.

- Compression Seals are continuous preformed elastomeric sections with extruded internal web systems and are held in place by mobilizing friction against adjacent vertical faces. They must be sized and installed to always be in a state of compression. The nominal uncompressed width of Compression seals should not exceed 4”and the skew should not exceed 30°.

  Seals should be designed according to the Selection Criteria for Compression Seals in A14.6.

  The following is a list of approved joints:

MANUFACTURER	STYLE
WATSON BOWMAN & ACME	WA Series
D.S. BROWN	CV Series

- Silicone sealants are poured in place directly over a foam backer rod placed in the expansion gap. The cured silicone sealant joint can accommodate tensile movements of up to 100% and compressive movements of up to 50% of the sealant width at installation. A minimum recess is required from the top of the pavement to the top of the silicone sealant in order to prevent tire traffic from contacting and debonding the sealant from the substrate.

  The following is a list of approved joints:

MANUFACTURER	STYLE
DOW CORNING	Dow 902 RCS

- Asphaltic plug joints consist of polymer modified asphalt (PMA) installed within a blockout over a steel plate. The steel plate spans across the expansion gap to retain the PMA during its installation. Application guidelines must be carefully followed to assure successful performance. They should not be used for joints having large skew angles, joints subjected to large rotations, joints subjected to differential vertical movements (for example, longitudinal separation joints), or in situations where the total height of the PMA above the steel plate is less than 2”. Asphaltic plug joints should not be used in situations where the adjacent pavement is subjected to significant acceleration or deceleration (off ramps, traffic signals) because the PMA has a tendency to creep out of the blockout and this tendency is amplified by any horizontal loading.

  The following is a list of approved joints:

MANUFACTURER	STYLE
LINEAR DYNAMICS	BRIDGE JOINT SYSTEM (BJS)
D.S. BROWN	MATRIX 502 ASPHALTIC EXPANSION JOINT
WATSON BOWMAN ACME	WABO EXPANDEX JOINT SYSTEM
DEERY AMERICAN CORP	DEERY FLEXIBLE JOINT SYSTEM

Polymer concrete headers are recommended at compression seal and silicone sealant joints. Polymer concrete provides tensile strength and toughness to resist traffic impact. Proprietary elastomeric concretes can also be used to further enhance impact resistance. Patent infringement issues may result when generic polymer concrete is used in combination with a Dow Corning silicone sealant.

MEDIUM MOVEMENT JOINTS

Strip Seals are the preferred system for medium movement joints. Maximum allowable joint opening, measured in the direction of traffic, is 4". This is to improve ride and reduce hazard to motorcycles and bicycles.

Strip Seals should be designed according to the Selection Criteria For Strip Seals in A14.6.

The following factors that affect movement should be considered in determining the joint size:

- Creep
- Construction tolerances
- Temperature range
- Bearing type and direction of allowed movements
- Skew
- External restraints

Movement from earthquake is not generally considered.

When designing joint modifications, review past inspection reports for recorded joint movement history. Adjustment of the expansion device to compensate for bridge temperature at time of installation must be possible.

The majority of joint failures have been due to failure of the anchorage system.

Complete full-width units should be shipped to the job site. Lengths up to 60' can normally be shipped without difficulty. No joints in the continuous rubber seal should be allowed. One pre-approved manufacturer's shop vulcanized splice per seal is permitted.

Closely analyze joints at sidewalks and parapets with respect to leakage, constructibility, and maintenance.

At highly skewed locations, an effort should be made to reduce skew to diminish joint complexity.

Structural steel extruded shapes with a minimum thickness of 1" should be used. Aluminum extruded shapes should not be used.

The following is a list of approved joints:

MANUFACTURER	STYLE	EXTRUSION TYPE
WATSON BOWMAN & ACME	SE Series	E or A
D.S. BROWN	A2R Series	SSA2

LARGE MOVEMENT JOINTS

Modular Expansion devices are the preferred system for movements in excess of 4". The skew angle should be less than 30°and the movement per sealing element should not exceed 3". The same criteria for Medium Movement Joints apply to modular joints, in addition to the following.

All large expansion devices should be designed for the movements required plus a safety factor of 15%. This excess allowance is intended to prevent destruction of the joint due to unpredictable movements at a given location. Consideration should be given to accommodating some earthquake movement.

All elastomeric sealing components shall be continuous full width of each roadway, including curb treatment. The entire joint device should be shipped completely assembled to the jobsite.

For large expansion joints specify all accepted name brands that provide good performance. Do not specify "OR APPROVED EQUAL".

The following is a list of approved joints:

MANUFACTURER	STYLE
WATSON BOWMAN & ACME	STM Series
D.S. BROWN	D Series

STANDARD DRAWINGS
The most current data for the RECOMMENDED MANUFACTURERS TABLE should be obtained from the manufacturer's web site.

MANUFACTURER	WEB SITE
WATSON BOWMAN & ACME	wbacorp.com
D.S. BROWN	dsbrown.com

The following Compression Seal Standard Drawings are located in the Bridge Design Manual:

SKEWS ≤ 10°
SKEWS > 10°

The Strip Seal Standard Drawing is for a skewed bridge and the details should be modified for a square crossing. The adjustment for temperature change in Note 12 must be calculated.

The Modular Joint Standard Drawing is for a square bridge and the details should be modified for a skewed crossing.

Generic details are provided for both the Silicone Sealant and Asphaltic Plug joints.

MAINTENANCE
Consideration should be given to the maintainability of the joints (particularly for movements over 4"), availability and replaceability of parts, and provisions for access to reach these parts.

WIDENING AND REHABILITATION
During the rehabilitation of bridge decks, it is recommended that existing joints and structure layout be studied to determine which joints can be eliminated or what modifications are necessary to revamp those joints that remain to provide an adequate functional system. Some latitude will be necessary on joint type selection for rehabilitation projects.

SNOW PLOWS
Snow plows have a fixed blade angle between 28°- 35°(left-forward). To minimize the possibility of having a snowplow blade drop into a joint, joint skews between 25°-38° (left-forward) should be avoided. If not avoidable, modify the joint so the blade can pass over the joint without snagging.

SUBSTRUCTURE PROTECTION
Concrete surfaces beneath expansion joints should be protected with a penetrating water repellant sealer that conforms to Section 511, Type C system. Faces of backwalls, beam seat surfaces, and exposed faces of the cap should be protected on abutments. Pier surfaces to be protected include the beam seat and pier cap. Areas to be protected should be shown on the contract plans.

SHOP DRAWINGS
Review the shop drawings in combination with the Plans and Special Provisions for the following information

- PLAN and ELEVATION of the expansion joint.
- Complete details of all components and sections showing all materials incorporated into the joint.
- All AASHTO or other material designations.
- Movement rating and load capacity.
- Installation procedures including services of a manufacturer's field representative if required.
- Treatment of sidewalks and parapets with respect to leakage and maintenance.
- Anchorage details including special reinforcement of blockouts.
- Considerations of weld details in areas of stress concentration.
- Opening dimensions at intermediate temperatures.

RECOMMENDATIONS

- Eliminate expansion joints where feasible.
  Use monolithic abutments on smaller bridges.
  Use continuity to reduce the number of joints on larger bridges.

- The following joint types should be used for the range of movement shown:

JOINT TYPE	TOTAL MOVEMENT
Compression Seal	1" - 2"
Silicone Sealant	1" - 2"
Asphaltic Plug	1" - 2"
Strip Seal	2" - 4"
Modular Joints	≥ 4"

- As a guideline, bridges with the following lengths may be designed continuous without expansion joints at end abutments if the skew is less than 30º.

STRUCTURE TYPE	LENGTH
Steel Structures	300'
Post-tensioned Concrete	400'
Cast-In-Place Concrete	400'
Prestressed Girder	400'

- Compression seals should not be used when the skew exceeds 30°. Joints with skews greater than 30° require special consideration - see "Selection Criteria for Strip Seals".

- To prevent snowplow blades from dropping into a joint, left-forward skews of 25°-38° should be avoided.

Commentary

As of October 2006, only two manufacturers of compression and strip seal joints could be found – D.S.Brown & WatsonBowmanAcme. Listing only two products on the plans in acceptable to FHWA Idaho Division Office.

Revisions:

April 2008 Added Commentary.

TFE/STAINLESS STEEL EXPANSION BEARING UNITS

GENERAL

The design of TFE/stainless steel bearings shall be in accordance the AASHTO *LRFD Bridge design Specifications*, Section 14. This type of bearing shall be used when elastomeric bearing pads become too large.

The TFE/stainless steel surface shall be designed to accommodate the horizontal movement of the bridge. The elastomeric pad shall be designed for the compressive stresses and rotation.

DESIGN GUIDELINES FOR ELASTOMERIC BEARINGS

Elastomeric bearings shall consist of either plain elastomeric pads or steel-reinforced laminated pads.

The bearings may be designed by either **Method A** (Article 14.7.6) or **Method B** (Article 14.7.5) of the AASHTO LRFD Bridge Design Specifications.

Method A is the required design procedure for plain elastomeric pads.

Method B is the preferred method of design for steel-reinforced laminated pads.

Design **Method B** and/or the use of Grade 4 elastomer requires that a long term load test be preformed on at least 10% of the completed bearings per ITD Standard Specifications, Subsection 711.02 (5e). When **Method B** or Grade 4 elastomer is used it should be clearly noted in the plans to ensure that the correct testing is performed.

The elastomeric material shall be specified by its hardness and grade when designing by **Method A**. Most bearing designs for **Method A** should be based on 60 durometer elastomer, Grade 3. The shear modulus for 60 durometer elastomer shall be assumed to range from 130 psi to 200 psi, with the least favorable value being used for the particular design parameter being considered. When designing by **Method B** the elastomeric material shall be specified by shear modulus and grade. The shear modulus should be specified as 130 psi with the design value increased or decreased by 15% whichever is the least favorable value for the particular parameter being considered. Grade 4 elastomer should be specified when the bridge is located in low temperature Zone D, if Grade 3 is used in Zone D the other components of the bridge must be designed to resist 1.5 times the shear force determined by Article 14.6.3.1 when a low friction sliding surface is used or 4 times this shear force for a non-sliding surface (see Article 14.7.5.2). Zone D for Idaho shall be defined as any area above an elevation of 3800 feet or any location in Bonner or Boundary County.

Live Load reactions for bearing design shall be based on the girder distribution factor for shear, Article 4.6.2.2.3.

In calculating the live load rotations the whole structure cross-section, acting as a unit, may be used.

All loads and rotations used in bearing design shall be determined using the Service I limit state including dynamic load allowance (IM).

Bearing Design Criteria for Prestressed/Precast Concrete Girders

For the purpose of simplification all girder reactions may be based on simple span analysis, even if the girders are made continuous for composite loads. However, the live load rotations may still be determined assuming continuous action.

The instantaneous compressive deflection (LL and IM only) shall be less than 1/8”. The long term compressive deflection due to dead load including creep shall be limited to 3/16” (the value of 3/16” for the allowable long term compressive deflection is considered acceptable for smoothness of ride without restricting the design of the bearings unnecessarily).

For expansion bearings without a sliding surface shear deformation shall be determined from the combined effects of thermal, creep and shrinkage movements of the structure. The thermal movement shall be based on 65% of the total design thermal range (LRFD Article 3.12.3.1). A creep and shrinkage factor of 0.002 may be used in lieu of a more precise calculation. For bearings with a stainless steel TFE sliding surface the shear deformation shall be determined from the shear force calculated by multiplying Dead Load reaction with the coefficient of friction for the appropriate bearing surface type in accordance with Table 14.7.2.5-1 of the LRFD specifications.

Combined compression, rotation and shear shall be checked by assuming Service I reactions are applied to the bearing in combination with Live Load rotation and initial lack of parallelism as defined below. Dead load rotation need not be included since the internal prestress force produces an end rotation that is at least as great as the dead load rotation and in the opposite direction, thereby canceling the effect of the dead load rotation.

Live Load Rotation – shall be taken as the rotation due to all design lanes on the structure being loaded in a manner that produces the maximum rotation and assuming the load is distributed to all girders equally. Dynamic load allowance shall be included. When combining compression and rotation the Live Load reaction may be either the maximum load or the load associated with the rotation.

Initial Lack of Parallelism – a rotation of 0.005 radians shall be assumed due to construction tolerances; this initial lack of parallelism includes the net difference between the dead load girder end rotation and the prestress end rotation. However this is not sufficient in many cases to include the effects of the roadway profile grade, consequently the beam seats of a precast concrete girder should be constructed on the same slope as the girder. (The slope of the girder is defined here as the difference in bearing seat elevations of the two girder ends divided by the length of the girder; this is not necessarily the same as the roadway grade.)

Bearing Design Criteria for Steel Girders

Compressive stress shall be determined from the combined Service I reactions as determined from the structural model used for the girder design.

The instantaneous compressive deflection (live load and impact only) shall be less than 1/8”. The long term compressive deflection due to dead load including creep shall be limited to 3/16”.

Shear deformation shall be determined based on 65% of the total design thermal range (LRFD Article 3.12.3.1). For bearings with a stainless steel TFE sliding surface the shear deformation shall be determined from the shear force calculated by multiplying Dead Load reaction with the coefficient of friction for the appropriate bearing surface type in accordance with Table 14.7.2.5-1 of the LRFD specifications.

Combined compression and rotation requirements shall be checked by assuming the Dead Load and Live Load reactions, as determined above, are applied to the bearing in combination with the Dead Load rotation (for non-cambered girders only), Live Load rotation and initial lack of parallelism as defined below:

Dead Load Rotations – need only be included for girders constructed of rolled sections that have not been cambered. Cambered girders should have no net dead load rotation.

Live Load Rotation – shall be taken as the rotation due to all design lanes on the structure being loaded in a manner that produces the maximum rotation and assuming the load is distributed to all girders equally. Dynamic load allowance shall be included.

Initial Lack of Parallelism – a rotation of 0.005 radians shall be assumed due to construction tolerances. However this is not sufficient in many cases to include the effects of the roadway profile grade. A beveled sole plate shall be used to compensate for grade. Consequently the bearing surfaces on the abutments and piers should be constructed level for steel structures.

Bearing Details

Steel reinforced elastomeric bearings shall consist of alternating layers of a minimum of 14 gage A36 steel and neoprene bonded together. All internal layers of neoprene shall be of equal thickness, there is no limit on the maximum thickness of internal layers as there was in previous specifications provided all design criteria are met. However, the minimum thickness of internal layers should be 3/16” so that a minimum external layer does not violate the requirement to be no more than 70% of the thickness of an internal layer (Article 14.7.5.1). Exterior layers shall have a minimum thickness of 1/8” to provide enough cover to protect the reinforcement.

Sole plates for steel girders shall be welded to the bottom flange to provide a level surface for the elastomeric pads to bear against. Sole plates shall be beveled to compensate for the longitudinal grade of the bridge. Sole plates shall be detailed with enough length (in the direction of the span) to accommodate construction tolerances.

Information for Required Testing

All steel reinforced elastomeric bearings are to be tested for the Short-Duration Compression Test (subsection 711.02 {5d}). In order to perform this test the maximum design load for each bearing size and type is required. Therefore the method (A or B) that was used in design, along with the maximum design load for each size and type of bearing, must be called out on the bearing detail sheet so that all the information for fabrication and testing is available in one place.

Commentary

In most cases Method B will result in significantly smaller and thinner bearing pads and although it requires more rigorous testing it will result in a more practical sized bearing.

Revisions

July 2009	Revision to specify material by shear modulus for Method B and the definition of thermal movement to comply with 2008 interims.

The following equations closely approximate the AASHTO curves shown in Figure C14.7.5.3.3-1 in the AASHTO LRFD Bridge Design Specifications.

To evaluate the equations, stress should be in psi. The value for strain is in percent.

70 DUROMETER COMPRESSIVE STRAIN EQUATIONS

Shape Factor(SF)≤ 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.05 * \left(\frac{\sigma}{600}\right)^{0.15}$ $\quad x = 0.65\left(\frac{SF}{6}\right)^{1.0-0.0004*\sigma}$

Shape Factor(SF)> 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.5 * \left(\frac{\sigma}{1000}\right)^{0.5}$ $\quad x = 0.25\left(\frac{SF}{12}\right)^{0.4}$

60 DUROMETER COMPRESSIVE STRAIN EQUATIONS

Shape Factor(SF)≤ 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.065 * \left(\frac{\sigma}{600}\right)^{0.15}$ $\quad x = 0.60\left(\frac{SF}{6}\right)^{0.725}$

Shape Factor(SF)> 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.65 * \left(\frac{\sigma}{1000}\right)^{0.5}$ $\quad x = 0.25\left(\frac{SF}{12}\right)^{0.15}$

50 DUROMETER COMPRESSIVE STRAIN EQUATIONS

Shape Factor(SF)≤ 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.10 * \left(\frac{\sigma}{600}\right)^{0.15}$ $\quad x = 0.60\left(\frac{SF}{6}\right)^{0.725}$

Shape Factor(SF)> 6.0

$\varepsilon = C\sigma^x$ $\quad C = 0.6 * \left(\frac{\sigma}{1000}\right)^{0.5}$ $\quad x = 0.275\left(\frac{SF}{12}\right)^{0.15}$

Revisions:

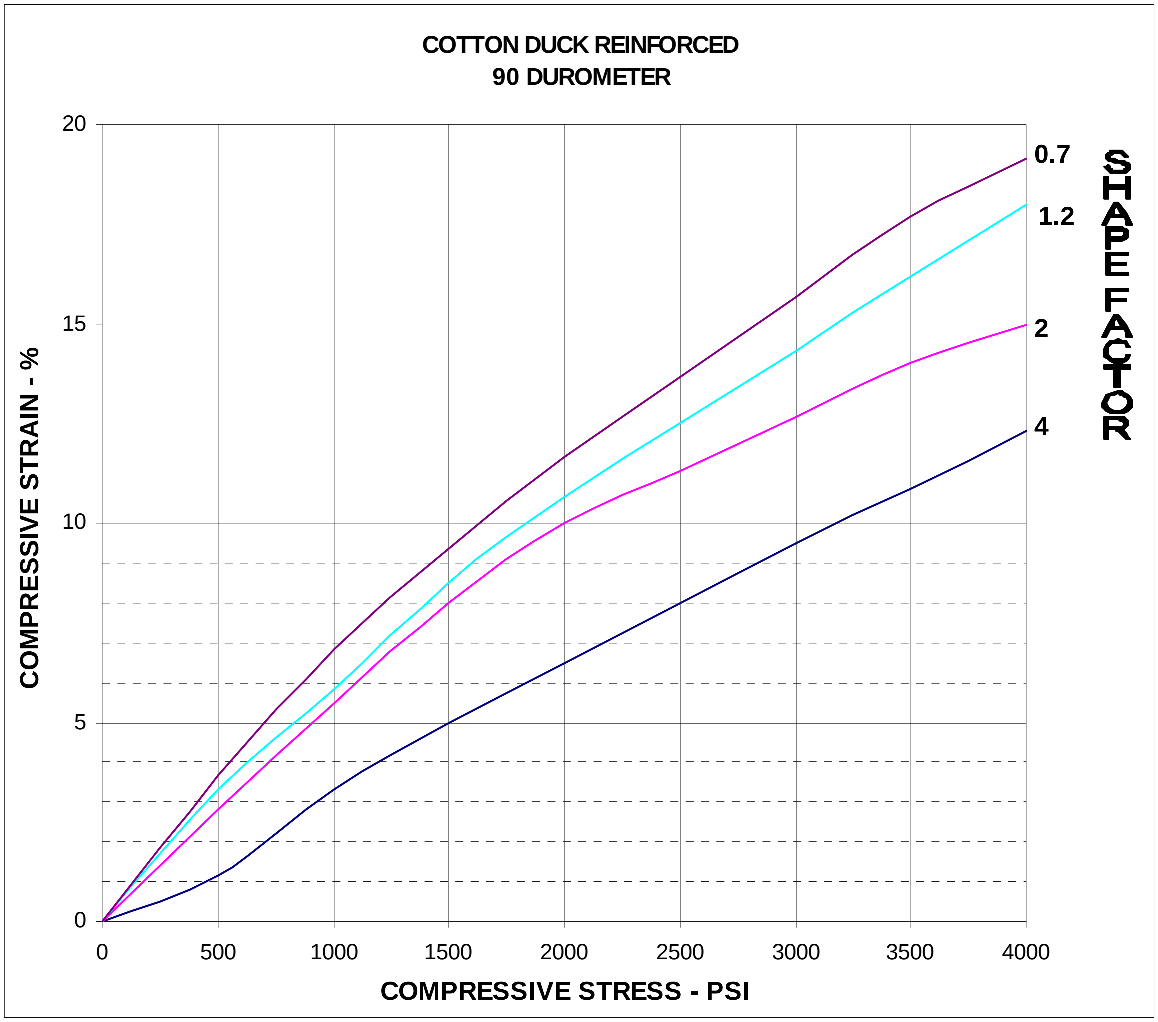
June 2006 Added explanatory notes defining the intent of the equations and the units for stress & strain.

COTTON DUCK REINFORCED PAD

The following data may be used for cotton duck reinforced pads in lieu of more precise information.

Hardness (Shore A)--------------------------------90

Compressive Stress – psi	Shear Modulus - psi
500	400
1000	850
2000	1150
3000	1325



BEARING TABULATION TABLE

Bridge name or reference.					
Bearing identification mark					
Number of bearings required					
Seating Material		Upper Surface			
		Lower Surface			
Allowable average contact pressure (psi)		Upper Face	Serviceability		
			Strength		
		Lower Face	Serviceability		
			Strength		
Design Load effects	Service Limit State		Vertical	max	
				perm	
				min	
			Transverse		
			Longitudinal		
	Strength Limit State		Vertical		
			Transverse		
			Longitudinal		
Translation	Service Limit State	Irreversible	Transverse		
			Longitudinal		
		Reversible	Transverse		
			Longitudinal		
	Strength Limit State	Irreversible	Transverse		
			Longitudinal		
		Reversible	Transverse		
			Longitudinal		
Rotation (RAD)	Service Limit State	Irreversible	Transverse		
			Longitudinal		
		Reversible	Transverse		
			Longitudinal		
Maximum bearing dimensions (in)	Upper Surface		Transverse		
			Longitudinal		
	Lower Surface		Transverse		
			Longitudinal		
	Overall height				
Tolerable movement of bearing under transient loads (in)			Vertical		
			Transverse		
			Longitudinal		
Allowable resistance to translation under service limit state (kips)			Transverse		
			Longitudinal		
Allowable resistance to rotation under service limit state (k/ft)			Transverse		
			Longitudinal		
Type of attachment to structure			Transverse		

TFE/STAINLESS STEEL EXPANSION BEARING UNITS

ELASTOMERIC PAD

- The elastomeric pad shall be designed as a Pinned Bearing. Refer to the design guidelines of Article 14.7.5 & 14.7.6.
- The width of the elastomeric pad = width of bottom flange of concrete or steel girder – 1".

PTFE

- The PTFE shall be designed in accordance with AASHTO Article 14.7.2.
- The PTFE shall be virgin, unfilled sheets and recessed for one-half its thickness.
- The sheets may be either rectangular or circular. Circular sheets may provide easier fabrication for milling the recess.

STAINLESS STEEL SHEET

- The thickness of the stainless steel mating surface shall meet the requirements of AASHTO Article 14.7.2.3.2.
- Seal weld the stainless steel sheet to a ⅜" steel plate.
- The ⅜" steel plate shall be the same dimension as the sole plate minus ¼" on all sides..
- The ⅜" steel plate/stainless steel sheet unit shall be attached to the sole plate with stainless steel cap screws.
- Length = Width of bottom flange of the girder + 2½" (length of sole plate - ½").
- Width = Elastomeric pad length + total temperature movement + Δ_r + 2".
- Δ_r is increase in the length of the bottom flange due to the girder end rotation under dead load.

Simple Span, One End Pinned: $\Delta_r = 6.4*DL*Y_b/L$

DL = maximum dead load deflection at midspan (inches)
L = span length (inches)
Y_b = distance from the neutral axis of the noncomposite section to the bottom of the girder (inches)

Continuous Span: $\Delta_r = 1.6*DL*Y_b/L$

DL = maximum dead load deflection in the end span (inches)
L = distance from the centerline bearing to point of maximum dead load deflection (inches)
Y_b = distance from the neutral axis of the noncomposite section to the bottom of the girder (inches)

SOLE PLATE

- The minimum thickness shall be ¾".
- The top surface that bears against the girder flange shall be tapered for the longitudinal slope of the girder.
- Length = width of bottom girder flange + 3".
- Width = Elastomeric pad length + total temperature movement + Δ_r + 2½".

STAINLESS STEEL CAP SCREWS

- The cap screws shall be installed parallel to the edge of the bottom girder flange and equally distributed on each side.
- The minimum spacing for the cap screws is 3" and the maximum spacing shall not exceed 5".
- Cap screws shall be ¼"-20 UNC x ¾" 82º hexagon socket flat countersunk head cap screw 18-8SS. The cap screw size is limited to ¼" diameter due to the depth of the countersunk head and the thickness of a 10 gage stainless steel sheet.
- The minimum number of cap screws required is calculated as follows:

$$\text{Number} = \frac{\text{FS * coef friction * Dead Load}}{\text{Screw thread area * allow shear stress}}$$

Dead Load = dead load reaction (kips)
FS = factor of safety = 2.0
Coefficient of Friction = value for TFE/stainless steel without silicone grease.
Screw Thread Area = root area for ¼" cap screw = 0.027 in^2.
Allowable Shear Stress = 0.75 * 80 ksi = 60 ksi

ANCHOR BOLTS

- Anchor bolts shall be ASTM F-1554 Grade 36 with a minimum 1” diameter. 1½” diameter anchor bolts are recommended.
- Anchor bolts shall be designed to resist a horizontal seismic force in accordance with AASHTO Article 3.10 for the seismic category at the bridge site.

MASONRY PLATE

- The masonry plate shall be ¾” minimum thickness.
- Length = Length of the restraint angle.
- Width = Width of bottom girder flange plus twice the length of the bottom leg of the restraint angle + ¼”.
- Bolt holes shall be the bolt diameter + 1/8”.
- The masonry plate is optional for smaller girder reactions; this will need to be determined on a case-by-case basis. When the masonry plate is used, it shall be securely attached to the concrete surface, normally by bolting.

SOLE PLATE CONNECTION

- A 1/4" fillet weld using an E-70 electrode is used to connect the sole plate to the girder.

- The length of weld required = $\dfrac{\text{Max. Transverse Force}}{3.35\text{ k/inch}}$

  E-70 ¼” weld strength = ¼”* 0.707 * 70 ksi * 0.27 = 3.34 k/inch

  The maximum transverse force shall include seismic forces.

- If the length of weld required exceeds the available weld length, the weld size must be increased.

KEEPER ANGLE (NOT FOR SEISMIC RESTRAINT)

- The length of the cope can be reduced if the sole plate is designed with a shear tab. This will reduce the required length of the keeper angle.

  HORIZONTAL LEG

  d_H = Length of horizontal leg of the restraint angle (inches)
  t = Thickness of the restraint angle (inches)
  P_L = Max. Longitudinal force (kips)
  F_T = Allowable tensile stress (ksi)
  FS = Factor of Safety
  C_W = Width of cope in the horizontal leg = 1 1/2".
  ∅ = Diameter of the anchor bolts (inches)

  $$d_{H\ min} \geq \frac{P_L}{FS\ F_T\ t} + C_W + \varnothing$$

  Minimum edge distance for anchor bolts must be checked.

  VERTICAL LEG (Recommended angle size of 6"x 6" x 3/4" to start design)

  d_V = Length of vertical leg of the restraint angle (inches)
  t = Thickness of the restraint angle (inches)
  P_T = Max. Transverse force (kips)
  FS = Factor of Safety
  C_H = Height of cope in the vertical leg = Sole P_L + Steel P_L + Stainless steel + PTFE + Pad + 1/4".
  C_L = Length of the cope = Soleplate + T + 1".
  T = Total temperature movement (inches)
  F_B = Allowable bending stress (ksi)

  $$d_V \min \geq C_H + \sqrt{\frac{P_T * C_L}{F_B * FS * t}}$$

EXPANSION JOINTS

NOTATIONS

θ = Skew angle.

α = Coefficient of thermal expansion
0.0000060/°F for concrete
0.0000065/°F for steel

β = Shrinkage Coefficient for reinforced concrete, 0.0002.

μ = Factor accounting for the restraining effect imposed by superstructure elements installed before the concrete slab is cast.
1.0 for flat slabs
0.8 for cast-in-place box girders and t-beams
0.5 for prestressed girders
0.0 for steel girder bridges

T_c= Structure temperature during construction of joint opening.

L = Length of structure contributing to expansion or contraction of the joint (feet).

W = Nominal uncompressed width of expansion seal (inches)

A = Joint opening normal to joint at the time of deck placement (inches).

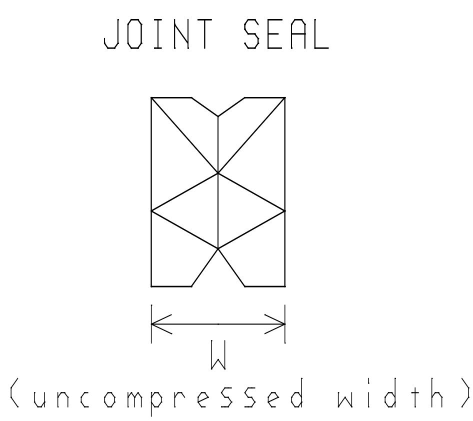
K = Temperature drop below the installation temperature divided by temperature range.
Assume installation temperature equals 60°F

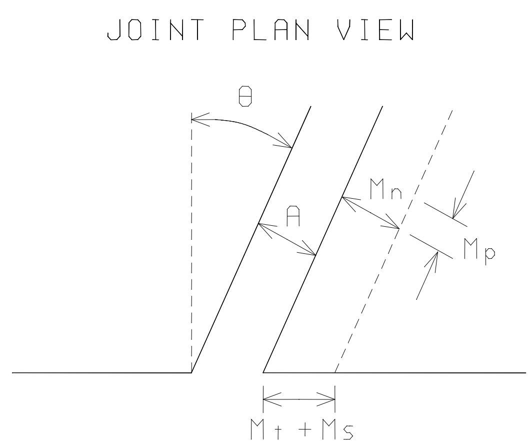
M_t= Movement due to temperature (inches).

M_s= Movement due to shrinkage after construction (inches)

M_p= Movement parallel to joint (inches).

M_n= Movement normal to joint (inches).





SELECTION CRITERIA FOR COMPRESSION SEALS

I. Design Limitations

A. Total anticipated movement of the expansion joint, $M_t + M_s$, should not exceed 2". When the nominal seal width computed by the following procedure exceeds 2", a joint system with greater movement capacity is required.

B. The maximum joint opening shall not be greater than 0.85W. The minimum joint opening shall not be less than 0.40W. The minimum joint opening at installation of the seal shall not be less than 0.60W.

C. The skew angle should not exceed 30°.

D. Temperature Range

Concrete structures.......… 0° to 80°F
Steel structures.......…….. -30° to 120°F

II. Design Procedure

A. Movement Calculations

1. $M_t = 12(L)(\alpha)(\text{temp. range})$
2. $M_s = 12(L)(\beta)(\mu)$
3. $M_p = (M_t + M_s) \sin\theta \leq 0.22W$
4. $M_n = (M_t + M_s) \cos\theta \leq 0.45W$

B. Selection of Seal Width

1. The maximum joint opening is equal to the minimum installation opening plus the movement due to temperature drop and shrinkage, therefore:
   $0.85W = 0.60W + (\cos\theta)(KM_t + M_s)$, or
   $W = 4(\cos\theta)(KM_t + M_s)$
2. The seal width to accommodate M_p:
   $W = M_p \div 0.22$
3. The seal width to accommodate M_n:
   $W = M_n \div 0.45$
4. The minimum seal width, W, shall be the largest of the values calculated in steps 1 thru 3 above.

C. Width of expansion joint opening at 60°F:

$A = (0.60)(W)$

D. Adjustment in joint opening for a 10°F change in temperature.

III. Design Example

Structure type, prestressed girder.
Total length, 300'.
Skew angle, 25°.
Expansion joints at both abutments.
Point of no movement for temperature and shrinkage is at the center of the bridge.

Value of Constants:

$\theta = 25°$
$\alpha = 0.000006/°F$
$\beta = 0.0002$
$\mu = 0.5$
$L = 300' \div 2 = 150'$
$K = (60-0) \div 80 = 0.75$

A. Movement Calculations

1. $M_t = (12)(150)(0.000006)(80) = 0.864"$
2. $M_s = (12)(150)(0.0002)(0.5) = 0.180"$
3. $M_p = (0.864 + 0.18) \sin 25° = 0.441"$
4. $M_n = (0.864 + 0.18) \cos 25° = 0.946"$

B. Selection of Seal Width

1. $W = 4(\cos 25°)[(0.75)(0.864) + 0.18] = 3.00"$
2. $W = 0.441 \div 0.22 = 2.00"$
3. $W = 0.946 \div 0.45 = 2.10"$
4. Therefore use W = 3.00"

WA-300	W = 3.00”
CV-3000	W = 3.00”

SELECTION CRITERIA FOR COMPRESSION SEALS

C. Width of expansion joint opening at 60°F:

$A = (0.60)(3.00) = 1.80''$

D. Adjustment for 10°F temperature change

$\Delta = (12)(150)(0.000006)(10°)(\cos 25°) = 0.098''$

SELECTION CRITERIA FOR SILICONE SEALS

I. Design Limitations

A. Sealant designed to accommodate 100% tension and 50% compression.
B. Use on rehabilitation projects.
C. Temperature Range
Concrete structures........… 0° to 80°F
Steel structures........……. -30° to 120°F

II. Design Procedure

A. Movement Calculations
1. $M_t = 12(L)(\alpha)(\text{temp range})$
2. $M_s = 0$
3. $M_{normal} = (M_t + M_s)(\cos\theta)$
4. $M_{10F} = 12(L)(\alpha)(10°F)$

B. Expansion gap widths at assumed extreme installation temperatures
1. $G_{min} = G_{exist} + (T_{normal}\text{-}Install_{min})/10(M_{10F})$
2. $G_{max} = G_{exist} – \ Install_{max}\text{-}T_{normal})/10(M_{10F})$

C. Check sealant capacity if installed at assumed minimum temperature
1. Closing movement
$M_c = (T_{max}\text{-}Install_{min})/10(M_{10F})$
2. Check 50% compression
Mc/Gmin < 0.50
3. Opening Movement
$M_o = (Install_{min} – T_{min})/10(M_{10F})$
4. Check 100% tension
$M_o/G_{min} < 1.00$

D. Check sealant capacity if installed at assumed maximum temperature
1. Closing movement
$M_c = (T_{max}\text{-}Install_{max})/10(M_{10F})$
2. Check 50% compression
$Mc/G_{max} < 0.50$
3. Opening Movement
$M_o = (Install_{max} – T_{min})/10(M_{10F})$
4. Check 100% tension
$M_o/G_{max} < 1.00$

III. Design Example

Existing 25 year old concrete bridge with 1” expansion gaps at each abutment at 60°F
160’ total length
Skew angle = 15°

Value of constants
$\Theta = 15°$
$\alpha = 0.0000060$
L = 160/2 = 80’
$T_{min} = 0°$
$T_{max} = 80°$
$T_{normal} = 60°$
$G_{exist} = 1”$
$Install_{min} = 40°$
$Install_{max} = 80°$

A. Movement Calculations
1. $M_t = (12)(80’)(0.0000060)(80°\text{-}0°) = 0.461”$
2. $M_s = 0$

SELECTION CRITERIA FOR SILICONE SEALS

3. M_{normal} = (0.461” + 0)(cos15°) = 0.445”
4. M_{10F} = (12)(80’)(0.0000060)(10°)(cos15°) = 0.0556”

B. Expansion gaps at assumed extreme installation temperatures
   1. G_{min} = 1” + (60°-40°)/10(0.0556) = 1.111”
   2. G_{max} = 1” - (80°-60°)/10(0.0556) = 0.89”

C. Check Sealant Capacity at minimum installation temperature
   1. M_c = (80°-40°)/10(0.0556) = 0.224”
   2. 0.224/1.111 = 0.20 < 0.50 OK
   3. M_o = (40°-0°)/10(0.0556) = 0.224”
   4. 0.224/1.111 = 0.20 < 1.00 OK

D. Check Sealant Capacity at maximum installation temperature
   1. M_c = (80°-80°)/10(0.0556) = 0.0”
   2. 0.0/0.89 = 0. 0 < 0.50 OK
   3. M_o = (80°-0°)/10(0.0556) = 0.445”
   4. 0.445/0.89 = 0.50 < 1.00 OK

SELECTION CRITERIA FOR STRIP SEALS

I. Design Limitations

A. Total anticipated movement of the expansion joint should not exceed 4". When the nominal seal width computed by the following procedure exceeds 4", a joint system with greater movement capacity is required. The movement is measured along centerline of bridge.

B. The minimum joint opening at installation of the seal shall not be less than 1.5" normal to the joint.

C. Skewed joints are classified as follows:

TYPE	SKEW ANGLE
1	≤30°
2	>30° ≤ 45°
3	>45°

For skews greater than 45° also contact the manufacturer's representative for help in selecting both the joint type and size.

D. Temperature Range

Concrete structures........… 0° to 80°F
Steel structures........……. -30° to 120°F

II. Design Procedure

A. Movement Calculations

1. Calculate the joint opening movement due to temperature drop from the installation temperature and shrinkage.

2. a. Calculate the total closing movement due to temperature rise from the installation temperature.

   b. Convert the 1.5" minimum installation width normal to the joint to a length along centerline of bridge.

   c. Use the larger value obtained from (a) or (b).

3. The total movement along the centerline of bridge is equal to (1) + (2).

B. Joint Size

1. Type 1 Joints: The joint size required equals the total movement along the centerline of bridge.

2. Type 2 Joints: The joint size required equals the larger of:
   - The total movement along the centerline of bridge,
   - The movement parallel to the joint centerline divided by 0.60.

3. Type 3 Joints: The joint size required equals the larger of:
   - The total movement along the centerline of bridge,
   - The movement parallel to the joint centerline divided by 0.50.

C. Calculate the width of expansion joint opening at 60°F. The width along centerline of bridge equals the total closing movement plus the gap at full closure.

D. Calculate the adjustment in joint opening for a 10°F change in temperature.

SELECTION CRITERIA FOR STRIP SEALS

III. Design Example 1

Structure type, prestressed girder
Total length, 400'.
Skew angle, 30°.
Expansion joints at both abutments.
Point of no movement for temperature and shrinkage is at the center of the bridge.
Value of Constants:

$\theta = 30°$
$\alpha = 0.000006/$☐F
$\beta = 0.0002$
$\mu = 0.5$
$L = 400' \div 2 = 200'$

A. Movement Calculations

1. Opening Movement

$M_t = (12)(200)(0.000006)(60\text{-}0) = 0.864"$
$M_s = (12)(200)(0.0002)(0.5) = 0.24"$
Total opening movement = 1.104"

2. Closing Movement

a. $M_t = (12)(200)(0.000006)(80\text{-}60) = 0.288"$
b. Assume 0" min. gap (1.5-0)/cos 30° = 1.732"
c. Total closing movement = 1.732"

3. Total Movement = 1.104 + 1.732 = 2.836"

B. Joint Size

Type 1 joint
Total = 2.836"
SE-300: total movement = 3.00" min. gap = 0"
A2R-400: total movement = 4.00" min. gap = 0.5"

C. Joint width at 60°

(1.5)/cos 30° = 1.732"
(0.5)/cos 30° = 0.577"
Total = 2.309"

D. Adjustment in joint opening for a 10°F change in temperature:

$\Delta = (12)(200)(0.000006)(10°)(\cos 30°) = 0.125"$

SELECTION CRITERIA FOR STRIP SEALS

III. Design Example 2
Structure type, concrete box girder
Total length, 600'.
Skew angle, 35°.
Expansion joints at both abutments.
Point of no movement for temperature and shrinkage is at the center of the bridge.
Value of Constants:

$\theta = 35°$
$\alpha = 0.000006/°F$
$\beta = 0.0002$
$\mu = 0.8$
$L = 600' \div 2 = 300'$

A. Movement Calculations

1. Opening Movement

$M_t = (12)(300)(0.000006)(60\text{-}0) = 1.296''$
$M_s = (12)(300)(0.0002)(0.8) = 0.576''$
Total opening movement = 1.872"

2. Closing Movement

a. $M_t = (12)(300)(0.000006)(80\text{-}60) = 0.432''$
b. Assume 0" min. gap (1.5-0)/cos 35° = 1.831"
c. Total closing movement = 1.831"

3. Total Movement = 1.872 + 1.831 = 3.703"

B. Joint Size

2a. Type 2 joint Total = 3.703"

2b. $M_p = (3.703)(\sin 35°) = 2.124''$
2.124/0.6 = 3.540"
SE-400: total movement = 4.00" min. gap = 0"
A2R-400: total movement = 4.00" min. gap = 0.5"

C. Joint width at 60°

(1.5)/cos 35° = 1.831"
(0.5)/cos 35° = 0.610"
Total = 2.441"

D. Adjustment in joint opening for a 10°F change in temperature:

$\Delta = (12)(300)(0.000006)(10°)(\cos 35°) = 0.177''$

SELECTION CRITERIA FOR MODULAR JOINTS

I. Design Limitations

A. Maximum movement of 3” per each seal element.
B. Maximum gap between adjacent center beams should be limited to 3½”.
C. Movements should be increased 15% to provide a factor of safety.
D. Temperature Range
Concrete structures........ 0° to 80°F
Steel structures.............. -30° to 120°F

II. Design Procedure

A. Movement Calculations
1. Calculate the joint opening movement due to temperature drop from the installation temperature and shrinkage.
2. Calculate the joint closing movement due to temperature rise from the installation temperature.
3. Total movement (M_t) along centerline of bridge is the sum of (1) & (2) times 1.15.
4. Total movement normal to joint is $M_t(\cos\theta)$.

B. Joint Size
1. Total movement range (MR) should be a multiple of 3” based upon A3,

C. Installation Gaps
1. Compute the minimum distance face-face of edge beams (G_{min}).
Number of seals (n) = MR/3
Number of center beams = (n-1)
w = Center beam top flange width
g = Minimum gap per seal at full closure
$G_{min} = (n-1)(w) + (n-1)(g)$
2. Compute the maximum distance face-face of edge beams (G_{max}).
$G_{max} = G_{min} + MR$
3. Compute gaps at different temperatures.

D. Center Beam Spacing
1. Check spacing between center beams at minimum temperature.
$G_{0F} = G_{60F}$ + total opening movement
Spacing = $[G_{0F} – (n-1)(w)]/n$
2. Check spacing between center beams at 60°F for seal replacement.
Spacing = $[G_{60F} – (n-1)(w)]/n$

III. Design Example

Structure type - box girder
Total length – 1200’
Skew Angle - 15°
Expansion joints at both abutments
Point of no movement for temperature and shrinkage is at the center of the bridge

Value of Constants:
$\Theta = 0°$
$\alpha = 0.0000060$
$\beta = 0.0002$
$\mu = 0.8$
L = 1200/2 = 600’

A. Movement Calculations
1. Opening Movement
$M_{temp} = (12)(600)(0.0000060)(60-0) = 2.592”$
$M_{shrink} = (12)(600)(0.0002)(0.8) = 1.152”$

SELECTION CRITERIA FOR MODULAR JOINTS

Total = 3.744”

2. Closing Movement
M_{temp} = (12)(6300)(0.0000060)(80-60) = 0.864”

3. Total Movement along centerline of bridge
M_{total} = (3.744+0.864)(1.15) = 5.299”

4. Total movement normal to joint
M_{normal} = 5.299”(cos15) = 5.118”

B. Joint Size

1. MR = 5.118” Use a 6” movement rating joint.

C. Installation Gaps

1. Assume center beam top flange width =2½”
Number of seals = 6/3 = 2
Number of center beams = (2-1) = 1
Minimum gap per seal at full closure = 0”
G_{min} = (1)(2.50”) + (1)(0”) = 2.50”

2. G_{max} = 2.5” + 6” = 8.5”

3. G_{60F} = $_{Gmin\ +}$ total closing movement = 2.5” + (0.864”)(1.15)(cos15) = 3.46”
Adjustment in opening for a 10°F change in temperature = (12)(600)(0.0000060)(10)(cos15)= 0.417”
G_{40F} = 3.46” + (60-40)/10(0.417) = 4.29”
G_{80F} = 3.46” – (80-60)/10(0.417) = 2.63”

D. Center Beam Spacing

1. Spacing at minimum temperature
G_{0F} = 3.46” + (3.744)(cos15) = 7.08”
Spacing = [7.08 – (1)(2.50”)]/2 = 2.29” < 3.50” OK

2. Spacing at 60°F
Spacing = [3.46” – (1)(2.50”)]/2 = 0.48” Minimum recommended installation width = 1.5”. Center beam must be mechanically separated in order to replace strip seal elements.

Revisions:

July 2009 Added definition of μ on Page 1 and revised 0.8 factor to include c-i-p box girders & t-beams
Added Selection Criteria for modular Joints
Added Selection Criteria for silicone sealant joints

CHAPTER 14
STANDARD DRAWING REVISION LOG

B14.1A Compression Seal Expansion Joint skew ≤ 10°

June 2006 Revised Note 11 reference to AASHTO M-297. AASHTO M-220 applies to pavement joints.

B14.1B Compression Seal Expansion Joint skew > 10°

June 2006 Revised Note 11 reference to AASHTO M-297. AASHTO M-220 applies to pavement joints.

B14.2 Strip Seal Expansion Joint

June 2006 Added reference in Note 2 for ASTM D-5973.

B14.6 Asphaltic Plug Expansion Joint

April 2008 Clarified note for sealing top of joint.

B14.7A TFE/Stainless Steel Expansion Bearing – steel girder

June 2006 Revised Note 1 for anchor bolts to conform to ASTM F-1554 Grade 36.

Revised Note 5 for stainless steel sheets to conform to ASTM A240. ASTM A167 moved the grades of steel to ASTM A-240.

Revised the minimum thickness of PTFE and minimum recess to conform to AASHTO Article 14.7.2.3.1.

Revised the temperature in the Expansion Bearing Plan to 60°F to be consistent with expansion joint temperature.

Added Note 14 for temperature adjustment.

April 2008 Intermediate steel plate thickness changed to "X" to match required design thickness.

Added requirement for dimpled lubricated PTFE sheets and certification testing.

B14.7B TFE/Stainless Steel Expansion Bearing – concrete girder

June 2006 Revised Note 1 for anchor bolts to conform to ASTM F-1554 Grade 36.

Revised Note 5 for stainless steel sheets to conform to ASTM A240. ASTM A167 moved the grades of steel to ASTM A-240.

Revised the minimum thickness of PTFE and minimum recess to conform to AASHTO Article 14.7.2.3.1.

Revised the temperature in the Expansion Bearing Plan to 60°F to be consistent with expansion joint temperature.

Added Note 14 for temperature adjustment.

April 2008 Intermediate steel plate thickness changed to "X" to match required design thickness.

Added requirement for dimpled lubricated PTFE sheets and certification testing.

CHECKING PROCEDURE FOR PRESTRESSED GIRDER SHOP DRAWINGS

The following is a list of items that should be checked to insure a complete and thorough review of prestressed girder shop drawings.

Submittal Data

Strand Certification

- Strand type (stress relieved or low relaxation)
- Ultimate strength
- Strand Area
- Modulus of elasticity

Jacking Force/Gauge Pressure

- Data from the calibration test and a graph or equation representing the relationship between jacking force and gauge pressure shall be included.

Bed Layout

- Total length of strand being tensioned
- Hold-down point locations with horizontal and vertical dimensions
- Hold-up point locations with horizontal and vertical dimensions

Elongation Calculations

- Elongation of each strand to be measured and the corresponding gauge pressure

Checking Procedure

The tensioning method varies with each fabricator and the method of calculating the elongations varies accordingly.

Jack Data

- Jacking data shall fit the graph or equation submitted and the jack used in the test shall be the jack used for tensioning.

Strand Data

- Strand data shall match the certification sheet. The prestressing manufacturer must be reminded in the review letter that if the actual strand properties being used vary by more than 5% from the assumed values in the calculations then new elongation calculations will have to be made.

Initial Prestress Force

- An initial force and initial gauge pressure shall be given. This is the force in the strand before any elongation measurements are made.

Final Prestress Force

- This is the desired force in the strand after all the elongation losses have occurred.
  Low relaxation strand P_f = 0.75x270xstrand area
  Stress relieved strand P_f = 0.70x270xstrand area

Calculated Elongations

Δi = initial elongation = $P_i L/AE$
Δf = final elongation = $P_f L/AE$

Elongation Losses

- Elongation losses are treated differently depending on when they occur. There are three different times an elongation loss can occur:
  a. Before the strand is seated, Δ_B (dead-end seating or abutment deflection due to the jacking in progress).
  b. When the strand is seated, Δ_S (strand slipping in the chuck when released).
  c. After the strand is seated, Δ_A (abutment deflection due to subsequent jacking of other strands).

- The values for the three different losses should be given since they are unique to each prestressing yard.

Gauge Pressure for Straight Strands

- Use the gauge pressure vs. jacking force graph or equation and solve for the gauge pressure.
  Fj = jacking force = $(\Delta f + \Delta_S + \Delta_A)\ AE/L$

Elongation for Straight Strands

- Measured before seating $\quad \Delta_M = \Delta f - \Delta i + \Delta_B + \Delta_A + \Delta_S$
- Measured after seating $\quad \Delta_M = \Delta f - \Delta i + \Delta_B + \Delta_A$

Elongation for Harped Strand

- Use the geometry for the hold-down and hold-up points to calculate the increase in the strand length due to harping.
  Δ_H = elongation due to increase in length of harped strand
  Δ_{MH} = elongation of harped strand = $\Delta_M - \Delta_H$

Gauge Pressure of Harped Strand

- Use the gauge pressure vs. jacking force graph or equation and solve for the gauge pressure.
  Fj = jacking force = $(\Delta f - \Delta_H + \Delta_S + \Delta_A)\ AE/L$

The above calculations should be made for each strand and a table prepared listing the measured elongation and corresponding gauge pressure for each strand.

Refer to Appendix H of the PCI *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, 4th Edition, for sample tensioning calculations.

PRELIMINARY STRUCTURE COST ESTIMATE

The charts on the following sheets provide cost per square foot data for the preliminary estimate of total structure cost. The costs apply to ordinary structure types. The data is based upon the low bid from projects awarded from 2000 thru December 2007.

The total cost per square foot includes all the bid items necessary to construct the structure except for the following:

- Preliminary Engineering
- E & C
- Traffic Control

The costs for Repair and Rehabilitation projects require an individual estimate of cost for each project because of the wide diversity of work. Consult with the Group Leader/Bridge Engineer to obtain costs from similar projects.

Cost data is available for the following types of structures:

NEW CONSTRUCTION

- CULVERTS
- PRESTRESSED GIRDERS
- STEEL GIRDERS

MISCELLANEOUS

- CONCRETE RETAINING WALLS
- PEDESTRIAN BRIDGES
- WIDEN CULVERTS

PRELIMINARY STRUCTURE COST ESTIMATE

NEW CONSTRUCTION

STRUCTURE TYPE	COST/SQ. FOOT OF DECK PLAN AREA
	2000 thru 2007
CULVERTS	
CAST-IN-PLACE	\$165
PRECAST	\$165
PRESTRESS GIRDERS	
AASHTO, BULB TEE, DECK TEE	\$165
VOIDED SLAB, TRIDECK	\$215
STEEL GIRDERS	
WELDED PLATE	\$200

PRELIMINARY STRUCTURE COST ESTIMATE

MISCELLANEOUS STRUCTURES

STRUCTURE TYPE	COST/SQUARE FOOT
	2000 thru 2007
CONCRETE RETAINING WALLS	$50
PEDESTRIAN UNDERPASS	$135
PEDESTRIAN OVERPASS PRESTRESSED GIRDER, STEEL TRUSS	$155
WIDEN CULVERT	$90

COST ESTIMATE

GENERAL INSTRUCTIONS

Apply the Unit Costs to ordinary structures. Unit Costs should be increased 10-15% for culvert extensions and stage construction jobs, and 50% for complex modification jobs.

Unit Costs should generally be increased for jobs that are small or at remote sites.

Do not show Item 629 - Mobilization - on the bridge drawings, but show it on the Cost Estimate.

Plan Quantity items should **not** be denoted on the Cost Estimate or Plans. If you desire to pay "Plan Quantity" for an item not listed in the Standard Specifications, then the Specifications must be modified.

Cost Estimates for all projects (In-house and Consultant) shall be entered into the PC Cost Estimate database program.

Commentary

The unit prices are based upon trend lines of the weighted average of the 3 low bidders for each year and trend lines of all the unit costs of all the bidders based upon the bid opening date. The data is taken from the abstract of bids from January 2005 through June 2009.

AVERAGE UNIT PRICES FOR STANDARD BID ITEMS

(Based Upon 3 Low Bids from Jan 2005 thru June 2009)

SECTION 203 - REMOVAL OF OBSTRUCTIONS

ITEM NO.	ITEM	UNIT	ALL DISTRICT S
203-020A	Removal of Single Span Bridges	SF	$17
203-020A	Removal of Multi-span Bridges	SF	$20
203-020A	Removal of Truss Bridges	SF	$15
203-035A	Removal of Culverts	SF	$15

NOTE : Use square foot cost for estimating purposes but show as cost per EACH on the Cost Estimate.

SECTION 205 - EXCAVATION & EMBANKMENT

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
205-040A	Granular Borrow	CY	$15

SECTION 210 - STR. EXC. & COMP. BACKFILL

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
210-005A	Structure Excavation Sch. No. 1	CY	$15
210-015A	Compacting Backfill	CY	$11

NOTE : Multiply Str. Exc. unit cost by 2.0 if underwater excavation involved, and by 2.5 if rock excavation involved.

SECTION 301 – GRANULAR SUBBASE

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
301-010A	Granular Subbase	CY	$20

SECTION 502 - CONCRETE

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
502-005A	Concrete Class 15	CY	$320
502-025A	Concrete Class 40A	CY	$640
502-030A	Concrete Class 40B	CY	$580
502-140A	Concrete Class 40B Sch. No. 1	CY	$520
502-250A	Concrete Class 40A Sch. No. 2	CY	$580
502-345A	Seal Concrete	CY	$350
502-400A	AASHTO Type 2 Girder	FT	$200
502-400A	AASHTO Type 3 Girder	FT	$215
502-400A	AASHTO Type 4 Girder	FT	$225
502-415A	Bulb Tee Girders	LF/in^2 of area	$0.35
502-430A	Concrete Parapet	FT	$105
502-435A	Approach Slab	SY	$215
502-445A	Voided Slabs	LF/in^2 of area	$0.36
502-470A	Prestr. T-beam & DeckTee	LF/in^2 of area	$0.40
502-500A	Prestr. Box Beam	LF/in^2 of area	$0.35

NOTE : Reduce deck concrete costs by $20/CY when girder type makes slab forms unnecessary.
Use LF/in^2 of area for estimating, but show as FT on Cost Estimate.

AVERAGE UNIT PRICES FOR STANDARD BID ITEMS

(Based Upon 3 Low Bids from Jan 2005 thru June 2009)

SECTION 503 - METAL REINFORCEMENT

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
503-005A	Metal Reinforcement	LB	$1.15
503-010A	Metal Reinforcement Sch. No. 1	LB	$1.05
503-015A	Metal Reinforcement Sch. No. 2	LB	$0.90
503-020A	Epoxy Coated Metal Reinforcement	LB	$1.15

SECTION 504 - STRUCTURAL METALS

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
504-005A	Steel Bridge	LB	$1.70
504-015A	Structural Steel	LB	$1.60
504-030A	2 Tube Curb Mount Rail	FT	$175
504-035A	Pedestrian/Bicycle Rail	FT	$200
504-040A	Combination Pedestrian/Bicycle & Traffic Rail	FT	$170
504-040A	Combination Rail w/Ped Screen	FT	$200

NOTE : Steel Bridge - Use Cost/lb for estimating purposes but show as lump sum on the Cost Estimate. Reduce costs by $0.10/lb for rolled girders. Increase costs by $0.05/lb for haunched girders. Increase costs by $0.10/lb for curved girders.

SECTION 505 - PILING

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
505-025A	Furnish & Drive HP 12X53 Piling	FT	$105
505-030A	Furnish & Drive HP 12X74 Piling	FT	$115
505-045A	Furnish & Drive HP 14X117 Piling	FT	$105
505-100A	Furnish & Drive Shell Piling, 12" Diam	FT	$70
505-110A	Furnish & Drive Shell Piling, 16" Diam	FT	$120
505-205A	Furnish & Install 12" φ Shell Pile Shoes or Tips	EA	$320
505-205A	Furnish & Install 16" φ Shell Pile Shoes or Tips	EA	$350
505-205A	Furnish & Install HP 12x53 Pile Shoes or Tips	EA	$130
505-205A	Furnish & Install HP 12x74 Pile Shoes or Tips	EA	$145
505-205A	Furnish & Install HP 14x117 Pile Shoes or Tips	EA	$160

NOTE : For HP12 Test Piles add $0 to the cost of the Furnish & Drive item. For HP14 Test Piles add $25 to the cost of the Furnish & Drive item. For 12" φ Shell Test Piles add $5 to the cost of the Furnish & Drive item. For 16" φ Shell Test Piles add $60 to the cost of the Furnish & Drive item

SECTION 506 - PRESTRESSING CONCRETE

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
506-005A	Prestressing Cast-In-Place Concrete	LB	$1.80

NOTE : Use Cost/lb for estimating purposes but show as lump sum on the Cost Estimate.

AVERAGE UNIT PRICES FOR STANDARD BID ITEMS

(Based Upon 3 Low Bids from Jan 2005 thru June 2009)

SECTION 510 - CONCRETE OVERLAY

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
510-005A	Concrete Overlay	CY	$1100

SECTION 511 - CONCRETE WATERPROOFING SYSTEM

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
511-005A	Conc Waterproofing System, Type A or D	SY	$15
511-005A	Conc Waterproofing System, Type C	SY	$10

SECTION 623 - CONCRETE SLOPE PAVING

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
623-005A	Concrete Slope Paving	SY	$45

SECTION 624 - RIPRAP

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
624-005A	Loose Riprap	CY	$50

SECTION 629 - MOBILIZATION

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
Z629-05A	Mobilization	LS	

NOTE : Use 10% of total of all other bid items for small projects and 7.5% for large projects for estimating purposes but show as lump sum on Cost Estimate.

SECTION 632 - REMOVAL OF BRIDGE DECK CONCRETE

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
632-005A	Concrete Bridge Deck Removal Class A	SY	$95
632-010A	Concrete Bridge Deck Removal Class B	SY	$65

SECTION 640 – CONSTRUCTION GEOTEXTILES

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
640-010A	Riprap/Erosion Control Geotextile	SY	$2.00
640-015A	Subgrade Separation Geotextile	SY	$1.50

AVERAGE UNIT PRICES FOR SPECIAL PROVISION BID ITEMS

(Based Upon 3 Low Bids from Jan 2005 thru June 2009)

EXPANSION JOINTS

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-06A	Compression Seal Joints w/armor angles	LF/inch	$65
S501-06A	Compression Seal only	LF/inch	$20
S501-06A	Strip Seal Joints	LF/inch	$60
S501-06A	Modular Joints	LF/inch	$110
S501-05A	Asphaltic Plug Joints	CF	$700
S501-06A	Silicone Sealant	FT	$30
S501-40A	Elastomeric Concrete Header	CY	$14,000

NOTE: Use LF/inch for estimating, but show as FT on Cost Estimate.

SURFACE TREATMENT

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-60A	Textured Concrete Surface	SY	$50
S501-51A	Anti-Graffiti Coating	SF	$0.80

PRE-DRILLING FOR PILES

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S505-05A	Predrilling For Piling in Soil	LF/inch of diam	$3.50
S505-05A	Predrilling For Piling in Rock	LF/inch of diam	$11.00

NOTE: Use LF/inch of diameter for estimating, but show as FT on Cost Estimate.

DEWATER FOUNDATION

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S500-11A	Cofferdam	SF	$40
S500-11A	Other	LS	$35,000

NOTE : Use Cost/sf of cofferdam form area for estimating purposes but show as lump sum on the Cost Estimate. Cofferdam form area is equal to the perimeter of the seal concrete times the difference in elevation of the bottom of the seal concrete and the cofferdam vent elevation.

RAIL RETROFIT

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-30A	Std Drawing G-2-F Type 1	FT	$120
S501-30A	Std Drawing G-2-F Type 2	FT	$130
S501-30A	Delaware Thrie Beam	FT	$130
S501-20A	Remove End Block	EA	$1600

RETAINING WALL

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-17A	MSE Wall	SF	$45
S501-18A	Coping	FT	$95
S501-17A	Segmental Block Wall	SF	$30
S501-17A	Welded Wire Wall	SF	$40
S501-17A	Welded Wire Wall w/Concrete facing	SF	$45

AVERAGE UNIT PRICES FOR SPECIAL PROVISION BID ITEMS

(Based Upon 3 Low Bids from Jan 2005 thru June 2009)

TEMPORARY SHORING

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-90A	Bridge	LS	$70,000
S501-90A	Culvert	LS	$45,000

PATCH & REPAIR CONCRETE SURFACE

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-51A	Substructure	SF	$230
S501-51A	Deck	SF	$100

CRACK REPAIR

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-30A	Preparation	FT	$75
S501-50A	Injection	GAL	$2000

REMOVE ASPHALT OVERLAY

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-35A	Remove Asphalt Overlay	SY	$12

PAINT STRUCTURAL STEEL

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-70A	At Expansion Joints in Field	SF of girder	$40

UTILITY CONDUITS

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-30A	Complete across bridge	FT/conduit	$20
S501-30A	At Abutments with diaph sleeves & deck inserts	FT/conduit	$10

HMWM DECK SEAL

ITEM NO.	ITEM	UNIT	ALL DISTRICTS
S501-35A	Preparation	SY	$15
S501-50A	Sealer	GAL	$135

BRIDGE IMPROVEMENT TYPE CODE

The bridge improvement type code is a two digit number that is needed on the ITD-2101form, Project Authorization and Agreement, by the FHWA for their approval process. The following codes should be used:

08 Bridge – New Construction
Construction of a new bridge that does not replace or relocate an existing bridge.

10 Bridge Replacement – Added Capacity
Total replacement of a structurally inadequate or functionally obsolete bridge with a new structure constructed with additional lanes in the same general traffic corridor to current geometric construction standards. Incidental roadway approach work is included. The use of this code requires a National Bridge Inventory structure number (Bridge Inspection Master Key Number).

11 Bridge Replacement – No Added Capacity
Total replacement of a structurally inadequate or functionally obsolete bridge with a new structure constructed without adding lanes in the same general traffic corridor to current geometric construction standards, or widening the lanes and/or shoulders of an existing structure without adding through lanes. A bridge removed and not replaced or replaced with a lesser facility is considered a bridge replacement. Incidental roadway approach work is included. The use of this code requires a National Bridge Inventory structure number (Bridge Inspection Master Key Number).

13 Bridge Rehabilitation – Added Capacity
The major work required to restore structural integrity of a bridge as well as work necessary to correct major safety defects. Bridge deck replacement (both partial and complete) and widening of bridges including addition of through lanes to specified standards are included. Construction of a dual structure to alleviate a capacity deficiency is also included. Work required to correct minor structure and safety defects or deficiencies, such as deck patching, resurfacing, protective systems, upgrading railings, curbs and gutters, and other minor bridge work. If HBRRP funds are involved, the use of this code requires a National Bridge Inventory structure number (Bridge Inspection Master Key Number).

14 Bridge Rehabilitation – No Added Capacity
The major work required to restore structural integrity of a bridge as well as work necessary to correct major safety defects. Bridge deck replacement (both partial and complete) and widening of bridges without adding through lanes to specified standards are included. Work required to correct minor structure and safety defects or deficiencies, such as deck patching, resurfacing, protective systems, upgrading railings, curbs and gutters, and other minor bridge work. If HBRRP funds are involved, the use of this code requires a National Bridge Inventory structure number (Bridge Inspection Master Key Number).

28 Facilities for Pedestrians and Bicycles
An independent project (not part of any other Federal-Aid Highway project) to construct a facility to accommodate bicycle transportation and pedestrians.

QUANTITIES

General

The quantities of the various materials involved in the construction of a project are needed for determining the estimated cost of the project and for establishing a base for the contractor's bid and payment.

Upon completion of structural design and detailing of plans, the quantities of materials in the construction of the project shall be computed. Quantities are to be computed and checked independently. Final quantities to be listed in the Special Provisions and Bid Proposal sheet are to be calculated to have an accuracy of +/- 1 percent.

Method of measurement for the various materials shall be in accordance with the ITD *Standard Specifications for Highway Construction*, current edition, and Supplemental Specifications.

Section 210 – Structure Excavation and Compacting Backfill

Structure Excavation, Schedule No. 1, shall include excavation for bridges, box and stiffleg culverts, and Structure Excavation, Schedule No. 2, shall include excavation for all other structures.

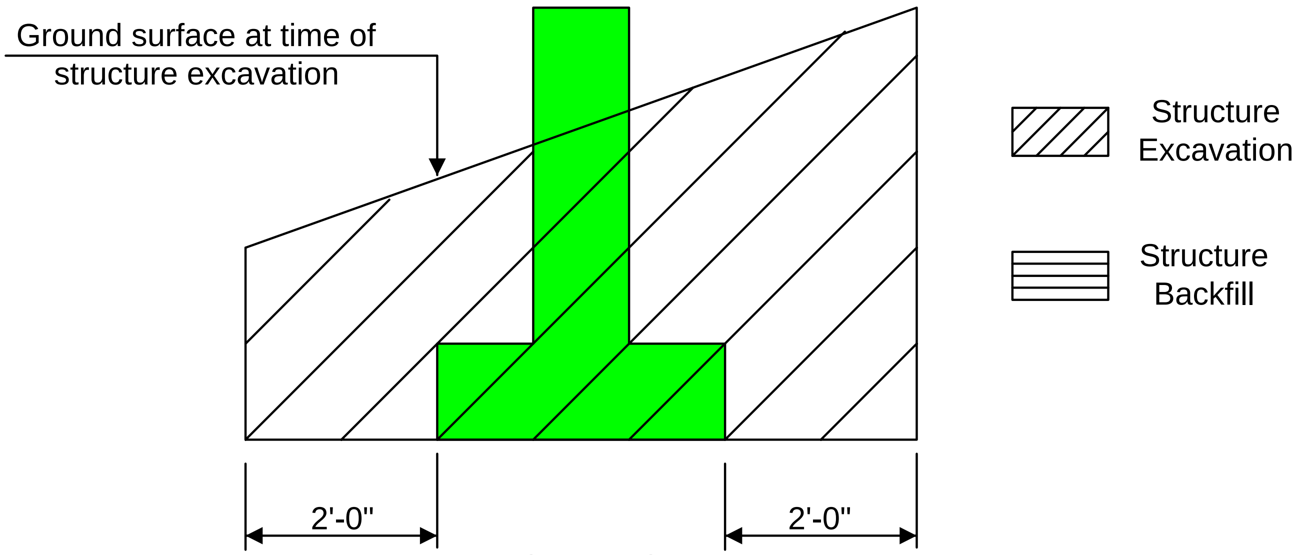
Structure excavation will be measured by the cubic yard of material in its original position, using the average end area method. The volume of material actually removed shall be measured within a prism with limiting planes as follows:

1. Conduit and Structural Plate Pipe: As shown on the plans.
2. Other Structures:
    a. The bottom of the foundation.
    b. The vertical planes 2 ft. outside of and parallel to the outside lines of the structure, in the case of bents with individual column footings, the entire bent shall be considered as one structure.
    c. With upper limits as follows:
        (1) In embankment sections, the existing ground surface as cross sectioned.
        (2) In roadway cut sections or channel changes, the planes of the roadway cut or channel change as excavated.

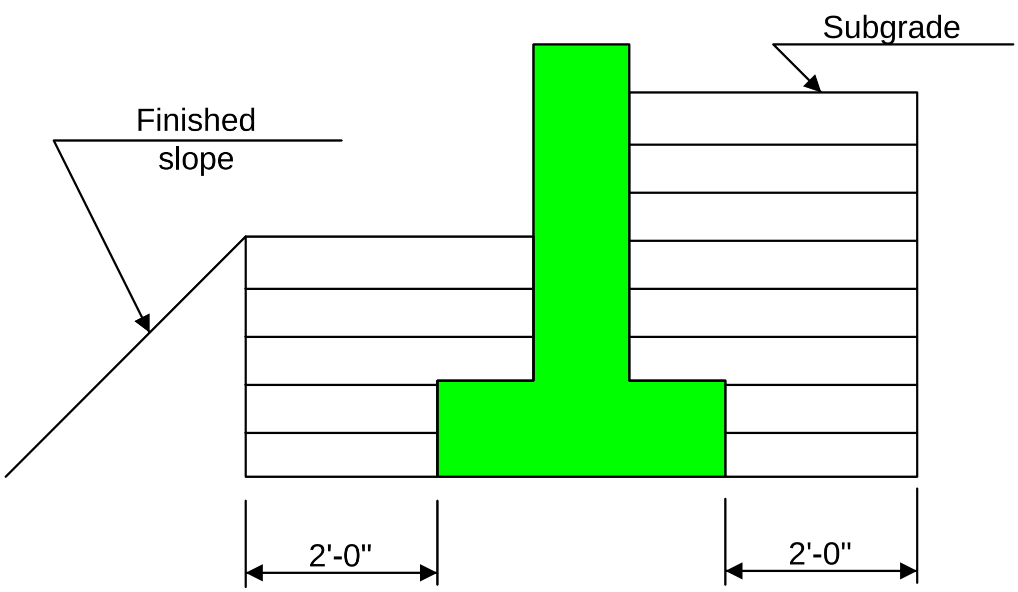
Compacting backfill will be measured by the cubic yard of backfill material placed. The volume will be determined as follows:

1. Conduit: As shown on the plans.
2. Other Structures:
    a. Below the original ground surface: A volume equal to the volume of structure excavation less the volume of the permanent structure including opening, contained within the limits of measurement for structure excavation.
    b. Above the original ground surface: The volume contained between the outside walls of the structure and vertical planes 4 ft. outside thereof; the original ground surface; and a horizontal plane 1 ft. above the top of the structure or of the subgrade, whichever is the lesser.
    c. Volumes of backfill placed through water around abutments, wing walls and piers, will not be included in the measurement of quantities for compacting backfill.

STRUCTURE IN EMBANKMENT

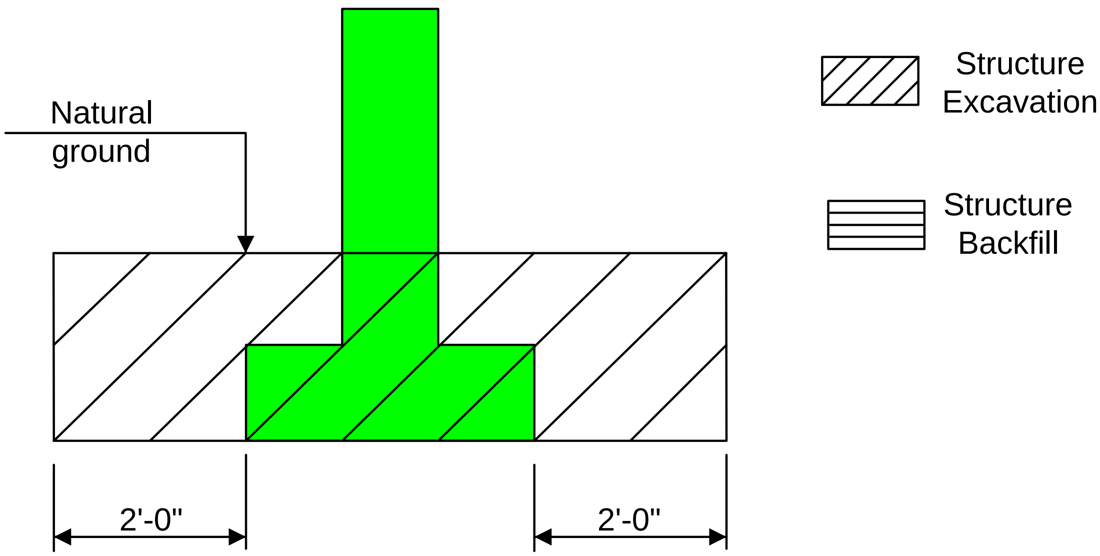


EXCAVATION



BACKFILL

STRUCTURE IN NATURAL GROUND



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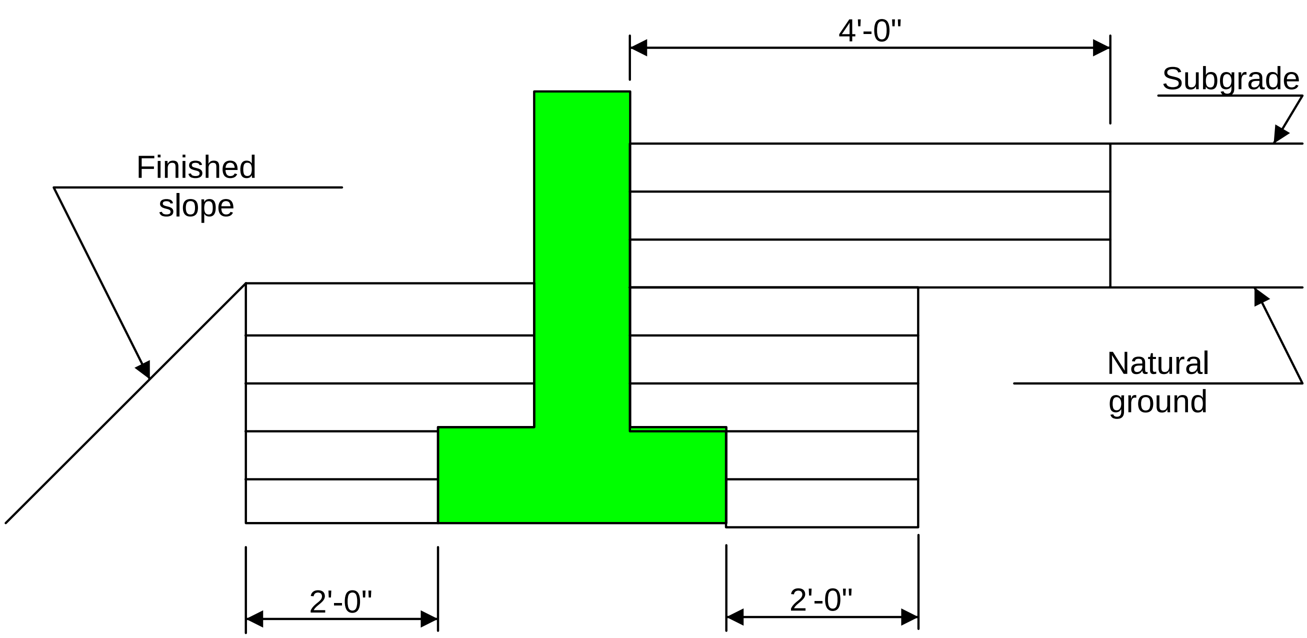
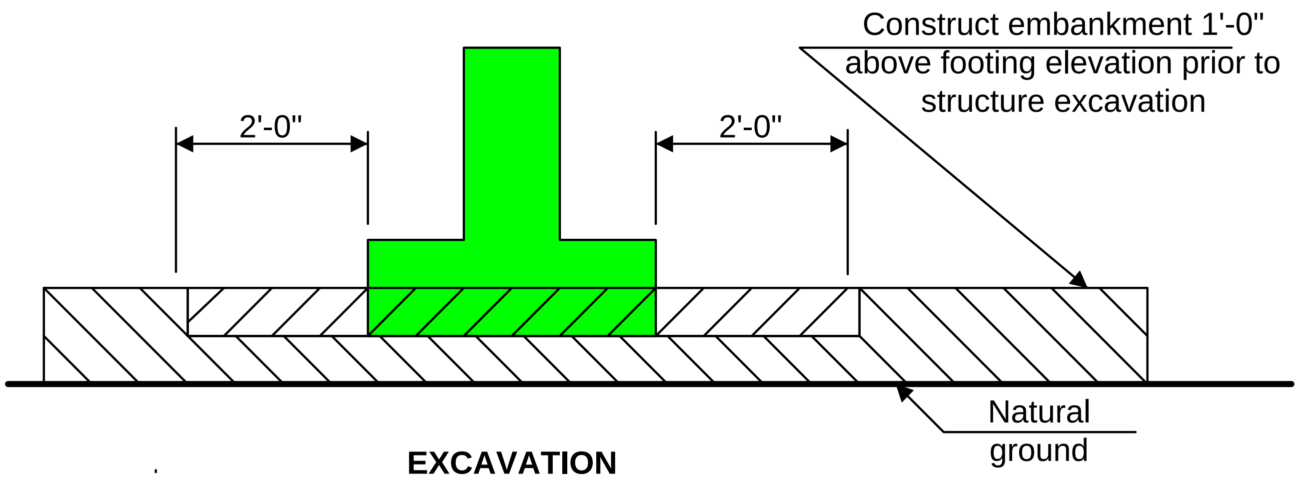
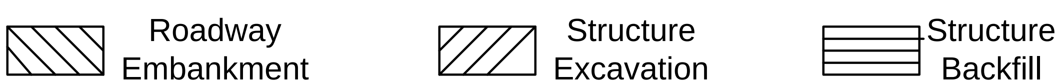


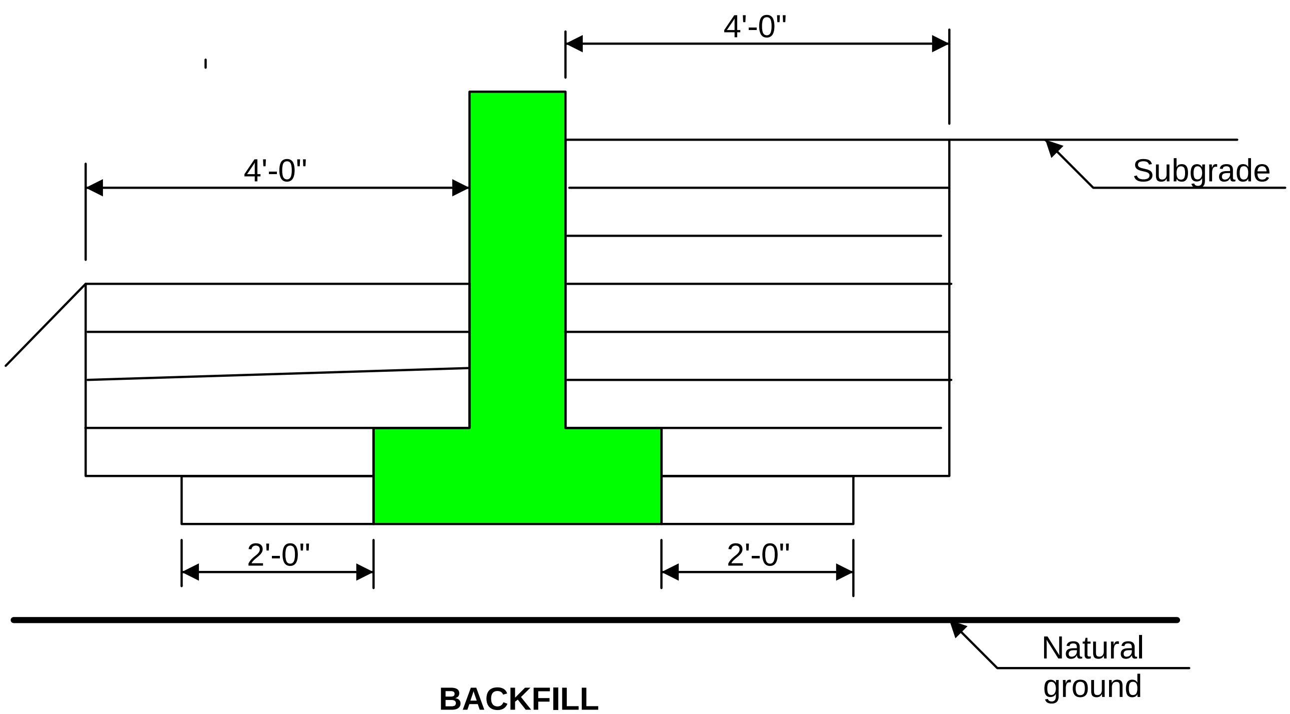
BACKFILL

STRUCTURE ABOVE NATURAL GROUND



EXCAVATION





BACKFILL

Precision of Units

The precision of the units to be shown on the Cost Estimate is shown in the following table.

ITEM NUMBER	DESCRIPTION	UNIT PRECISION
205-F	Granular Borrow	Whole CY
210-005A	Structure Excavation Sch. No. 1	Whole CY
210-015A	Compacting Backfill	Whole CY
502-005A to 502-350A	Concrete – All classes	Nearest 0.1 CY
502-400A to 502-422A	Prestressed Girders	Nearest 0.1 FT
502-430A	Concrete Parapet	Nearest 0.1 FT
502-435A	Approach Slab	Nearest 0.1 SY
502-440A to 502-500A	Prestressed Slabs, T-beams, Box Beams	Nearest 0.1 LF
503-005A to 503-020A	Metal Reinforcement	Nearest LB
504-005A to 504-015A	Structural Steel	Nearest LB
504-025A to 504-040A	Railing	Nearest 0.1 FT
505-020A to 505-110A	Furnish & Drive Piling, Test Piling	Nearest LF
510-005A	Concrete Overlay	Nearest 0.1 CY
511-005A	Concrete Waterproofing System	Nearest 0.1 SY
623-005A	Concrete Slope Paving	Nearest 0.1 SY
624-005A or 624-015A	Riprap	Whole CY
632-005A to 632-010A	Concrete Bridge Deck Removal	Nearest 0.1 SY
S501-06A	Expansion Joints	Nearest 0.1 FT
S501-60A	Textured Concrete Surface	Nearest 0.1 SY
S501-70A	Paint Concrete	Nearest 0.1 SF
S501-30A	Pre-drilling for Piles	Nearest 0.1 FT
S501-30A	Rail Retrofit	Nearest 0.1 FT
S501-17A	MSE Wall	Nearest 0.1 SF
S501-18A	Coping for MSE Wall	Nearest 0.1 FT
S501-51A	Patch & Repair Concrete Surface	Nearest 0.1 SF
S501-30A	Crack Preparation	Nearest 0.1 FT
S501-50A	Crack Injection	Whole GAL
S501-35A	Remove Asphalt Overlay	Nearest 0.1 SY

ELIGIBLE & EXEMPT ITEMS FOR COST ESTIMATE

GENERAL

The bridge construction unit costs are submitted yearly to FHWA. This cost data is used in the apportionment process as described in Title 23, U.S.C. Section 144(e). The unit cost report is for new and replaced highway bridges constructed with Federal funds on Federal-aid highways (highway bridges on the NHS and other Federal-aid highways) and on non Federal-aid highways (highway bridges on local roads and rural minor collectors; off-system highway bridges). Unit costs shall be based on bridge costs only.

To make this data as accurate as possible and uniform nationwide, the following guidelines for eligible costs should be followed when coding the Bridge Cost Estimate database.

EXEMPT ITEMS (Items not to be included in the Unit Cost Calculations)

Mobilization
Demolition of existing bridge
Stream channel work
Riprap
Slope paving
Earthwork (exclusive of structural excavation and structural backfill)
Clearing and grubbing
Retaining walls not attached to the abutment
Guardrail transitions to bridges
Maintenance and protection of traffic
Detour costs
Signing and marking
Lighting
Electrical conduit
Inlet frames and grates
Field office
Construction engineering items
Training
Right-of-way
Utility Relocation
Contingencies
Painting steel or concrete members for aesthetic purposes
Textured concrete surface

ELIGIBLE ITEMS (Items to be included in the Unit Cost Calculations)

As a general rule, if the item is necessary to construct the bridge, it is eligible. The following items have been ruled eligible by FHWA HQ.

Temporary shoring for bridge construction (formwork, falsework, and retention of roadway fill)
Work bridges or platforms
Deck treatments or wearing surface overlays
Deicing systems used for deicing decks for traffic safety
Approach slabs (when paid for as a bridge item, e.g. on integral abutment bridges)
Painting steel members to prevent corrosion

Commentary

The above information is based on FHWA Memorandum for Bridge Construction Unit Costs Attachment C and on information from Greg Kolle, FWWA Nebraska Division Bridge Engineer.

Revisions:

April 2008	Added new article.

DETAILING PRACTICES & PLAN PREPARATION

PURPOSE

The purpose of these standards is to enable the Bridge Section to produce consistent and effective plan sheets which will have uniform appearance and information. Engineers and detailers are responsible for ensuring that the criteria are implemented.

DRAWING ORIENTATION & LAYOUT CONTROL

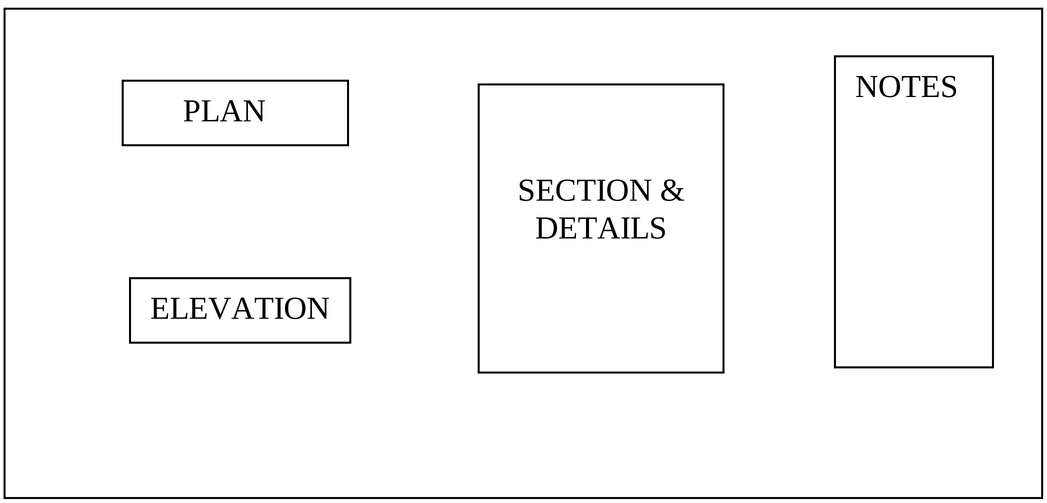
The standard sheet for bridge drawings is 34"x22" with the title block as shown on page B17.3.

Plastic lead or ink shall be used on mylar drafting film for making changes and signing drawings.

Drawings shall be carefully organized so the intent of the drawing can be easily read.

- North arrows shall be shown on Situation layout and Footing Layout sheets.
- Related details shall be grouped together in an orderly arrangement.
- The drawing should not be overcrowded with details.

The standard sheet configuration should be as follows:



PLAN ORGANIZATION

Plan sheets shall be assembled in the normal order of construction as follows:

Situation and Layout
Sheet Index, Quantities, & Vicinity Map
Design and General Notes
Foundation Investigation
Stage Construction Details
Footing Layout and Pile Details
Abutment Details
Wingwall Details
Bent or Pier Details
Framing Plan
Girder Details
Deck Typical Section
Deck Details and Pour Sequence
Bearing Details
Expansion Joint Details
Railing/Parapet Details
Approach Slab Details
Reinforcement Details

LETTERING & DIMENSIONING

Lettering shall follow the CADDS standards shown in Article 17.4.

Underline all titles with a single line having the same weight as the lettering used with the scale noted beneath the line.

Lettering shall be oriented so as to be read from the bottom or right edge of the sheet.

A dimension shall be shown once on a drawing, unless repeating it is necessary for clarity. All dimensions shall be placed above the dimension line so that they may be read from the bottom or the right edge of the sheet.

Reinforcing bar clearances need not be specified unless different from the General Notes sheet.

When details or structural elements are complex, utilize two drawings; one for dimensions and the other for reinforcement details.

Dimensions 12" or more shall be shown in feet & inches unless the item dimensioned is conventionally designated in inches; e.g. 16"φ pipe.

Dimensions should be placed outside the view. However, in the interest of clarity and simplicity, it may be necessary to place them otherwise.

Breaks are allowable in lines that cross, provided that their intent is clear. Detail or object lines take precedence over dimension lines; dimension lines take precedence over note or information lines.

REINFORCEMENT CALL-OUT FORMAT

Rebar shall be noted as follows: 20 F1~#6 19 spaces @ 12" = 19'-0"

Rebar sets shall be noted as follows: 1 set W1~#8 9 spaces @ 6" = 4'-6"

Epoxy coated rebar shall be denoted as follows: 25 S1~# 6(E) 24 spaces @ 12" = 24'-0"

SCALE

An engineering scale shall be used for the Situation Layout sheet and may be used for framing plan sheets. An architectural scale shall be used for all detail sheets.

All Plan, Elevation, and Details should be drawn to a scale to be clearly represented. The use of "not to scale" details should be limited to a minimum and used only on very simple details.

When selecting a scale, it should be kept in mind that the drawing will be reduced. Generally, the minimum acceptable scale is as follows:

- Situation and Layout. 1" = 20'
- Section Details with rebar. ⅜" = 1'-0"
- Details for steel sections. ½" = 1'-0"
- Plan View without rebar. ¼" = 1'-0"

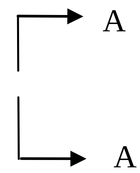
Section, details and views may be enlarged for clarity, but the number of different scales used on a sheet should be kept to a minimum.

SECTIONS, VIEWS, AND DETAILS

A Section cuts through the structure or object; a View is from the outside; a Detail shows part of a structural element in more detail and usually at a larger scale.

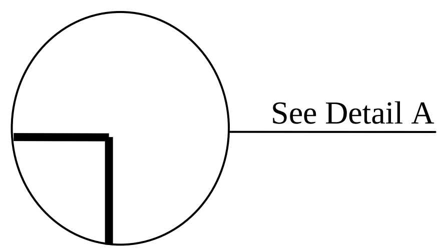
Care shall be taken to ensure that the orientation of a detail is identical to that of the plan, elevation, etc., from which it is taken.

Section or Views are noted by using an arrow with a letter as follows:

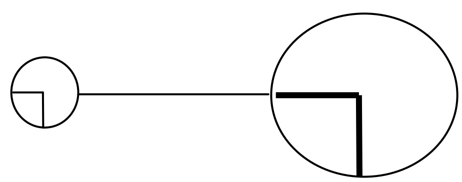


If the section can not be placed on the same sheet where it is taken, the following note should be added next to the section symbol; "For Section A-A see sheet #_".

Details are noted by using a bubble with a letter and a leader line to a bubble around the specific detail or view to be expanded for clarity as follows:



or



The letter designation for views and details shall begin with "A" on each sheet; e.g., Detail A, View A-A.

Each structural component (abutment, pier, etc.) shall be detailed separately as a general rule. If similar components are the same except for height, then the details can be shown on a single sheet using a table for the variable dimensions.

REVISIONS

The revision block shall be used to record revisions made after the plans have been signed by the Bridge Engineer.

Minor revisions shall be done using plastic lead or ink and erasing or crossing out the items to be changed. Major revisions may require the changes be done using the CADD and resigning the drawing.

Revisions shall be noted by a number in a triangle next to the change. A number corresponding to the change shall be placed in the triangle in the revision block, followed by the date, the initials of the person making the change, and a brief description of the changes made on the drawing.

FINAL PLANS

The completed drawings shall be stored in the vertical plan file and the index filled out to indicate which packet contains the drawings. Both the full size stamped mylars and half size plans for bidding with electronic signature shall be stored in the vertical file.

Revisions:

June 2006	Added new article.
July 2009	Revised order of sheets in Plan Organization

SITUATION AND LAYOUT FOR HIGHWAY & WATERWAY CROSSINGS

PURPOSE

The purpose of the Situation and Layout sheet is to provide an accurate overview and orientation of the project. The Engineer, checker, and detailer are responsible to ensure this data is correct.

Once the Situation and Layout is approved, it should not be changed and final design should begin.

SITUATION AND LAYOUT

- Refer to the Bridge Design Manual page B17.4 for plan sheet format data.
- An engineering scale should be used for the sheet.
- The Situation and Layout data shall be the first sheet of the structure plans and shall consist of the following plan sheets.
  - Sheet 1 of the final plans shall contain the following items:
    - PLAN
    - ELEVATION
    - PROFILE DATA
    - HORIZONTAL ALIGNMENT DATA
    - HYDRAULIC DATA
  - Sheet 2 of the final plans shall contain the following items:
    - VICINITY MAP
    - SHEET INDEX
    - QUANTITIES
  - Sheet 3 of the final plans shall contain the following items:
    - DESIGN NOTES
    - GENERAL NOTES
- The PRELIMINARY Situation and Layout data should include a fourth sheet if necessary showing the following data:
  - Typical section (Could be put on sheet 3 and then removed on final plan preparation)
    - Curb-curb and out-out widths
    - Sidewalk and curb widths
    - Type of railing
    - Slab thickness
    - Slab reinforcement cover for both mats
    - Girder type and spacing
    - Centerline and profile grade point
    - Crown slope
  - Design features
    - Show enough details of design features to clarify the concept. This should include abutment and pier/bent elevations and/or sections.

- The Title Block shall be completed as follows:
  - Sheet Title: SITUATION AND LAYOUT
  - Project Description: The project description shall include the following:
    - Total length of a bridge to the nearest foot and clear span length of a culvert to a tenth of a foot.
    - Type of main supporting member
    - Names of features involved in the crossing
      - Examples:
        262’ STEEL GIRDER UNDERPASS
        ROBERTS I.C.
        I-15 STA 300+77.51   SH-48 STA 29+46.79

        256’ PRESTRESSED CONCRETE BRIDGE
        E. BRIDGE ST. OVER WEISER RIVER
        STA 8+24.78

15’ CONCRETE BOX CULVERT
US-95 OVER CALF CREEK
STA 289+30.00

Bridge Inspection Master Key: Obtain the correct number from Bridge Inspection. Only those structures that carry highway traffic or cross a highway require a number. The number needs to be shown only on sheet 1.

PLAN VIEW

- Title the view PLAN and show the scale factor below the title.
- Show the total length of structure (out -out of backwalls) along the survey line.
- Show the abutment/pier number, station, and finished grade elevation at the intersection of the abutment/pier centerline and the survey line at the following locations:
  - Begin/End of structure
  - Centerline bearing of abutments
  - Centerline of piers/bents
- Show the span lengths along survey line as follows:
  - Single Spans or End Spans: abutment centerline bearing - centerline pier/bent
  - Interior Spans: centerline pier/bent - centerline pier/bent
- Show the total bridge width (out - out). The width should include the parapet, curb or sidewalk.
- Show the curb-to-curb width.
- Show the roadway lane and shoulder widths.
- Show the lane direction and name of closest town/geographical feature in that direction.
- Show the North arrow.
- Show the intersection angle if not a 90º crossing.
- Show the horizontal and vertical clearances as follows:
  - Highway Crossings: Show the point of minimum vertical and horizontal clearance for the highway.
  - Stream Crossings: Show the point of minimum clearance above Q_{50} high water elevation.
- Identify the survey and profile lines.
- If the new structure is at or adjacent to an existing bridge, show enough details of the existing bridge to insure that all possible conflicts are taken into account in the layout of the new bridge. As-built plans or field measurements should be used to accurately depict the existing bridge.
- If the existing bridge is to be removed, show the drawing number of the existing bridge plans.
- Orientation of the PLAN view shall allow the ELEVATION view to be a direct projection beneath the PLAN view.
- Stationing for bridges shall be along the centerline of structure and shall advance from left to right on the sheet.
- Stationing for culverts shall be along the centerline of structure and shall advance from bottom to top of the sheet.
- Show the limits of riprap. If riprap is not included in the structure bid items, add a note referencing the roadway pay items.
- Contour lines, if shown, should not project through the structure limits, dimension lines, or notes. Contour lines should be drawn in gray tones so they will not dominate the PLAN view.
- Show any utilities crossing the structure and show the location of any deck drains.
- Show the location of a Survey Cap at the top of the parapet or curb. The note should read, "A Survey Cap will be furnished by the State and shall be installed by the Contractor".

ELEVATION VIEW

- Title the view ELEVATION and show the scale factor below the title.
- Show the total length between abutment centerlines of bearing along the survey line.
- Show the abutment/pier number and station at the following locations:
  - Centerline bearing of abutments
  - Centerline of piers/bents
- Show the span length.
- Show the span number for multi-span bridges.
- Identify the type of fixity between the substructure and superstructure at the abutments and piers/bents using the following designations:
  - E Expansion
  - P Pinned
  - F Fixed

- Show the minimum vertical clearances as follows:
  - Highway Crossings: Show the minimum vertical clearance for the highway to the nearest tenth of a foot and locate the point.
  - Stream Crossings: Show the minimum clearance above Q_{50} high water elevation to the nearest tenth of a foot and locate the point.
- Show the natural ground line along the centerline of structure.
- Show the abutment slopes and call out the slope perpendicular to the stream or highway.
- The ELEVATION view should be a projection of the PLAN view. Show the end projection only for the abutments and piers. Showing the actual projection for skewed bridges is confusing.
- Show the roadway approach guardrail and reference the roadway plans for details.

PROFILE DATA

- Title the view PROFILE DATA. The view can be drawn "Not to Scale".
- Show the profile grade across the structure.
- Show the location of the structure on the alignment.
- Show the begin/end of bridge station and elevation.
- Show the profile grades for all highways involved in the crossing.
- Show the following vertical curve data:
  - Stations and elevations at point of curvature, point of intersection, and point of tangency.
  - Length of vertical curve
  - Incoming and outgoing grades in percent

HORIZONTAL ALIGNMENT DATA

- Title: HORIZONTAL ALIGNMENT DATA.
- Show the stations at point of curvature, point of intersection, and point of tangency on the PLAN view if possible. If not possible, list the stations in the curve data.
- Show the following horizontal curve data:
  - Δ, T, L, R, S, RL, and Z.
  - Horizontal curves shall be described by the degree of curve.
- Show the superelevation transition data if applicable. Cross-sections at the control points are recommended.
- If the structure is on a tangent alignment, show the bearing in the PLAN view.

HYDRAULIC DATA

- Title: HYDRAULIC DATA
  - Show the following hydraulic data for streams and rivers:

FLOOD	DISCHARGE	H.W. ELEVATION	VELOCITY
Design (Q_{50})	cfs	ft	fps
Base (Q_{100})	cfs	ft	fps
Scour (Q_{sc})	cfs	ft	fps

  - Show the following hydraulic data for canals:

Canal Flow	cfs
H.W. Elevation	ft
Velocity	fps

    Flow controlled by __________________ Canal Company.
  - Hydraulic data is not required for minor structure rehabilitation or extension projects.

VICINITY MAP

- A map of the State of Idaho showing location of the project.
- A vicinity map showing the location of the bridge site.

INDEX OF SHEETS

- Title: INDEX OF SHEETS
- The bridge plans shall be numbered independently from the roadway plans and shall start with sheet 1.

<u>QUANTITIES</u>

- Title: <u>QUANTITIES</u>.
- Show all the bid items listed on the cost estimate for the structure except Mobilization.
- The quantities do not need to be shown until the final plans are prepared.

<u>DESIGN & GENERAL NOTES</u>

The Design Notes shown on page B17.1A – B17.2D of the Bridge LRFD Manual are intended to be used as a checklist for the usual situation and should be modified to fit each individual case.

- Multi-span prestressed girder bridges should include one of the following notes as specified on page 5.14.1.2.7:
  a. Girders designed as simple spans and reinforcement added to resist negative moment.
  b. Girders designed fully continuous for live load.
  c. Girders designed as simple spans; slab reinforcement added to limit cracking.
- The computed and ultimate values for the Pile/Footing Design Loads should be shown on the final Situation and Layout submittal.
- Projects involving rehabilitation or repair should add the following note under CONSTRUCTION: “The contractor shall verify dimensions in the field before ordering material.”
- The estimated ADT data 20 years after the projected year of construction should be used to compute the single lane ADTT. The 20 year projected single lane ADTT shall be shown on the plan sheet with its corresponding year.

Revisions:

June 2006	Article was renumbered to 17.2 to allow for addition of new Article 17.1.
	Deleted Traffic Data from Situation Layout requirements. Single lane ADTT was added to the Transient Loads on the Design & General Notes sheet.
April 2008	Added paragraph for Vicinity Map, Index, & Quantities Sheet.
July 2009	Revised “Index of Sheets” to “Sheet Index” on page 1.

SITUATION AND LAYOUT FOR RAILROAD CROSSINGS

GENERAL

The Situation and Layout for Railroad crossings shall be the same as for Highways & Waterways with the addition of the following data.

PLAN VIEW

- Show the railroad milepost at the intersection with the highway and direction of increasing railroad milepost.
- Show the limits of the railroad Right of Way.
- Show the Township, Range, and Section data.
- Identify each track as mainline, siding, spur, etc. and show track spacing dimensions.
- Show all access roadways.
- Show the point of minimum horizontal and vertical clearance and the distance from the nearest track.
- Show any utilities crossing the structure and show the location of any deck drains. No deck drains will be installed over the railroad right of way. Show all utilities in the vicinity of the railroad (if none, a note should state "No known utilities").

ELEVATION VIEW

- Show the minimum vertical clearance taken from the top of rail.
- Distance between the top of pier footings and top of rail.
- Show the abutment slope protection and call out the slope perpendicular to the railroad.
- Show Railroad drainage ditches. (Make sure they are unobstructed by the construction)
- Show existing and future track locations.
- Show limits of fencing and protective railing or splashboards.

TYPICAL SECTION

- Identify superstructure structural members.
- Show type of railing/fencing and height.

RAILROAD CONSTRUCTION CLEARANCES

- Show an Elevation view that has minimum horizontal and vertical construction clearances.

PROFILE DATA

- Show the railroad profile grades at top of rail for 1000' each side of the structure.

DESIGN NOTES & GENERAL NOTES

The Notes shown on pages B17.1A thru B17.1D of the Bridge LRFD Manual are intended to be used as a checklist for the usual situation and should be modified to fit each individual case.

Revisions:

June 2006	Article was renumbered to 17.3 to allow for new article 17.1.
April 2008	Referenced new Design & General Notes drawings B17.1A – B17.1D.

CADDS DETAILING STANDARDS

SHEET FORMAT
Bridge plans prepared for ITD shall use the sheet format shown on page B17.3 of the Bridge Design Manual.

SHEET SCALE GUIDE
To increase the size of a 34" x 22" bordered sheet to fit over the smallest scaled object, set the active scale or "AS=" to the sheet size required.

ACTIVE SCALE	SHEET SCALE	SHEET SIZE
AS = 1200	1" = 100'	3400' x 2200'
AS = 720	1" = 60'	2040' x 1320'
AS = 600	1" = 50'	1700' x 1100'
AS = 480	1" = 40'	1360' x 880'
AS = 360	1" = 30'	1020' x 660'
AS = 240	1" = 20'	680' x 440'
AS = 120	1" = 10'	340' x 220'
AS = 60	1" = 5'	170' x 110'
AS = 4	3" = 1'	11'-4" x 7'-4"
AS = 8	1 ½" = 1'	22'-8" x 14'-8"
AS = 12	1" = 1'	34'-0" x 22'-0"
AS = 16	3/4" = 1'	45'-4" x 29'-4"
AS = 24	1/2" = 1'	68'-0" x 44'-0"
AS = 32	3/8" = 1'	90'-8" x 58'-8"
AS = 48	1/4" = 1'	136'-0" x 88'-0"
AS = 64	3/16" = 1'	181'-4" x 117'-4"
AS = 96	1/8" = 1'	272'-0" x 176'-0"
AS = 128	3/32" = 1'	362'-8" x 234'-8"

Numbers are valid when MU = ft, SU = 12 in, & PU = 1600

TEXT & TERMINATOR SCALE GUIDE
Text size based upon equivalent Ames Lettering Guide. Font is "ROMANS" (15º slant). Font = 24

	LT =TERM	#5	#6	#7
SCALE	TS =	ft in	ft in	ft in
1" = 100'	15.62	15 7.5	18 9	21 10.5
1" = 60'	9.37	9 7.5	11 2.5	13 1
1" = 50'	7.81	7 9.75	9 4.5	10 11.25
1" = 40'	6.25	6 3	7 6	8 2
1" = 30'	4.69	4 8.25	5 7.5	6 6.75
1" = 20'	3.12	3 1.5	3 9	4 4.5
1" = 10'	1.56	1 6.75	1 10.5	2 2.25
1" = 5'	0.78	0 9.375	0 11.25	1 1.125
3" = 1'	.05	0 0.625	0 0.75	0 0.875
1 1/2" = 1'	.10	0 1.25	0 1.5	0 1.75
1" = 1'	.16	0 1.875	0 2.25	0 2.625
3/4" = 1'	.21	0 2.5	0 3.0	0 3.5
1/2" = 1'	.31	0 3.75	0 4.5	0 5.25
3/8" = 1'	.42	0 5.0	0 6.0	0 7.0
1/4" = 1'	.63	0 7.5	0 9.0	0 10.5
3/16" = 1'	.83	0 10	1 0	1 2
1/8" = 1'	1.25	1 3	1 6	1 9
3/32" = 1'	1.67	1 8	2 0	2 4
1/16" = 1'	2.5	2 6	3 0	3 6

Numbers are valid when MU = ft; SU = 12inches; PU = 1600

#5 Dimensioning text – including sheet index, Quantities, etc.
#6 Titles for: Views, Lists, Diagrams, Data
#7 Sheet Title

LEVELS, LINE WEIGHTS, AND COLORS

ITEM	LEVEL	WEIGHT	COLOR	LINE CODE
Object Lines	1	3	0	0
Break Lines	1	2	1	0
Construction Joints	1	1	3	0
Hidden Lines / Reference Lines	2	0	0	5
Rebar	3	2	6	0
Dimensioning Text / Notes	4	1	2	0
Dimensioning Lines	4	0	2	0
Subtitles / Sheet title	4	2/3	4	0
Detail Centerlines	5	0	4	7
Roadway Centerlines	5	1	4	7
Contour Lines (Major)	6	2	8	0
Contour Lines (Minor)	6	0	8	0
Riprap / Gabion	6	0	8	0
Cross Hatching Lines	7	0	10	0
Concrete Symbol	7	0	38	0
Active Points	8	4	14	0
Phantom Lines	9	1	11	6

COLOR	DESCRIPTION
0	White (Plots Black)
1	Blue
2	Chartreuse Green
3	Red
4	Yellow
5	Purple
6	Orange
7	Dark Brown
8	Light Brown
9	Blue-Green
10	Aqua Blue
11	Lavender
12	Lima Bean
13	Dark Blue
14	Faded Red
15	Faded Chartreuse
38	Tan
187	Gold Yellow

LINE CODE	DESCRIPTION
0	Continuous
1	Dot Dash
2	Medium Dash
3	Long Dash
4	Centerline (Long Dash/Short Dash)
5	Short Dash
6	Phantom Line (Long Dash/2 Short Dash)
7	Centerline (Long Dash/Medium Dash

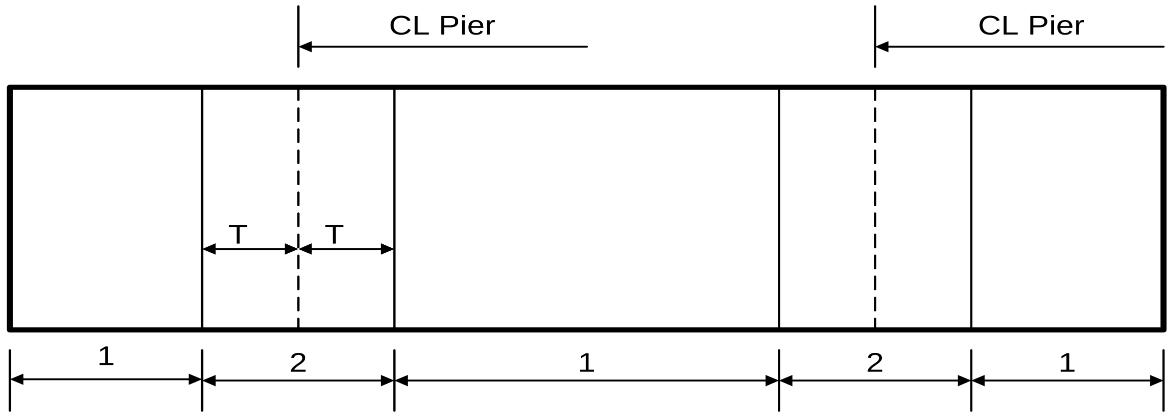
PLOT LINE THICKNESS-inches		
WEIGHT	22”x 34”	11”x 17”
0	0.0075	0.0035
1	0.0125	0.0060
2	0.0175	0.0075
3	0.0225	0.0110
4	0.0375	0.0180

Revisions:
June 2006 Article was renumbered to 17.4 to allow for new article 17.1.

DECK PLACING SEQUENCE

A deck placing sequence diagram and notes shall be shown on the plans for all multi-span bridges. Continuous placement shall not be allowed.

STANDARD SEQUENCE FOR BRIDGES DESIGNED AS SIMPLE SPANS OR DESIGNED CONTINUOUS FOR LIVE LOAD WHEN GIRDERS ARE AT LEAST 90 DAYS OLD.



T = 20% - 25% of the adjacent span length

NOTES
1. No deviation from the deck placing sequence shown will be permitted.
2. Areas marked 1 shall be placed before areas marked 2, but areas of the same number need not be placed simultaneously.
3. Placement of areas marked 2 shall not commence until at least 48 hours have elapsed after completion of placement of areas marked 1.
4. When the deck slab is continuous over the piers, the pier diaphragm shall be placed simultaneously with the adjacent deck area.

BRIDGES DESIGNED CONTINUOUS FOR LIVE LOAD WHEN GIRDERS ARE LESS THAN 90 DAYS OLD

When the deck is placed on girders that are less than 90 days old from the time they are cast, any rational deck placement sequence (other than continuous placement) may be used provided the girders are designed in accordance with Article 5.14.1.4.3 for the loading, creep and shrinkage that result from the sequence that is chosen.

Commentary

Refer to Article 5.14.1.4 -Simple Span Precast Girders Made Continuous

In order to mitigate the development of positive moments at the piers due to girder creep, the girders are required to mature for 90 days to prevent excessive creep after the deck is placed. In accordance with the AASHTO Specifications this allows the same placing sequence as girders designed as simple spans without the need to calculate the effects of creep as is required for girders less than 90 days old.

Revisions:

June 2006	Article was renumbered to 17.5 to allow for new article 17.1.
August 2006	The deck placement sequence for bridges designed continuous for live load was revised to reflect changes that were adopted by AASHTO 2006 ballot item #13.

SITUATION AND LAYOUT CHECKLIST FOR HIGHWAY / WATERWAY CROSSINGS

PROJECT NAME: ________________________________
PROJECT KEY NUMBER: ________________________
BRIDGE DRAWING NUMBER: ____________________ **CHECKED BY:** ____________

Use pencil to mark items. Use an X or ✓ to indicate completion. Use "INC" to indicate items which are incomplete and "N/A" to indicate items which do not apply. For additional information on the design requirements refer to Chapter 17 of the "LRFD" Manual.

BORDER

- ☐ Designed and Detailed Names
- ☐ Design Checked and DWG Checked Names (required when work has been checked)
- ☐ Corrections Name (need only be completed when corrections have been made)
- ☐ Engineers Stamp (For Full and Half sized Sheets)
- ☐ Project Number
- ☐ Sheet Title
- ☐ Project Description (Length, Type of Support, Crossing, Station)
- ☐ Bridge Inspection Master Key (Only required on sheet 1)
- ☐ Bridge Drawing Number (required but may not be available during preliminary design)
- ☐ Project County and Key Number
- ☐ Sheet Numbering (required for final design and PS&E submittals)

PLAN VIEW

- ☐ View Title with scale factor
- ☐ Length of Structure (out to out) along survey line
- ☐ Station and Finished Grade Elevation at the Beginning and End of structure along Centerline.

Abutment / Pier number, Station, and Finished Grade Elevation shown at the Intersection of the Abutment / Pier Centerline and Survey line at the following locations:

- ☐ Centerline of bearing of Abutments
- ☐ Center of Piers / Bents

Span lengths along survey line shown as follows:

- ☐ Single Spans or End Spans: abutment centerline bearing - centerline pier/bent
- ☐ Interior Spans: centerline pier/bent - centerline pier/bent
- ☐ Bridge Width shown (out - out). Width should include the parapet, curb and sidewalk as applicable.
- ☐ Curb-to-Curb Width shown
- ☐ Roadway Lane and Shoulder Widths shown
- ☐ Lane Direction and Name of Closest Town/Geographical Feature in that Direction indicated
- ☐ North arrow shown
- ☐ Intersection Angle shown if not a 90º crossing

horizontal and vertical clearances shown as follows:

- ☐ Highway Crossings: Show the point of minimum vert. and horiz. clearance for the highway
- ☐ Stream Crossings: Show the point of minimum clearance above Q50 high water elevation
- ☐ Identification of Survey and Profile lines
- ☐ Existing Bridge Details shown (as needed)
- ☐ Existing Bridge Drawing Number given (Needed only if existing bridge is to be removed)
- ☐ Plan View Oriented so Elevation View can be placed below Plan View
- ☐ Bridge Stationing at Centerline of Structure shown and runs Left to Right of sheet
- ☐ Culvert Stationing at Centerline of Structure shown and runs Bottom to Top of sheet
- ☐ Rip Rap Limits shown with pay note (as applicable)
- ☐ Contour lines shown and gray shaded
- ☐ Utilities Crossing the structure shown (as applicable)
- ☐ Deck drains shown (as applicable)
- ☐ Survey Cap shown with installation note

* Horizontal Alignment Data should be included in this view if possible. See Horizontal Alignment Data Checklist for items to be included.

ELEVATION VIEW

- [ ] View Title with scale factor
- [ ] Total length between abutment centerlines along survey line shown

Abutment/Pier Number and Station shown at the following locations:

- [ ] Centerline Bearing of Abutments
- [ ] Centerline of Piers/Bents
- [ ] Span Length Shown
- [ ] Span Number Shown (Multi-Span Structures only)
- [ ] Fixity Shown ("E" Expansion, "P" Pinned, or "F" Fixed) (not required on stifflegs)

Minimum Vertical Clearances shown as follows:

- [ ] Highway Crossing: Minimum Clearance from roadway
- [ ] Stream Crossing: Minimum Clearance form Q_{50} High Water Elevation
- [ ] Ground Line along the Centerline of Structure Shown
- [ ] Abutment Slopes shown and annotated
- [ ] Abutment / Pier Projection lines shown (Do not show where projection lines may be confusing)
- [ ] Roadway approach Guardrails shown with associated note

PROFILE DATA

- [ ] View Title with scale factor
- [ ] Profile Grade Across Structure Shown
- [ ] Structure Location Shown on Profile
- [ ] Station and Elevation for the Beginning and End of Structure Shown
- [ ] Profile Grades for all Highways involved in Crossing Shown

The following Vertical Curve Data Shown:

- [ ] Stations and Elevations at Point of Curvature, Point of Intersection, and Point of Tangency
- [ ] Length of Vertical Curve
- [ ] Incoming and Outgoing Grades as a percent

HORIZONTAL ALIGNMENT DATA

* Horizontal Alignment Data should be included in the Plan view if possible.

- [ ] View Title
- [ ] Stations and Elevations at Point of Curvature, Point of Intersection, and Point of Tangency Shown
- [ ] Horizontal Curve data Shown (Δ, T, L, R, S, RL, and Z)
- [ ] Horizontal Curve described in Degree of Curve
- [ ] Super Elevation Transition Data Shown (If possible)
- [ ] Alignment Bearing (Should be shown in Plan View if possible)

HYDRAULIC DATA

- [ ] View Title

Hydraulic Data for Streams and Rivers shown for the following conditions:

- [ ] Design (Flood, discharge, H.W. Elev., and Velocity)
- [ ] Base (Flood, discharge, H.W. Elev., and Velocity)
- [ ] Scour (Flood, discharge, H.W. Elev., and Velocity)
- [ ] Hydraulic Data for Canals Shown (Canal Flow, H.W. Elev., Velocity, and Flow Controller)

TRAFFIC DATA

- [ ] View Title
- [ ] Traffic Data for Construction Year and 20 years past Construction year Shown (Current ADT, Future ADT, Current ADT % Trucks, Future ADT % Trucks, Design Speed)

INDEX OF SHEETS

- [ ] View Title
- [ ] Sheet number and Sheet Title Shown for all Sheets

QUANTITIES

- [ ] View Title
- [ ] Bid Item Number, Description, and Unit Shown for all applicable items
- [ ] Bid Item Quantity Shown (Not Required until Final Design)

DESIGN AND GENERAL NOTES

The notes shown on B17.1- B17.2 are intended as a design and detailing aid. Only those notes required should be shown on the plans. Notes should be modified or added to match the requirements of each structure.

The values required for the Footing & Pile Design Loads are to be furnished as follows:

X Geotechnical Engineer

xx Bridge Designer

Revisions:
April 2008 Added new article.

CHAPTER 17
STANDARD DRAWING REVISION LOG

B17.1A Design and General Notes – concrete girder railroad bridge

April 2008	Added new sheet.
July 2009	Deleted IC from Extreme Event loads

B17.1B Design and General Notes – concrete girder railroad bridge

April 2008	Added new sheet
July 2009	Added Method B for Elastomeric Bearings Deleted MSE Wall note “Designed in accordance…”.

B17.1C Design and General Notes – steel girder railroad bridge

April 2008	Added new sheet
July 2009	Deleted IC from Extreme Event loads

B17.1D Design and General Notes – steel girder railroad bridge

April 2008	Added new sheet
July 2009	Added Method B for Elastomeric Bearings Deleted MSE Wall note “Designed in accordance…”.

B17.2A Design and General Notes – concrete girder highway bridge

June 2006	Revised Construction Specification note to “Project plans & specifications”. Revised Design Procedures notes to list only proprietary software programs and added Design Speed. Added Single Lane ADTT to Transient Loads because traffic data was deleted from the Situation Layout sheet. Revised Footing Design Loads format to show nominal bearing resistance value and the effective footing area/width.
April 2008	Denoted drawing for use on a highway bridge. Added reference to 2008 Interims Changed Seismic data to agree with 2008 Interims Modified Footing & Pile Design loads
July 2009	Added Method B for Elastomeric Bearings Deleted MSE Wall note “Designed in accordance…”.

B17.2B Design and General Notes – steel girder highway bridge

June 2006	Revised Construction Specification note to “Project plans & specifications”. Revised Design Procedures notes to list only proprietary software programs and added Design Speed. Added Single Lane ADTT to Transient Loads because traffic data was deleted from the Situation Layout sheet. Revised Footing Design Loads format to show nominal bearing resistance value and the effective footing area/width.
April 2008	Denoted drawing for use on a highway bridge. Added reference to 2008 Interims Changed Seismic data to agree with 2008 Interms Modified Footing & Pile Design loads
July 2009	Added Method B for Elastomeric Bearings Deleted MSE Wall note “Designed in accordance…”.

B17.2C Design and General Notes – concrete culverts

June 2006	Revised Construction Specification note to “Project plans & specifications”. Revised Design Procedures notes to list only proprietary software programs and added Design Speed. Added Fill Depth and Live Load Surcharge to Permanent Loads. Revised Footing Design Loads format to show nominal bearing resistance value and the effective footing area/width.
April 2008	Added reference to 2008 Interims Modified Footing Design loads

CHAPTER 17
STANDARD DRAWING REVISION LOG

B17.2D Design and General Notes – metal pipes

June 2006	Revised Construction Specification note to "Project plans & specifications".
	Revised Footing Design Loads format to show nominal bearing resistance value and the effective footing area/width.
April 2008	Added reference to 2008 Interims
	Modified Footing Design loads

B17.3 Sheet Index, Quantities, & Vicinity Map

April 2008	Added new sheet.

B17.4 Consultant Plan Sheet

June 2006	Renumbered sheet from B17.1 to B17.3.
	Added Note 7 for electronic stamp note levels.
April 2008	Renumbered sheet from B17.3 to B17.4.